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Improving proposals for design standards of vertical cylindrical steel tanks for oil and oil products storage

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The calculation of subsidence points along the contour and in the center of the tank bottom is complicated in the case of erosion on inhomogeneous, subsidence, damaged soils, mules, flooded and seismically dangerous areas. The article analyzes the most dangerous geological phenomena, processes and complicated geotechnical conditions for the territory of Ukraine. The method of an artificial base arrangement with improved properties due to vertical reinforcement by soil cement elements is considered. Techno-economic comparison of the proposed variant with the usual pile variant of foundations (subsidence soils, estimated seismic intensity of 9 points) has been carried out. Probabilistic analysis of the artificial base has been carried out and the corellation between the probability of its failure from the percentage of reinforcement has been determined.

Keywords: complex geotechnical conditions, seismic and dynamic effects, seismic resistance, oil storage tank, artificial base, soil-cement elements, probabilistic design, stressed-deformed state, finite elements method, random variables

Пропозиції щодо вдосконалення норм проектування резервуарів вертикальних циліндричних сталевих для нафти і нафтопродуктів

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Визначено, що проектування резервуарів вертикальних сталевих (РВС) для нафти і нафтопродуктів на території України регламентується ДСТУ Б В.2.6-183:2011, який за своєю суттю замінив ВБН В.2.2-58.2-94. З’ясовано, що цей нормативний документ дає загальні рекомендації щодо вибору конструктивних рішень, навантажень і впливів при проектуванні резервуарів різного об’єму, але не враховує досягнення сучасних досліджень розрахунку напружено-деформованого стану (НДС) системи «основа – фундаменти – резервуар» у складних інженерно-геологічних умовах, особливо при динамічних і сейсмічних впливах, а також при влаштуванні штучних основ. Визначено, що головною специфікою цих розрахунків є той факт, що при проектуванні резервуарів один із основних факторів – це правильне визначення переміщень (осідань) точок по контуру та по центру його днища; такий розрахунок ускладнюється при зведенні резервуарів на неоднорідних, просадочних, заторфованих, насипних і намивних ґрунтах, мулах, підтоплених та сейсмічно небезпечних територіях. Проаналізовано найбільш характерні для території України небезпечні геологічні явища і процеси та складні інженерно-геологічні умови. Розглянуто методику влаштування штучної основи з поліпшеними властивостями за рахунок вертикального армування ґрунтоцементними елементами за бурозмішувальною технологією. Проведено техніко-економічне порівняння запропонованого варіанта зі звичайним пальовим варіантом фундаментів при будівництві резервуару на просадочних ґрунтах і при розрахунковій сейсмічній інтенсивності в 9 балів. Значну увагу приділено ймовірнісному аналізу штучної основи й визначено взаємозв’язок імовірності відмови від процента армування ґрунтової основи.

Ключові слова: складні інженерно-геологічні умови, сейсмічні та динамічні впливи, сейсмостійкість, нафтовий резервуар, штучна основа, ґрунтоцементні елементи, імовірнісний розрахунок, напружено-деформований стан, метод скінченних елементів, випадкові величини

Introduction. Steel tanks of various capacities intended for collection and storage of oil are important elements in the technological process of its extraction, preparation and transportation. In addition, they are used for pre-dehydrating oil on production field or as buffer tanks in oil trunk transport, or at petrol stations for collecting, storing and distributing various petroleum products.

Reservoirs often are built in territories with complex geotechnical conditions [1]: 1) soils with special properties (subsidence and peat (water-saturated), mule, bulk and alluvial massifs, soils which, under certain conditions, their volume increase, etc.); 2) the possibility of dangerous geological (gravitational) processes development (karst, landslides, erosion, suphosion, abrasive processes, etc.), as well as of non-uniform deformations of soil foundations during water saturation, mining operations, etc. Distribution of the main dangerous engineering geological and geological factors and processes that complicate tanks for oil and petroleum products construction and operation, on the territory of Ukraine is shown in Fig. 1 – 5 (according to the State Service of Geology and Subsoil of Ukraine and according to seismological studies [2, 3]).

The accidents with tanks for oil and petroleum products are followed by the outflow of huge masses of liquid that leads to catastrophic consequences and pollution, large-scale fires, violation of normal modes of operation the objects of transportation and storage of oil and oil products, and to significant environmental pollution and serious economic consequences also.

The main causes of accidents can be [4]: a) defects of welded joints; b) distortion of the shell shape because of its installation low quality or poor-quality of the foundation; c) the effect of low ambient air temperature; d) vibrational effects of pumps when pumping liquid; e) non-uniform subsidence of the base; f) local subsidence of the base; g) erosion of the base layer with a liquid in the case of bottom corrosion damage; h) emergency (including seismic) effects.

The degree of object damage during the earthquake depends not only on the level of seismic influences, but also on the quality of seismic design and construction. According to recent seismological studies [3], it has been established that in the territory of Ukraine, including its platform part, there is a danger of local and strong earthquakes with magnitude greater than 5 (more than 6 points on the MSK-64 scale). It creates an additional risk of existing and new oil tanks exploitation.

Figure 1 – Distribution of landslides within the administrative regions of Ukraine
(according to the State Service of Geology and Subsoil of Ukraine, http://www.geo.gov.ua)

Figure 2 – Distribution of flooding within the administrative regions of Ukraine
(according to the State Service of Geology and Subsoil of Ukraine, http://www.geo.gov.ua)

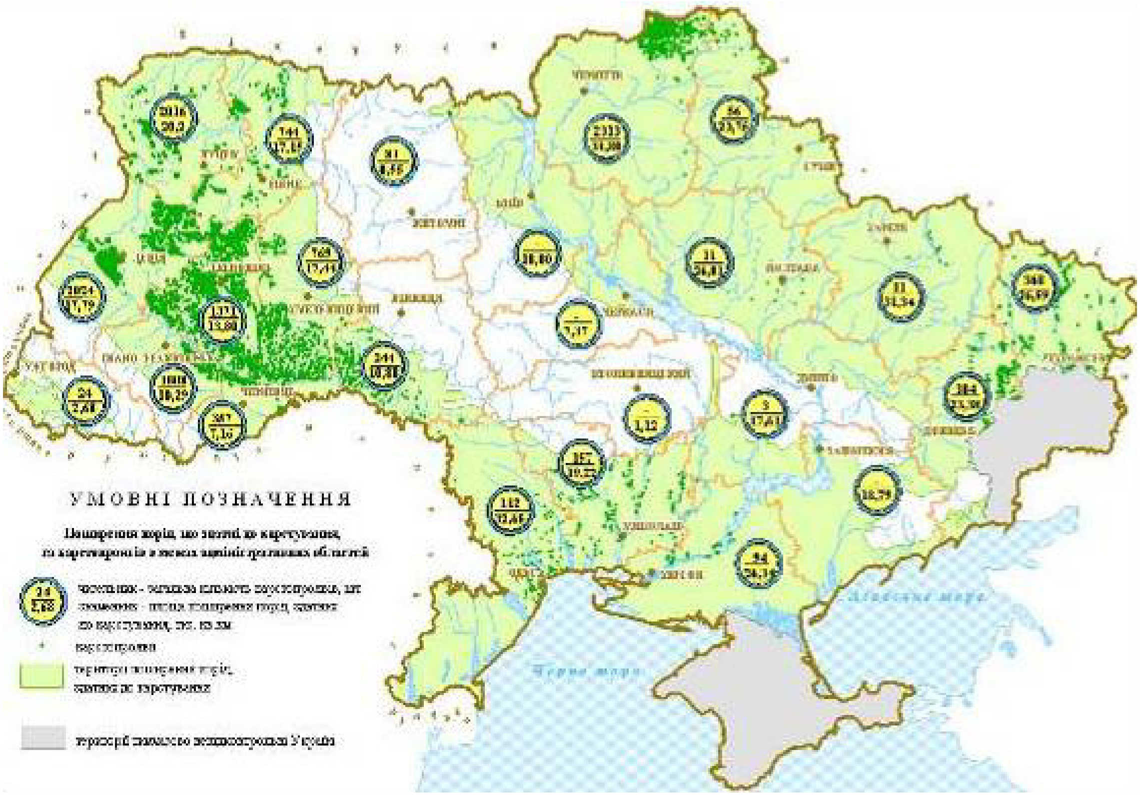


Figure 3 – Distributionof karsting rocks and of karsting within the administrative regions of Ukraine
(according to the State Service of Geology and Subsoil of Ukraine, http://www.geo.gov.ua)

Figure 4 – Distribution of loess soils within the administrative regions of Ukraine
(according to the State Service of Geology and Subsoil of Ukraine, http://www.geo.gov.ua)

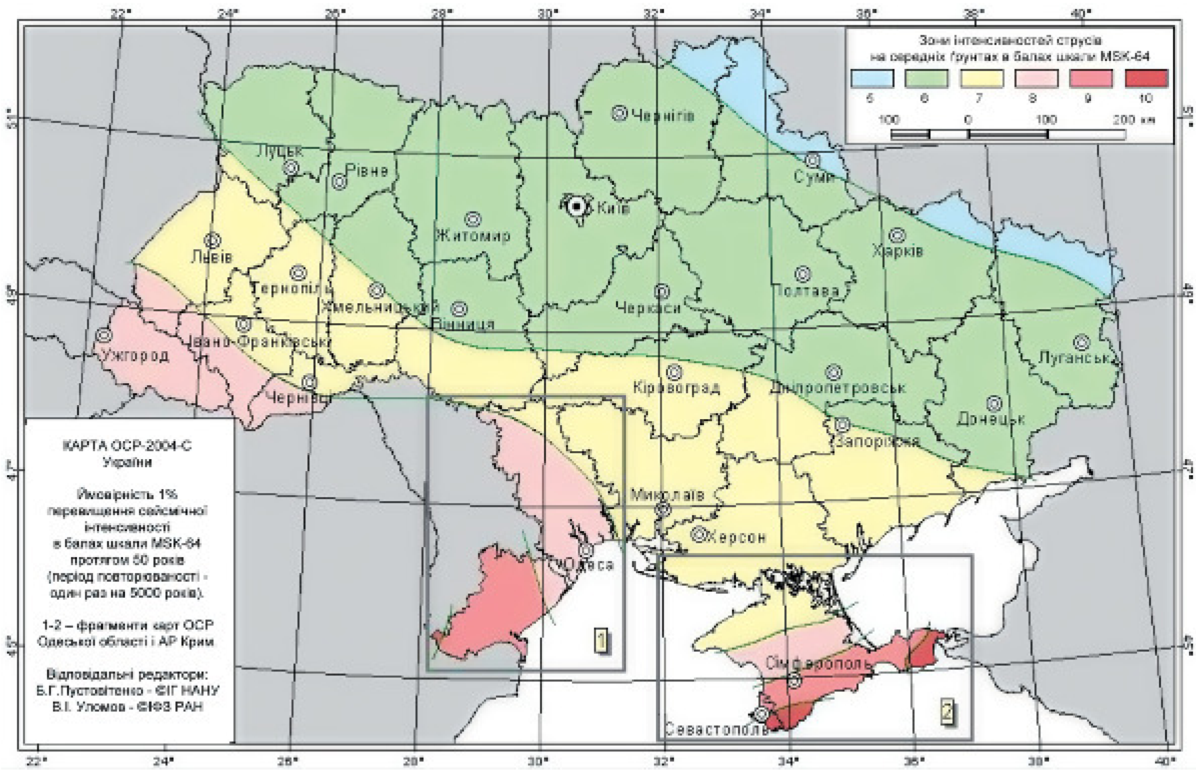


Figure 5 – Map of GSZ-2004-B general seismic zoning on the territory of Ukraine
(according to the data of SBC V.1.1-12: 2014) for high-risk facilities

In addition, modern technological requirements for tank battery lead to increase in the volumes of the TVS (tank vertical steel). At the same time, the pressure from the TVS, that is transmitted to the soil base, also significantly increases compared to the accumulated experience of these structures operation. Therefore, the cost of modern TVS construction in complicated engineering and geological conditions is increasing significantly.

It should be noted that according to [5] oil and oil product tanks are classified as high-risk objects (CC3 class of consequences). It, in turn, requires extremely high attention when designing and constructing these objects especially in difficult conditions. This situation requires development of qualitatively new geotechnical technologies that minimize the risks and ensure the safe operation of modern TVs, especially in difficult geotechnical conditions. Therefore, the improvement of design standards and the development of reliable methodologies for analysing such objects is definitely an urgent task.

Analysis of recent sources of research and publications. The studies of stress-strain state (SSS) of TVS are considered in papers [6], peculiarities of work under non-uniform deformations of foundations – [4], with seismic influences – [7]. In all these works, SSS of tanks is considered separately from the bases and foundations.

In papers [8 – 14] the results of changes investigations in the properties of soils during their soil-cement reinforcement, incl. dynamic. It should be noted that laboratory and field studies of deformation characteristics of soils, obtained mainly by specialists from Japan and the USA, determined that in conditions of significant seismic loading deformation characteristics of the soil become non-linear. Analysis of world and national experience in using different methods to reduce the dynamic and vibrational effects on weak, water-saturated, structurally unstable soils showed that the most effective means for their artificial transformation is cementation with the help of jet grouting or boring and mixing technologies [11 – 18]. Their main feature is that they enable to strengthenpractically the entire range of soils from gravel deposits to fine clays, mules, turfs. In this case, the destruction and simultaneous mixing of the soil with cement mortar in the mode of «mix-in-place» (mixing in place). Between the solid particles there are strong bonds due to binders, which greatly increase the strength of the soil and reduce its compressibility.

The effect of stabilization (strentghening, reinforcement) of the bases is that in a certain volume of weak soil a part of it is replaced by a rigid material – a soil cement, with a relatively large deformation module (E = 70 – 200 MPa). Natural soil, sandwiched between the created vertical soil-cement elements (SCE), also increases its mechanical parameters due to the impossibility of lateral protrusion.

Identification of general problem parts unsolved before. Today, the accumulated experience of operating the TVS in complex engineering-geological conditions on artificial grounds is still insufficient, especially when attak of such a dangerous geological phenomena as an earthquake. Therefore, the purpose of the work is to analyze the geotechnical decisions of the construction of the TVS on subsidence and weak soils in seismic areas, to develop an effective form of artificial earthquake-resistant foundations and to develop a method for its analysing, to estimate the probability of tank failure on artificial bases, to implement the development of design standards TVS.

Basic material and results. As a rule, tank subsidence occurs evenly, however, in difficult geotechnical conditions, the possibility of meeting design specifications during their operation due to non-uniform deformations of the foundation base is complicated.

Almost immediately after the hydraulic tests there is an non-uniform subsidence between the central part and the wall of the TVS due to different specific pressure on the soil of the wall weight and hydrostatic load. Pressure under the wall varies within 0.9 – 1.5 MPa, and under the central part of the bottom no more than 0.16 – 0.2 MPa. From the practice of the tank exploitation, there are cases when the difference of subsidence between the central and peripheral parts of the bottom reaches 0.6 – 0.8 m.

As it is known, non-uniform subsidence on the bottom area and on its perimeter cause additional deformations in the structural elements of the TVS, especially in the lower junction of the wall and the edges, and the additional stresses associated with them. The combination of operational stresses with additional from non-uniform subsidence can lead to the damage in the junction area or rupture of the bottom panel. Also, with a significant deformations of the cylindrical shell, possible damage in the junction of the stationary roof, wall and support ring, pontoon disfunction .

One of the effective methods of subsidence leveling in terms of the bottom area and its perimeter is the construction of an artificial base with interchangeable deformation characteristics. For the construction of this base is proposed a vertical reinforcement of soil-cement elements. The modulus of deformation of the created artificial base is considered weighted. Its value can be controlled by changing the distance between SCE.

Depending on the loads and engineering-geological conditions, determine the required number and length (up to 40 m) of SCE. A grid of SCE can be made with different steps without any interference or with their «intersection». Such an approach provides normative subsidence and trouble-free operation of the TVS in the most complicated engineering and geological conditions.

Another unresolved problem of normative documents is the earthquake resistance of TVSs, taking into account the improved soil properties.

For a better understanding of the problem of reducing seismic hazards, lets consider the main aspects of the theory of the seismic waves propagation in soil fields and seismic geotechnics. The transfer of wave energy from point to point occurs due to the elastic properties of the medium, therefore the stress wave is elastic. In the process of propagation of the wave, a part of its energy is lost, which leads to a decrease in the intensity of the dynamic load with a distance from its source and is called attenuation. The causes of the attenuation are different and are associated mainly with nonideal elasticity, discreteness and heterogeneity of any soils structure as a medium of propagation of elastic waves. By means of different mechanisms of energy loss, the following types of attenuation are distinguished: 1) the difference is caused by the decrease of the specific energy per unit of the wave area front in connection with its increase as the distance from the source; 2) dispersion in various heterogeneous media, which leads to a decrease in the energy of the wave in a particular direction; the dispersion of waves on any obstacle depends on its shape and size, as well as on the density and compressibility of the obstacles substance; 3) the absorption is due to the energy consumption of plastic and nonlinear-elastic deformation. Consequently, the presence of various obstacles and weak inhomogeneous soils in the path of a seismic wave leads to reflection and absorption of its energy that reduces seismic intensity.

The task of vibration analysis of the system «base – foundations – construction» is complicated by poorly predicted effects of resonant strengthening of seismic vibrations by loose near-surface soils: depending on the type and thickness of vibration layers of some frequency intervals can be selectively amplified, while others are almost completely absorbed. This phenomenon is due to the violation of the natural vibrations of the stratum near the free surface in the waves of this type. That is, the upper layers of the soil change the parameters of seismic waves, coming from the depths, and, thus, can change (both increase and decrease) the intensity of seismic vibrations depending on their dynamic properties. The level of ground water (water saturation) can produce an effect on seismic intensity in the case when water changes the physical and mechanical properties of soils, which should manifest itself in changing their elastic properties, in particular the velocity of transverse waves. Therefore, in order to design earthquake-resistant building is required not only information about the strength and location of possible earthquakes, but reliable data on forced vibrations of structures on certain soil substrates. To do this, certain characteristics of the soil, such as its dynamic compression and shear modulus, the damping factor, are determined, including the forecast of changes in soils properties during the operation of the structure, and depending on them, one of the possible models of basement soil behavior is taken for the analysis. One of the most important dynamic properties of soils is their seismic stiffness $V_s\rho$ (where V_s – the velocity of transverse waves propagation, ρ – soil density). At the same time, the higre $V_s\rho$ of the soil active layer is, the smaller the amplitude of its vibrations is.

In terms of dynamics, the thickness of the active layer is determined by the frequency characteristics ratio of the soil layer and the structure. At the same time, the most unfavorable in seismic terms is the case – the intersection of the frequency characteristics bandwidth of soil mass with its structure own oscillation frequencies. Thickness H of the soils that significantly effect the vibrations of earthquakes, is determined by the ratio $H=V_s/4f$ (where V_s – the average velocity of transverse waves in the soil layer; f – the value of the bandwidths intersection low frequency part of the soil layer frequency characteristics and the structure internal vibrations). If the frequency parameters of the soil layer and the internal vibrations of the structure do not overlap, there is a weak link between these processes. During seismic vibrations seismic waves occur. Longitudinal waves lead to compression and stretching of soil particles, while transverse waves cause tilt or lateral displacement. At a lateral displacement there is a danger of cohesive strength violation between particles of soil. Therefore, there is an additional problem – the ability of soils to change their mechanical properties when passing elastic waves through them. The essence of the effect lies in the fact that the soils consist of small and the smallest particles, in intervals (pores), between which there is water and gases. All resistance of such soil to external loading is carried out by means of a huge number of contacts between these particles, many of which are very weak. When the elastic wave passes, vibrations of soil particles with different velocities are excited, and the part of the contacts is broken. In addition, when a wave moves from a more dense to a loose soil layer and a substantial (twice or larger) increase in the amplitude of the vibration, adhesion failure becomes real. As a result, the strength of the soil significantly (sometimes several times) decreases. At the same time the soil becomes «liquid», this phenomenon is called «liquefaction» of the soil (it is in a state of suspended water). Water thus seeks to squeeze out, but this process requires some time, since it is limited to the permeability of the soil.

The most important task in analysing the vibrations of the system «base – foundation – structure» in all forms is the forecast of its resonance frequencies and peak displacement amplitudes, which are considered as the limiting (most unfavorable) conditions of the work of the structure. The fact is that in the spectrum of the seismic wave there are vibrations with frequencies close to the natural frequency of a number of structures (characteristic periods from 0.2 to 2 s). In the case of resonance, the stresses on the contact of the foundation with the soil sharply increase, as well as in the construction of the structure and the probability of destruction of this system increases.

The influence of the soil base on seismic fluctuations of the structure has a number of aspects: 1) through it the seismic effect on the structure is transferred (the structure, due to its massiveness and rigidity, has a reverse effect on the movement of the soil); 2) the base has its own mass and rigidity, which reduces the frequency of natural vibrations of the dynamic system «structure-base» (with increasing mass and stiffness of the soil the amplitude decreases and the frequency of vibration of the base increases); 3) during an earthquake, seismic waves are reflected from the foundation and dissipated in the basis.

The last two factors influence the size of the dynamic response of the structure, and therefore the seismic inertial loads that have an effect on on the structure.

The design of cylindrical TVS is a vertical, thin-walled cylinder, which is limited with the bottom and the roof. At the bottom of the reservoir lying on the base, there are relatively small stresses under pressure of the load from the liquid contained in it. In this case, the thickness of the bottom is 4 ... 12 mm and is mainly due to the conditions of the technology of assembly and welding works and corrosion resistance of the metal.

During the design of tanks in seismic areas with intensity higher than 6 points it is necessary to consider the additional requirements: 1) use of tanks with lower height; 2) in tanks with a floating roof or a pontoon to apply locks of soft type; 3) in case of usage the tanks with a stationary roof it is necessary to carry out calculation of the maximum height of filling of the tank with liquid to avoid hydrodynamic blow in a roof the wave arising in the tank from a horizontal push; 4) special devices (compensators) should be provided in the pipiline entry units with shut-off valves that ensure the strength and reliability of the mentioned unit.

The presence of liquid in the tank leads to a change in its own vibrations and forms of construction, the additional hydrodynamic pressure on the walls and bottom of the reservoir. In this case, for the thin-walled reservoirs, the hydrodynamic calculation is basic, since the mass of the liquid is much larger than the mass of the tank itself. The calculation determines the level of stresses in a tank wall and evaluates a surface wave height that arises during vibrations (to avoid spill from the tank and impact on the roof). For tanks with a floating roof or a pontoon one should consider horizontal inertial forces from a floating roof or a pontoon. That is, it is necessary to calculate various hydrostatic systems that simulate the tank for seismic influences, which are given by accelerograms. In general, reservoirs are expected to withstand the rollover and shifts from wind loads, non-uniform deformations of the foundations or seismic influences. At the next stage, the main and auxiliary structures of the tank are calculated. Tanks foundation is calculated for two groups of limit states: 1) ultimate limit state – on the bearing capacity for check of stability of tanks on overturning; 2) serviceability limit state – on deformations (absolute vertical settlements of the center and a contour circle of the foundation, differential settlements of ground taking into account local moistening of collapsible thickness, tilt).

In earthquakes conditions there is an addition to external vibrations – component of the seismic load from vertical soil vibrations, occurs further loads of the product on the wall and bottom of the tank (Fig. 6) – namely: 1) hydrostatic loads and loads of overpressure; 2) impulsive (inertial) component of hydrodynamic pressure; 3) convection (kinematic) component of hydrodynamic. The impulsive component of pressure arises from a part of the product moving in an earthquake together with a tank wall. Liquid sloshing in a tank create convective pressure and leads to emergence of waves on a product surface. Vertical vibrations of a tank base also induce additional load of its wall.

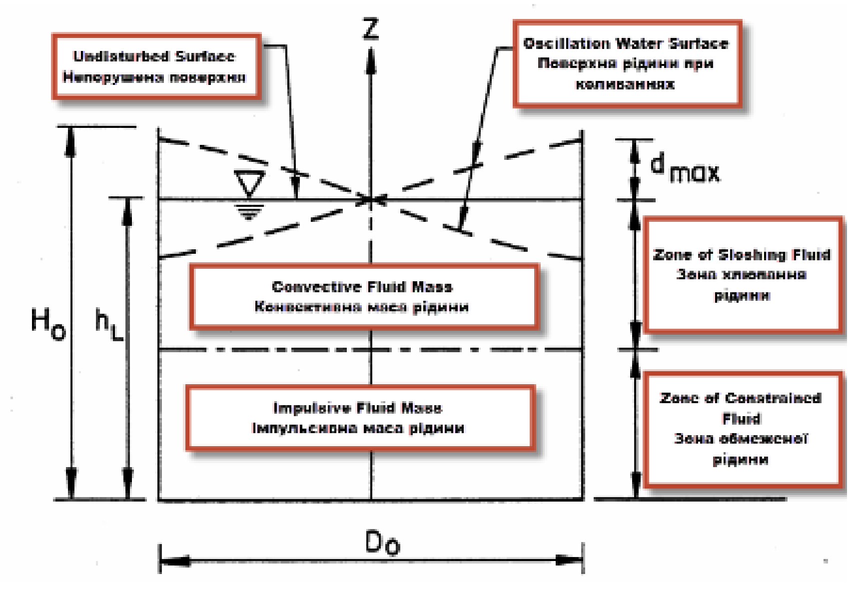


Figure 6 – The scheme of liquid sloshing in a cylindrical tank

Tank seismic resistance is considered to be provided if: a) the tank does not overturn during an earthquake (overturning criterion is the limit state at which on the external radius of the lifted part of the bottom a full plastic hinge appears); b) stability of the lower belt of a wall at action of longitudinal and cross loading is provided; c) durability condition for all bearing elements of the tank is provided. There are several concepts for reducing the seismic hazard by arranging pile foundations or improving the properties of soil bases by reinforcing them with vertical SCEs.

Concept number 1. Limitation of damage from liquefaction «limiting soil seismic shock absorber» (Fig. 7) is a seismic and extrusion technology to mitigate shocks during shocks during earthquakes, which includes a solid soil mass from the whole soil-cement wall. Such a structure can effectively limit the deposition and lateral displacement of the soil, as well as the displacement of structures.

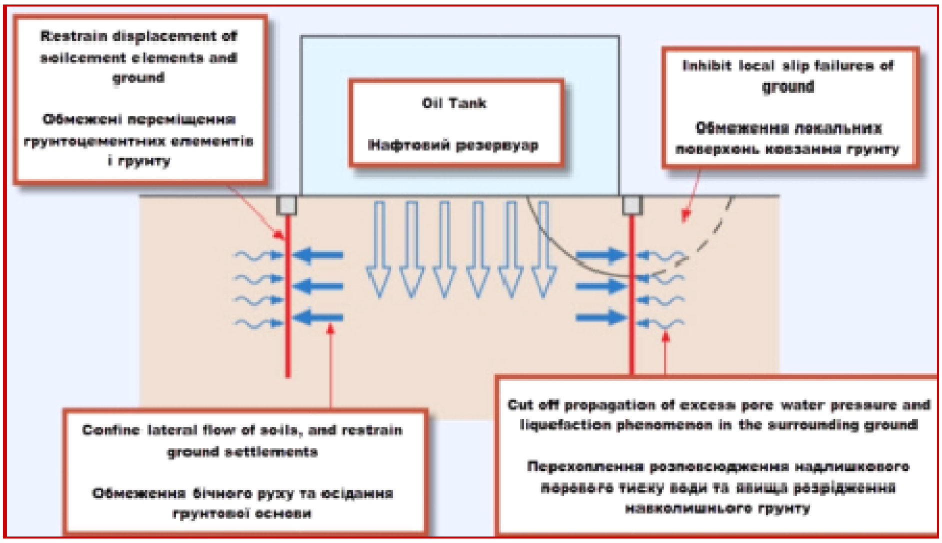


Figure 7 – Concept № 1 – Damage limitation from liquefaction «limiting soil seismic shock absorber»

Concept number№ 2. The method of limiting the lateral displacement of the soil due to its SCE reinforcement. Lateral displacement of the soils caused by their liquefaction during the earthquake and the horizontal displacement and subsidence of the tanks are reduced due to the SCE reinforcement of the ground in chessboard method from the existing containment bund, which protects the tanks with oil and petroleum products. Such technology further prevents the leakage of contaminated liquid from damaged pipelines.

Concept number № 3. Method of solid soil-cement lining. Soil liquefaction during an earthquake is limited by the use of a «limiting soil seismic shock absorber», resulting in the soil being in a closed solid cement wall. The leakage of contaminated liquid due to the displacement of the tank or damage to the pipeline is prevented by reducing both the values of foundations base subsidence of, and its unevenness.

Concept № 4. Method of SCE enclosing. The sedimentation and loss of stability of the oil-protective soil dam caused by earthquakes, as well as the leakage of contaminated liquid outside the dam, may be limited or prevented by the arrangement of the soil cement wall along the contour of the dam.

In addition, the soil liquefaction during an earthquake is limited by the use of a «limiting soil seismic shock absorber», through which the soil turns out to be a closed solid soil cement wall. This can prevent leakage of contaminated liquid due to displacement of the tank or damage to the pipeline.

*Concept number 5. Method of continuous artificial soil cement base (*Fig. 8). Reducing the influence of dynamic load on the aboveground part of the structure during the earthquakes due to the increase of elastic deformation parameters of the base.

Reducing the effect of the dynamic load on the aboveground part of the structure during the earthquakes can be achieved by reducing the acceleration and the vibration amplitude of the base of its foundations. One of the ways to reduce seismic intensity is to increase seismic impedance $V_s\rho$ of the active layer of soil by increasing the velocity of propagation of seismic waves in it. Such an effect can be achieved by increasing the elastic deformation characteristics of the base, using the boring and mixing technology [11, 15].

With this approach it is possible to increase the modulus of elasticity of the base to 500 – 2000 MPa, the rate of propagation of waves up to 600 – 1000 m/s at constant density.

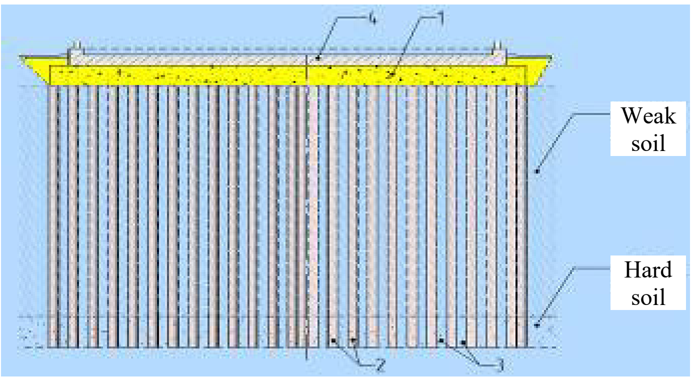
Improvement of the damping characteristics of an artificial base occurs due to the presence of a gravel or sand pillow, as well as due to the presence of friction and adhesion of sandwitched soil on the surface of the SCE contacts. Therefore, the vertical components of the seismic impulse (vertically directed tremors) the pillow damps and only in a weakened form, transfers to the base slab. In the zone of artificial base, vertical SEEs partially absorb and dissipate the energy of a seismic wave (dissipation) in a manner similar to a pile foundation. The elimination of thixotropic properties and properties of soil liquefaction occurs due to local cementation within the SCEs and the increase in the resistance of the soil shift between these elements. It also increases the resistance of the filtration consolidation of the weak base due to its reinforcement. The main idea of the authors is the development of a universal artificial base, which will provide regulatory and technological requirements for both the static conditions of TVS exploitation on subsidence and weak soils, as well as in the case of seismic influences of varying intensity. As an example, geotechnical solutions for the oil tank are given TVSI-20000 of oil pumping station «Avgustivka» (Avgustivka village, Biliaivka district, Odessa region).

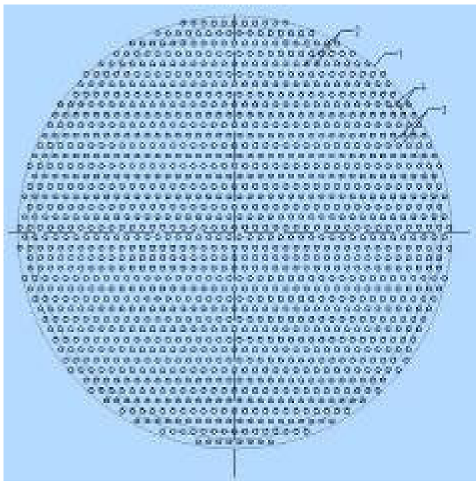
Technological parameters of TVS-20000 presented in the Tab. 1.

Geometric parameters TVS-20000:
1) nominal volume 20000 m^3;
2) geometric volume 20956 m^3;
3) height of the wall 17.926 m;
4) inner diameter 39.9 m;
5) area of the product mirror 1250.4 m^2.

The diameter of the base slab is approximately 40.5 m. Pressure under the bottom of the base slab at hydro testing is 168.14 kPa, at operation – 180.86 kPa. The uniformly distributed load along the contour of the foundation during testing is 31.65 kN/r.m., at operation – 40.33 kN/r.m., at wind loads – ±6 kN/r.m., at seismic influences – +353,73/-268.67 kN/r.m. The magnitude of the equilibrium of horizontal seismic forces transferred from the construction of the tank to the base slab was 65.500 kN.

The weak soil of the base in the case of earthquake action can go into a liquid state (to obtain a thixotropic properties), to obtain an additional compacting, resulting in excessive deformations of the tank. The complexity of the second site is characterized by the presence of a subsidence thickness of more than 5 m. The estimated seismic intensity of the plot is 9 points.





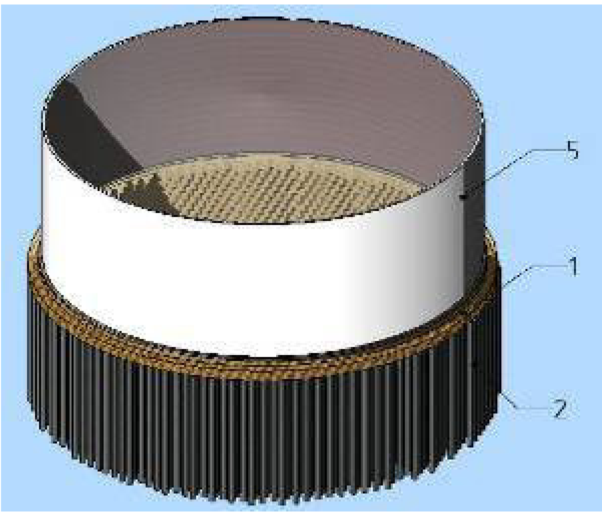


Figure 8 – Dynamically stable soil cement base:
a – section of dynamically stable soil cement base;
b – plan of the reinforced base;
c – spatial view of the structure on a reinforced base;
1 – sand or gravel-pebble pillow;
2 – vertical soil cement elements;
3 – natural soil, sandwitched between vertical soil cement elements;
4 – foundation slab;
5 – aboveground structure

Table 1 – Technological parameters of oil storage tank TVSI-20000

Parameters	Value
Product density (oil), t/m^3	0.89
Expected level of product filling, m	16.2
Water level at hydrotest, m	16.75
Internal overpressure	missing
Standard internal vacuum	missing
Operation rate (cycles per year), min/max	20/100
Characteristic value of snow load, kg/m^2	102
Characteristic value of wind load, kg/m^2	51
Seismic intensity, points	8
The temperature of the coldest days with the use factor of 0,98, ^{0}C	- 24
Maximum temperature of oil storage, ^{0}C	+ 25
Design service life, years	40
Design wave altitude of oil at seismic loadings, m	0.32
The size of an allowance for corrosion for sheets of a wall, mm	0
Minimum marking of surface edge in regard to the surface of the crust, mm	+0.5

At these conditions, several geotechnical decisions were considered:

1) cutting of the soil weak and subsidence mass with reinforced concrete piles in the section 350x350 mm for their resistance to strong soils, as well as the elimination of the sedimentary properties of the soil of the inter-piles space by deep compaction (intallation of soil piles in diameter of 300 mm), errection of the foundation wall of 0.7 m thick over the piles, a swing joint of piles with a slab;

2) the same as in the first variant, but on top of the piles is assumed that the gravel pillow is arranged in order to damper the fluctuations of the reservoir and avoid the transfer of horizontal loads to the piles;

3) arrangement of the artificial foundation by reinforcing the weak and subsidence soil with vertical SCE with a diameter of 500 ... 650 mm, then the same as in the second variant.

The perception of horizontal seismic loading due to the work of piles on the soil is provided only with a significant amount (~ 1000 pcs.), Which is economically inexpedient, since the vertical load on the piles will not be more than 35% of the permissible. Therefore, this option was not further compared.

The variant with vertical SCE was much cheaper and could be realized over a shorter period of time.The length, the diameter and the step of SCEs were determined by the iterative method. The main criterion for calculating was the providing of less than the critical values of center subsidence and the extreme calculation points of the base slab, the tank tilt, as well as the load bearing capacity of the underlying seismic influences. As a result of calculations it was established that the optimal diameter of SCE is 500 mm, step – 1,0 m (2d).

Table 2 shows comparison of two variants of the foundations.

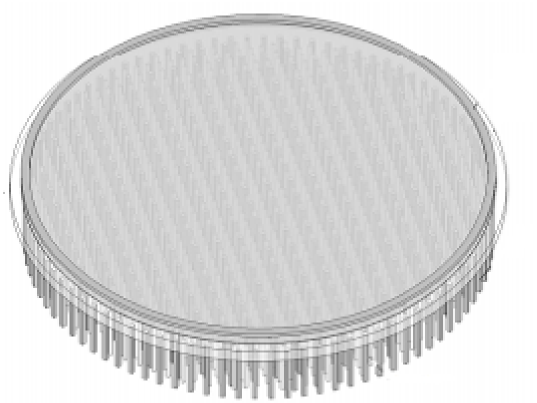
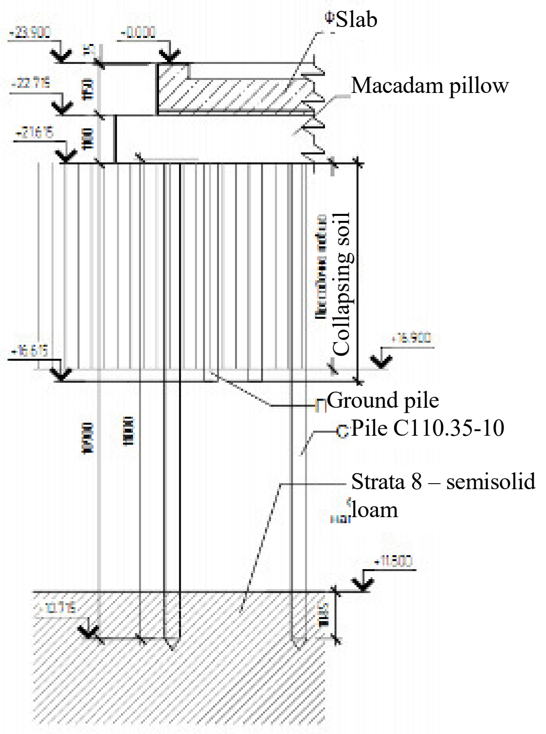
From the Table 2 is evident that variant with the use of the artificial base, by reinforcing the weak and subsidence soil with vertical SCEs for one of the TVS-20000, an economic effect of more than 2.7 million hryvnas was achieved and allows to shorten the construction period to 38 days.

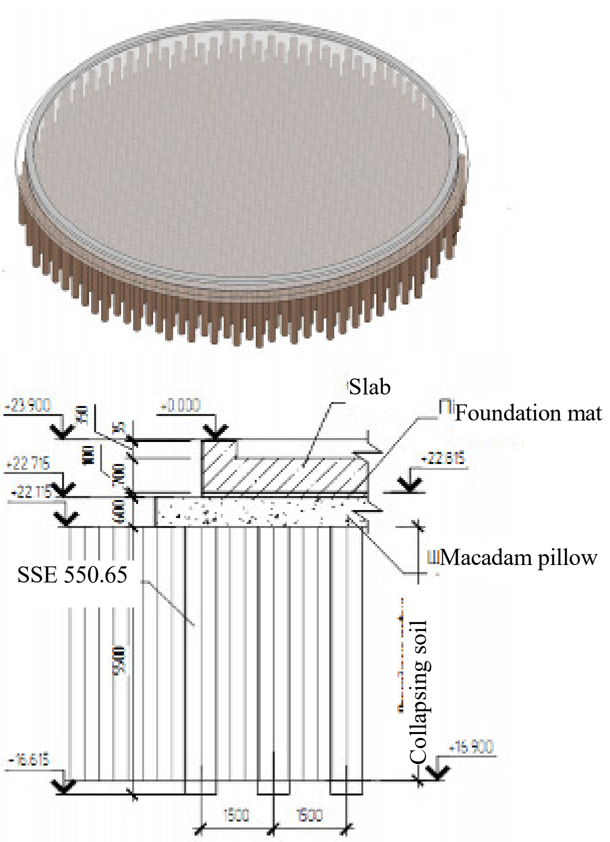
The authors also performed the probabilistic calculation of the system «artificial base – foundations – tank». Distribution and statistical parameters of the RV (random value) of the TVS foundation slab tilt are shown in Fig. 9 (the value of the tilt is multiplied by 10^{-4}). With subsequent loading of the tankss, the absolute subsidence of the base increases and is clearly fading. At the same time, the probability of failure on the criterion of subsidence does not exceed 0.001, and according to the criterion of the tilt – 0.07.

On the basis of statistical data simulation of the reinforced base subsidence and foundation slab tilt, the probability of construction failure and faultless operation according to the criteria of absolute settling (Fig. 10) and the tilt is determined, depending on the percentage of SCR weak base reinforcement. By simulation, it was found that the probability of failure on the criterion of a tiltl depending on the percentage of reinforcement (15 ... 25%) of SCE weak base ranged from 0.03 to 0.05.

The probability of failure of the construction according to the criterion of the maximum allowable foundation slab weight is less than 0.01. Based on the results of simulation for the reliability level p = 0.9, the minimum required percentage of reinforcement of the base is selected VSCE (i = 19%). This task has shown that with the help of a probabilistic approach, the necessary percentage of weak base reinforcement is substantiated to ensure the failure-free operation of the structure during its exploitation.

Table 2 – Techno-economic comparison of foundations TVS-20000

<table>
<tr><th>№</th><th>Name and scheme of the foundation variant</th><th>Performance time</th><th>Cost of work and materials</th></tr>
<tr><td>1</td><td>2</td><td>3</td><td>4</td></tr>
<tr><td rowspan="10">1</td><td rowspan="10">Cutting of the subsidence thicket of the soil by reinforced concrete piles and their reliance on strong soils, elimination of the collapsing properties of the soil of the inter-tidal space by deep seals
(the arrangement of ground piles)


</td><td colspan="2">1. Reinforced precast-concrete piles of length 11 m and of section 350x350 mm – 374 piles.</td></tr>
<tr><td>32 calendar day</td><td>Without delivery – 2 079 000 UAH.
Work –
1 247 400 UAH.</td></tr>
<tr><td colspan="2">2. Ground piles of diameter 300 mm and of length 5 m – 917 piles.</td></tr>
<tr><td>45 calendar day</td><td>450 000 UAH</td></tr>
<tr><td colspan="2">3. Macadam pillow H = 1100 mm.</td></tr>
<tr><td>11 calendar day</td><td>176 000 UAH</td></tr>
<tr><td colspan="2">4. In-situ concrete slab.</td></tr>
<tr><td>11 calendar day</td><td>3 200 000 UAH</td></tr>
<tr><td>**Total –
128 calendar day**</td><td>**Total cost –
7 152 400 UAH**</td></tr>
</table>

1	2	3	4
2	Arrangement of the artificial bases by reinforcing the collapsing soil with soil-cement elements 	1. Soil-cement elements of diameter 650 mm and of length 5.5 m – 616 piles.	
		40 calendar day	1 124 230 UAH
		2. Macadam pillow H = 600 mm.	
		10 calendar day	96 000 UAH
		3. In-situ concrete slab.	
		40 calendar day	3 200 000 UAH
		Total – 90 calendar day	**Total cost – 4 420 230 UAH**
	Economic effect	**38 calendar day**	**2 732 170 UAH**

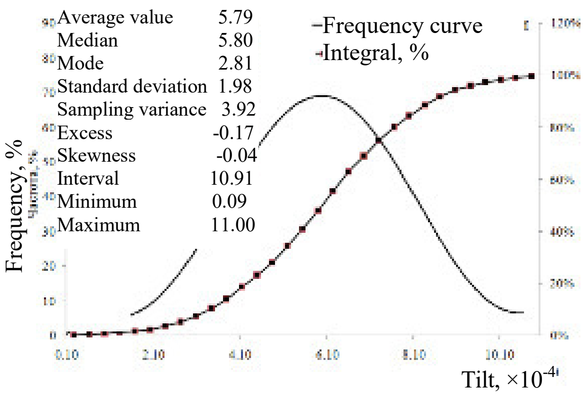


Figure 9 – Distribution of tilt RV of tank foundation slab on the results of the simulation of the MCE (the value of the tilt is multiplied by 10^{-4})

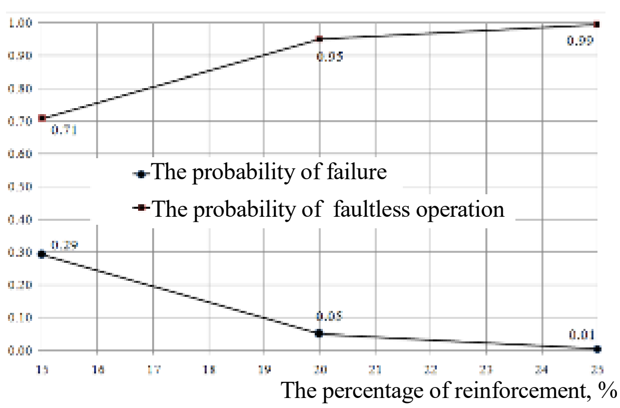


Figure 10 – The probability of failure and failure-free operation of the structure on the criterion of absolute settling, depending on the percentage of the base reinforcement

As a result, it is possible to draw the following **conclusions:**

1. Normative documents for the design of TVS should be supplemented with modern methods of the system «base – foundations – tank» analysis, technologies for vertical reinforcement of soils with specific properties, etc.

2. With consistent calculation of the soil base, foundations and tank, it is possible to consider more accurately the SDS of the entire system. The probability of design failure according to the criterion of the tilt and absolute deformation is satisfied with the percentage of reinforcement in the range of 15 ... 20 %.

3. In the arrangement of artificial foundation, the method of SCE vertical reinforcement is the ability to regulate the deformability of the base and in accordance with the alignment of subsidence of the central part of the bottom and points along the contour of the tilt in accordance with actual stresses and engineering-geological properties.

4. The variant of the foundation slab on an artificial soil cement base, which transforms the weak and subsidence thickness into a composite material, is earth-resistant, worth less than a pile and is more technologically efficient. All the technological and regulatory requirements for the exploitation of tanks are met.

5. Due to the reinforcement of a part of the weak and subsidence base soil mass, the amplitude of the vibrations of the reservoir decreases, the nature of the acceleration of the soil changes, and the values of soil acceleration in the pit bottom level drop. This result is ensured by increase in the velocity of seismic waves spread in an artificial soil cement base, as well as by increasing the strength and deformability characteristics.

6. At seismic influences with intensity of 9 points the maximum horizontal displacement of the tank top didn't exceed 6 mm, a bottom – 10 mm. The difference of bottom concerning top displacement is 16 mm that less than 20 mm. The tank does not overturn; shear strength of foundation relatively of crushed-stone pillow is achieved.

References

1. ДБН В.1.1-45:2017. (2016). *Будівлі і споруди в складних інженерно-геологічних умовах. Загальні положення*. Київ. Мінрегіонбуд України.

2. ДБН В.1.1-12:2014. (2014). *Будівництво у сейсмічних районах*. Київ. Мінрегіонбуд України.

3. Кендзера, А.В., Егупов, К.В., Марьенков, Н.Г. и др. (2015). Сейсмическое микрорайонирование строительных площадок для сейсмостойкого проектирования зданий и сооружений в сейсмических районах Украины. *Наука та будівництво*, 4, 12-18.

4. Чепур, П.В. (2015) *Напряженно-деформированное состояние резервуара при развитии неравномерных осадок его основания*. (Автореф. дис. канд. техн. наук). Российский государственный университет нефти и газа имени И.М. Губкина, Москва.

5. ДБН В.1.2-14-2009. (2014). *Загальні принципи забезпечення надійності та конструктивної безпеки будівель, споруд, будівельних конструкцій та основ*. Київ. Мінрегіонбуд України.

6. Коновалов, П.А., Мангушев, Р.А., Сотников, С.Н. и др. *Фундаменты стальных резервуаров и деформации их оснований*. Москва: АСВ.

7. Jeong, G.H., Heon, J.P., Moon, K.L., Heyrim, L., Dong-Soo, K., Sunyong, K. & Hyun-uk Kim (2017). *Seismic behavior of LNG storage tank considering soil-foundation-structure interaction with different foundation types,* Proc. of the 19th Intern. Conf. on Soil Mechanics and Geotechnical Engineering (Sep. 17 – 22, 2017 / COEX, Seoul, Korea). Retrieved from https://www.issmge.org

8. Selvaraju, S., Wei He, Z. & Weng Leong K. (2017). *Vibro replacement stone columns for large steel storage tanks in Vietnam*, Proc. of the 19th Intern. Conf. on Soil Mechanics and Geotechnical Engineering (Sep. 17 – 22, 2017 / COEX, Seoul, Korea. Retrieved from https://www.issmge.org

9. Абрамова, Т.Т. & Вознесенский, Е.А. (2015). Современные методы управления свойствами грунтов на участках высоких динамических нагрузок. *ГеоТехника*, 4, 6-25.

10. Вознесенский, Е.А., Кушнарева, Е.С. & Фуникова, В.В. (2014). *Природа и закономерности затухания волн напряжений в грунтах*. Москва: ФЛИНТА.

11. Zotsenko, N., Vynnykov, Yu. & Zotsenko, V. (2015). Soil-cement piles by boring-mixing technology. *Energy, energy saving and rational nature use*. Oradea University, 192-253.

12. Зоценко, М.Л., Винников, Ю.Л. & Зоценко, В.М. (2016). *Бурові ґрунтоцементні палі, які виготовляються за бурозмішувальним методом*. Харків: Друкарня Мадрид.

13. Kramer, S.L. (1996). *Geotechnical Earthquake Engineering*. New Jersey: Prentice Hall, Upper Saddle River.

14. Kryvosheiev, P., Farenyuk, G., Tytarenko, V., Boyko, I., Kornienko, M., Zotsenko, M., Vynnykov, Yu., Siedin, V., Shokarev, V. & Krysan, V. (2017). *Innovative projects in difficult soil conditions using artificial foundation and base, arranged without soil excavation,* Proc. of the 19th Intern. Conf. on Soil Mechanics and Geotechnical Engineering (Sep. 17 – 22, 2017 / COEX, Seoul, Korea). Retrieved from https://www.issmge.org.

15. Vynnykov, Yu., Voskobiinyk, O., Kharchenko, M. & Marchenko, V. (2017). *Probabilistic analysis of deformed mode of engineering constructions soil-cement grounds,* MATEC Web of Conf. Proc. of the 6th Intern. Scientific Conf. «Reliability and Durability of Railway Transport Engineering Structures and Buildings» (Transbud-2017). https://doi.org/10.1051/matecconf/201711602038.

16. Ganne, P., Denies, N., Huybrechts, N., Vervoort, A., Tavallali, A., Maertens, J., Lameire, B. & De Cock F. (2011). *Soil mix: influence of soil inclusions on structural behaviour,* Proc. of the 15th European Conf. on Soil Mechanics and Geotechnical Engineering (Athens, 2011). doi:10.3233/978-1-60750-801-4-977

17. Ezaoui, A., Tatsuoka, F., Furusawa, S., Yirao, K. & Kataoka, T. (2013). *Strength properties of densely compacted cement-mixed gravelly soil,* Proc. of the 18th Intern. Conf. on Soil Mechanics and Geotechnical Engineering (Paris, 2013).

18. Hor, B., Hyun Jee, S., Jun Song, M. & Young Kim, D. (2017). *Ground improvement using rigid inclusion for the foundation of LNG tanks,* Proc. of the 19th Intern. Conf. on Soil Mechanics and Geotechnical Engineering (Sep. 17 – 22, 2017 / COEX, Seoul, Korea). Retrieved from https://www.issmge.org.

19. ДБН В.2.1-10-2009. (2012). *Основи та фундаменти будинків і споруд. Основні положення проектування (зі змінами №1 і №2)*. Київ: Мінрегіонбуд України.

20. ДСТУ Б В.2.6-183:2011. (2012). *Резервуари вертикальні циліндричні сталеві для нафти та нафтопродуктів. Загальні технічні умови*. Київ: Мінрегіонбуд України.

21. ВБН В.2.2-58.2-94. (1994). *Резервуари вертикальні сталеві для зберігання нафти і нафтопродуктів з тиском насичених парів не вище 93,3 кПа*. Київ: Державний комітет України по нафті і газу (Держкомнафтогаз).

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Work piles - columns with soil under constant influence of vertical and cyclically approximated horizontal loads

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The article describes an experimental study aimed at identifying common patterns of joint work of piles - columns with soil with vertical and cyclically applied horizontal loads. The study examines the deformation processes occurring in the soil. At any pressure value, soil deformations can be divided into two groups, which are restored (elastic) and residual. When the pressure is less than the structural strength, elastic deformations appear. With a pressure of greater structural strength, elastic and residual deformations appear. Elastic deformations appear throughout the depth, residual deformations develop in the depth of the deformation zone, where the stress exceeds the structural strength of the soil. After removing the load, the elastic deformation disappears, and the residual remains. The lower limit of the residual strain zone is at a depth, where the stresses from the load transmitted by the column of piles below its base are balanced by the structural strength of the soil.

Keywords: geological structure of the site, pile - columns, experimental study

Спільна робота палі-колони з ґрунтом основи при дії вертикального і циклічно прикладеного горизонтального навантажень

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Описано експериментальні дослідження, спрямовані на виявлення загальних закономірностей спільної роботи палі-колони з ґрунтом основи при постійно діючому вертикальному і циклічно прикладеному горизонтальному навантаженнях. Установлено, що при значенні тиску деформації ґрунту можуть бути розділені на дві групи: ті, що відновлюються (пружні), й залишкові. Доведено, що при напрузі, меншій ніж структурна міцність, виникають пружні деформації, при більшій – в основах спостерігаються два види деформації – пружні в межах усієї глибини зони і залишкові, що розвиваються в межах глибини зони деформації, де напруга перевищує структурну міцність ґрунту. Зазначено, що після зняття навантаження пружні деформації зникають, а залишкові залишаються. Нижня межа зони залишкових деформацій знаходиться на глибині, де напруги від навантаження, яке передається палею-колоною нижче її підошви, врівноважуються структурною міцністю ґрунту. Пружні деформації наростають по глибині за лінійним законом, при залишкових деформаціях залежність між деформаціями і напруженнями буде нелінійною. Визначено повне осідання і залишкову складову деформації, величини яких зі збільшенням горизонтального навантаження зростають. Установлено, що величинам пружної складової осідання, горизонтальних переміщень дорівнюють різниці між величинами повної та залишкової складових деформації. Виявлено, що під дією горизонтального навантаження паля-колона повертається в точці нульових переміщень – у результаті ґрунти перед навантаженою гранню колони ущільнюються, наслідком ущільнення є переміщення палі: її поворот у рівні денної поверхні в бік дії сил, а підошви – у зворотному напрямку.

Ключові слова: геологічна будова майданчика, паля-колона, експериментальне дослідження

Introduction. The determination of deformations occurring in soils is of great importance for the practice of designing foundations, since deformations of foundations are the factors that characterize the durability of structures.

Under the influence of loads in the soil mass, stresses arise and, as a result, their deformation develops in soils. The formation of tension in the soil column does not occur instantaneously under load influence, but can develop for a rather long time. There are three phases of soil stress state [1]:

Phase 1 – the phase of compaction, damped deformations, when the deformation speed tends to zero.

Phase 2 – phase shifts, the strain speed acquires a constant value, some local shifts occur at the edges of the foundation; soil carrying capacity is not exhausted yet.

Phase 3 – the bulging phase is a sharp increase in precipitation with a slight increase in load magnitude, as well as soil swelling to the sides and upwards.

O.N. Tsytovich considered in his works not three phases of the stressed state of soils arising under the basement foundations with a constant increase of load on the ground, but two: the phase of compaction and local shifts and the phase of development of significant shifts [2].

In the article the deformations that develop in the soil under the pile-column bottom, corresponding to the stress state arising in the first phase, the compaction phase are reseatched.

The analysis of recent research and publications. A great contribution to the study of the soils deformation processes arising in the foundations have been made the works of the following scientists: Tugayenko Y.F., Marchenko M.V., Tkalicha A.P., Mosicheva I.I. [3, 4, 5]. In the work Tugayenko Y.F. [3] summarizes the studies results of increase in deformation at the base of piles, foundations and pile foundations. The studies results on the formation of a compaction zone during pile driving and the development of a deformation zone due to external loads are presented. According to the test results, the value of the piles settlement, its elastic and residual components were recorded, the facts of changes in the deformations in depth were presented, and their quantitative assessment was given.

The emphasizing of previously unsolved parts of the general problem to which the article is dedicated. In the above presented works, the deformation of soils arising under the action of vertical force is considered. In this article, the deformations that develop during the testing of the pile-soil system under the joint action of vertical and cyclically applied horizontal loads are studied.

The formulation of the problem. The purpose of the work is to identify the general patterns of joint work of the pile - columns with the foundation soil. The task of the study is to conduct a full-scale test of a pile-column with a foundation soil under the action of loads.

The main material and results. A short-pile column was tested on one of the construction sites. The structure of the site is characterized by the following engineering-geological elements: layer 1. Bulk grounds, represented by brown and dark brown loams with rubble, construction and household debris, with a capacity of 0.1 m; layer 2. Loams are dark brown, macroporous, with a capacity of 3.8 m (E = 18/7 MPa; p = 1.69 g/cm^3; p_d = 1.43 g/cm^3; w = 0.18; $\varphi = 23^0$; s = 10 kPa); layer 3. Yellowish-brown, loess-like, macroporous loam, 4.0 m thick (E = 29 MPa; p = 2.03 g/cm^3; p_d = 1.68 g/cm^3; w = 0.21; $\varphi = 26^0$; c = 15 kPa). During the exploration ground waters were found at a depth of -6.3 m.

The geometric parameters of the tested pile: cross section 0.3 ⋅ 0.3 m, length – 5 m, depth of immersion in the ground – 3 m. The experimental pile is made with broadening. Broadening is an increase in the cross section of the underground part of the pile in the level of the surface. The cross-section of broadening is 1.0 ⋅ 0.8 m.

The installation method – a pile-column is driven into the ground with the help of a pile driver S-878 A based on the tractor T-100 MBGP, hammer C-330.

The test was carried out in full size under the action of vertical force and horizontal loads (Fig. 1), in the laboratory such studies were carried out using an elastomer [6].

The vertical load was created by placing the calibrated load on a special platform, which is mounted on a pile-column. Loading was carried out uniformly, one step at full size.

After stabilization of the precipitation from the vertical force, horizontal loads, which were created by a winch, attached to the anchor pile column, were applied. Horizontal loads were applied in steps, each step of the load was maintained until the horizontal displacements stabilized, after which full unloading was performed, then the next step of horizontal force was applied. The load in each cycle consisted of the sum of the previous and next steps, except for the first and second steps in which the horizontal forces are equal to Q = 17 kN.

For the conditional amount of stabilization in the study, it was assumed: increment speed: precipitation from a load of not more than 0.1 mm per day, horizontal displacements of 0.1 mm in the last two hours.

Horizontal forces were measured with a dynamometer with a scale division of 7.5 kg.

Measurements of the sediment from the loads were made using a depth gauge with a precision of 0.1 mm. The pile draft was determined by measuring the distance from the edges of a double-solo reference beam rigidly connected with the pile and supporting rods. The support rods were stained with concrete in the well to a depth of 1.0 m outside the zone of influence of the load on the base soil. The pile settlement was determined as the average of two measurements.

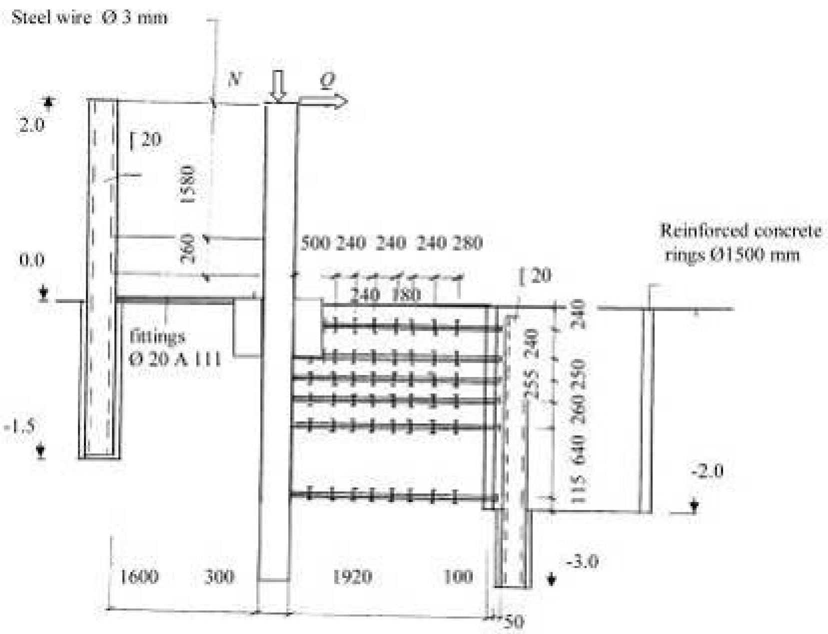

Figure 1 – The diagram of load application and installation of magnetic grades

Horizontal movements at the level of the day surface and along the height of the superfundum part were recorded using slider defibomers.

The horizontal displacements of the underground part of the pile were measured by the displacements of the tubes by the depth gauge from the reference system located in the pit, located at a distance of 2.0 m from the loaded face of the pile. Tubes were laid from a shurfa with a vertical step 240 ... 640 mm measurement accuracy of 0.1 mm.

The movement of the soil in front of the loaded face of the pile was measured by magnetic marks and a special device. Stamps were mounted in horizontal wells drilled from the hole in two rows: along the axis of the column with a vertical step of 240 ... 640 mm, to a depth of 2 m, horizontally 180 ... 280 mm and along the axis at a distance of 290 mm from the center columns. The movements of marks were recorded by the device with a reference accuracy of up to 0.01 mm relative to the reference system installed in the pit.

The results of the study. According to the experiment, a graph of precipitation versus permanent vertical and cyclically applied horizontal loads (Fig. 2), characterizing the dynamics of precipitation increase over time, was built. The graph shows the process of precipitation stabilization, from each stage of the horizontal load in the process of experience and after its completion.

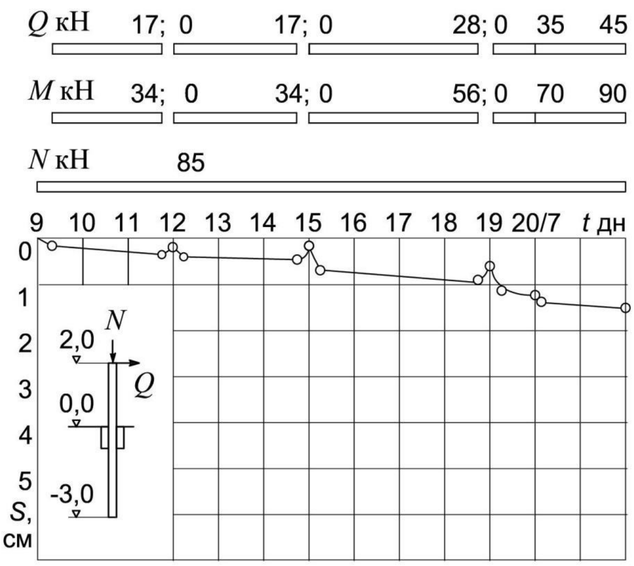

Figure 2 – The graph dependence of pile-column precipitation on the loads in time

If in the process of loading the soil, when some values are reached, unloading is performed, then it can be noted that, at any pressure value, the unloading does not cause a complete recovery of the soil deformation (Fig. 2). Consequently, at any pressure value, the deformations of the soil can be divided into two groups - regenerative (elastic) and residual.

At stress, less structural strength, elastic deformations occur, with greater, two types of deformation are observed in the bases - elastic within the entire depth of the zone and residual, developing within the depth of the deformation zone, where the stress exceeds the structural strength of the soil. After the removal of the load, the elastic deformations disappear, and the residual ones remain. The lower boundary of residual deformations zone is located at a depth, where the stresses from the load transmitted by the pile-column below its base are balanced by structural strength.

Elastic deformations are a consequence of the mutual repulsion forces between the atoms of the soil crystal lattice during their mechanical «approach»; the manifestation of the elastic properties of gases and liquids in the pores; the emergence of forces, «wedging» pressure in the soil [7, 8].

The residual deformations are the result of soil plastic properties manifestation, namely: the destruction of individual structural bonds during deformation; the displacement of particles (crystals, grains, debris, etc.); the squeezing of water and gases from the pores; the gradual accumulation of microdefects in the soil structure, which cannot be restored themselves [7, 8].

Elastic deformations increase in depth, according to a linear law, that is, the equations and dependences of the theory of elasticity can be used to determine stress, based on a linear relationship between stresses and strains, with residual strains, the relationship between strains and stresses will be non-linear. Residual deformations determine the irreversible processes occurring in the foundations of the foundations.

As a result of the study, the total sediment and the residual component of deformation were determined, the magnitudes of which increase with increasing horizontal load. With an increase in the horizontal force by 1.6 times, with a constant vertical load, the residual strain is 4 times greater at Q = 28 kN than at Q = 17 kN (Fig. 2).

Under the action of a horizontal load, the pile rotates at the point of zero-left displacements. As a result, the soil in front of the column loaded face is compacted, the result of compaction is the movement of the pile, its rotation at the level of the day surface towards the action of forces, and the soles in the opposite direction.

The rotation reduces the contact area of pile side surface with the ground, reducing the amount of pile resistance on the side surface. The reduction of friction forces causes an additional pile draft which value increases with increasing horizontal forces.

Figure 3 shows a graph of the horizontal displacement of a pile-column from the loads in time. The graph shows the process of horizontal displacements stabilization, from each stage of the horizontal load in the process of experience.

The magnitude of precipitation elastic component, the horizontal displacements is equal to the difference between the values of the deformation total and residual components.

Under the action of loads Q = 45 kN; M = 90 kN m; the elastic component is equal to 27% of the total precipitation and 29% of soil total horizontal displacement.

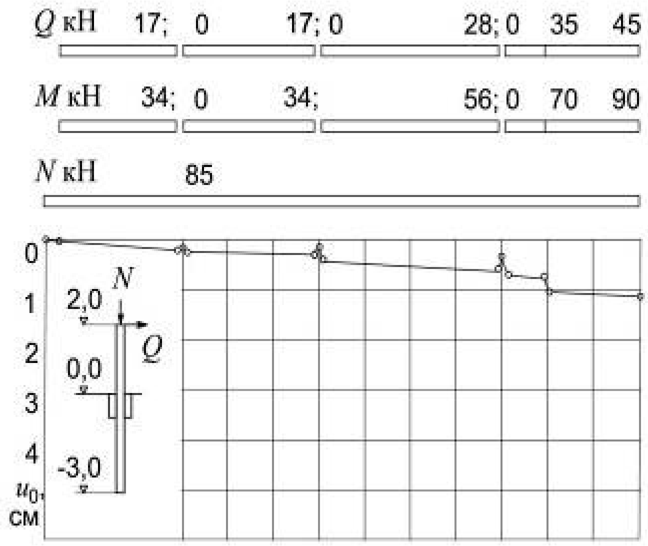

Figure 3 – The graph dependence of pile-column horizontal movements on the load in time

After removing of the horizontal load with a constantly acting vertical force, in the soil there is a decrease in deformation (precipitation and horizontal movement) by the amount of the elastic component of the deformation.

According to the results of the study, it was found that when a horizontal load is applied to the pile - column of the second stage, soil is hardened at a load of Q = 17 kN; M = 34 kN m the elastic component is equal to 58% of the total precipitation and 59% of the total value of horizontal displacements. Under the action of a repetitive force, the magnitude of which does not exceed its initial value, the residual deformations occur in the soil less than their initial value. The primers in this loading cycle are in a hardened state [9, 10]. The strengthening of repetitive loads leads to an increase in the resistance of the soil to external loads and refers to measures that improve its properties.

Conclusions:

1. The studies have shown under the loads action Q = 45 kN; M = 90 kN m; the elastic component is equal to 27% of the total precipitation and 29% of soil total horizontal displacement.
2. By means the tests it was fixed that zone lower boundary of residual deformations is located at a depth, where the stresses from the load transmitted by the foundation, below its sole, are balanced by its structural strength.
3. It has been established by the tests that, under the action of a horizontal load, the pile is rotated at the point of zero displacements. As a result, the soil in front of the loaded face of the column is compacted; the result of compaction is the pile movement, its rotation at the level of the day surface towards the action of forces, and the soles in the opposite direction. The rotation of a pile-column reduces the contact area of pile side surface with the ground. The reduced friction forces cause an additional pile draft.
4. The studies have revealed that after the removing of horizontal load under the action of a constant vertical force, there is adecrease in precipitation and horizontal movement by the amount of deformation elastic component.
5. By means of the tests it has been established that when applying repetitive loads to previously compacted soil, it is strengthened, i.e., soil resistance to external forces increases, which refers to measures improving its properties.

References

1. Gersevanov, N.M. & Polin, D. (1958). *Theoretical foundations of soil mechanics and their practical application*. Moscow: Stroyizdat.
2. Tsytovich, N.A. (1965). Questions of the theory and practice of construction on weak clay soils. *Proceedings of the All-Union Conference on Construction on Weak, Water-Saturated Soils*. Tallinn, 5-17.
3. Tugayenko, Y.F. (2008). *Soil deformation processes in foundations, piles and pile foundations. monograph*. Odesa: Astroprint.
4. Tugayenko, Y., Marchenko, M., Tkalich, A. & Mosicheva, I. (2015). Peculiarities of the soil deformation process at the bases of experimental settlement plates. *Technical journal Scientific professional journal of University North*, 9(1), 40-46.
5. Tugaenko, Y., Tkalich, A., Marchenko, M. & Loginova, L. (2015). Differential metod of estimation of soil resistance characteristic according to the pile test. *Scientific professional journal of University North*, 9(2), 180-185.
6. Safaqah, O. & Riemer, M. (2007). The Elastomer Gage for Local Strain Measurement in Monotonic and Cyclic Soil Testing. *Geotechnical Testing Journal*, 30(2), 164-172. doi:10.1520/GTJ100124.
7. Terzaghi, K. & Peck, R. (1958). *Soil Mechanics in Engineering Practice*. Moscow: Gostransizdat.
8. Geurze, E.C.W.A. (1964). *Rheological and Soil Mechanics*. IUTAM Simposium Grenoble-1964. Berlin: Springer-Verlag, 256-270.
9. Tjong-kie, T. (1957). Three-dimensional theory of consolidation and clay layers. *Scientia sínica,* 5(1), 203-215.
10. Tjong-kie T. (1966) *Determination of the Rheological Parameters and the Hardening Coefficients of Clays*. Rhéologie et Mécanique des Sols. International Union of Theoretical and Applied Mechanics. Retrieved from https://link.springer.com

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Experience of geosentetic materials use in drainage system device

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At present, objects of responsibility (consequences) construction various classes, including SS-3 occurs in areas with possible manifestations of dangerous engineering-geological processes. One of such processes types is flooding. Based on the world experience, the main possibilities of using geosynthetic materials in various fields of construction are considered. Requirements for such materials and the conditions for their use are set out in European norms. An analysis of the program that is used to calculate drainage systems to meet force requirements in Ukraine is performed. These materials have good prospects for building in Ukraine with appropriate justification, considering the normative documents in force in our country.

Keywords: drainage, filtration, geotextile, filtration coefficient, Darcy law

Досвід застосування геосинтетичних матеріалів при влаштуванні дренажних систем

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Відомо, що на сьогоднішній день будівництво об’єктів різних класів відповідальності (наслідків), у тому числі СС-3, відбувається на територіях, на яких можливі прояви небезпечних інженерно-геологічних процесів. Зазначено, що одним із видів таких процесів є підтоплення. При зведенні нових об’єктів на таких ділянках необхідно виконувати спеціальні заходи щодо захисту заглиблених частин будівель та споруд від дії ґрунтових вод та збереження існуючого гідрогеологічного режиму. При цьому слід виконувати вимоги діючих в Україні нормативних документів щодо інженерного захисту територій. Ґрунтуючись на світовому досвіді, розглянуто основні можливості застосування геосинтетичних матеріалів у різних галузях будівництва. Як приклад наведені сучасні матеріали, які можуть замінити традиційні рішення при проектуванні пластового дренажу. Зазначено, що геосинтетичні матеріали використовують у світі більше двадцяти років при будівництві цивільних та промислових доріг, спортивних майданчиків, полігонів твердих побутових відходів, парків, тунелів. Вимоги до таких матеріалів і умови їх використання викладено в Європейських нормах. Наведено характеристики матеріалів та їх складових частин, а також умови їх використання. Розглянуто переваги використання сучасних геосинтетичних матеріалів порівняно з традиційними рішеннями, пов’язаними з проблемами підтоплення територій, будівель і споруд. Виконано аналіз програми, яка використовується для розрахунку дренажних систем на відповідність вимогам, що діють в Україні. Показано, що ці матеріали мають гарну перспективу при будівництві в Україні при відповідному обґрунтуванні з урахуванням діючих у нашій країні нормативних документів.

Ключові слова: фільтрація, геотекстиль, коефіцієнт фільтрації, закон Дарсі

Introduction. At present, objects of responsibility (consequences) various classes construction, including CC-3, takes place at the sites where dangerous engineering and geological processes are possible. This is predetermined by the fact that the sites, relatively simple in terms of engineering-geological and hydrogeological conditions, have already been developed. The phenomenon of flooding is one of the dangerous geological processes types. This is due to the fact that considerable areas of Ukraine are located in the zone where the hydrogeological regime with high groundwater levels has already been formed, or the flood has an anthropogenic character associated with human activity. These conditions require the implementation of special measures that must protect the underground parts of erected constructions from groundwater, as well as compliance with the requirements of regulatory documents being in force in Ukraine required for engineering protection of territories.

Analysis of the latest sources of research and publications. Requirements for engineering protection of territories from flooding are set out in DBN A.2.1-1-2008 [1], DBN B.1.1-24: 2009 [2], DBN B.1.1-25-2009 [3]. However, the solutions proposed in these standards for the protection of buildings and constructions from flooding offer mainly traditional solutions, without consideration positive experience of using up-to-date materials, popular in the world. Abroad, geosynthetic materials as drainage have been widely used for more than twenty years. The conditions for such materials use are set out in European norms [4, 5, 6]. A large number of articles published in foreign scientific publications and reports made at international scientific conferences has been devoted to the application of drainage made of geosynthetic materials [7, 8].

In our country, these materials are not widely spread yet, although in the world practice, drainage systems made of geosynthetic materials are widely used in the performance of the following tasks:

– drainage of underground structures and parts of buildings located below the ground (vertical and horizontal);
– drainage of roads (including railway infrastructure);
– drainage of sports grounds;
– drainage of roofs and terraces;
– drainage of gardens and parks;
– drainage (gas) of landfills of solid household waste;
– waterproofing and drainage of tunnels.

Drainage systems made with the use of geosynthetic materials can be used both for diverting of ground water and for removing gases from landfills of solid household waste.

The following objects can serve as the examples of their use: the third tramway line in Nantes (France), the National Library in Beirut (Lebanon), sports complexes in Toulon (France), Quebec (Canada), construction of a microdistrict in Ore (France), the South Hospital in Marseille (France), the construction of solid waste landfills in France.

The main geosynthetic material for drainage, considered in this work, is Draintube produced by the company Afitex (France).

Distinguishing of previously unresolved parts of a common problem. When designing horizontal drainage (bed and combined) in civil engineering, usually a layer of crushed stone is used as a water-permeable layer through which water is diverted outside the protected area. Typically, layer thickness of crushed stone is at least 0.5 m when applying traditional solutions. The volume of crushed stone used as drainage is determined by the engineering-geological and hydrogeological conditions, as well as the need to lower the groundwater level to the required level. In such cases, the amount of work associated with the excavation of pit increases, the bottom of the excavation pit is additionally lowered, which may lead to complications of its work, or the performance of additional measures for the construction of water depression, the costs for the transportation of the worked out soil, and the purchase and delivery of crushed stone. The use of up-to-date drainage materials such as Draintube enables to minimize the volume of ground works, soil transportation (removal/delivery) and crushed stone, application of special measures related to water depression.

Task specification. The main task is to show the perspective of using up-to-date geosynthetic materials in the construction of buildings and constructions under flood conditions, as these materials allow solving the problems of groundwater drainage from the construction site and protecting the parts of buildings and constructions that are below the ground from the groundwater, as well as reducing the costs associated with ground works and soil transportation in the excavation of pits. The evaluation of the calculation method applied in the world in the design of drainage, as well as of the approaches to the solution of this problem applied in Ukraine, were carried out.

The basic material and the results. In the late 90s of the 20th century, a new type of flat drainage geocomposite Draintube was developed. This material consists of two components:

– two or three layers of non-woven geotextiles that act as a back filter, that is, they allow the passage of water through them, while preventing the entry of ground particles from the base. At the same time, on the side that is in contact with the structure, a waterproof layer in the form of a membrane, which is a waterproofing, can be made;
– built-in corrugated polypropylene perforated pipes, located at regular intervals. The distance between the pipes can vary from one pipe for every two meters of width to four per meter. The number of pipes depends on the operating conditions. Usually the following distances between them are acceptable: 0.25 m; 0.5 m, 1.0 m, 2.0 m. The following diameters of the pipes are acceptable: 0.16 mm, 0.20 mm, 0.25 mm. Perforated pipes in this material provide the basic function of drainage to ensure the transit of groundwater beyond the area under consideration. Figure 1 shows the gen-

eral view of the Draintube material. Figure 2 shows the main dimensions acceptable for drainage using the Draintube material.

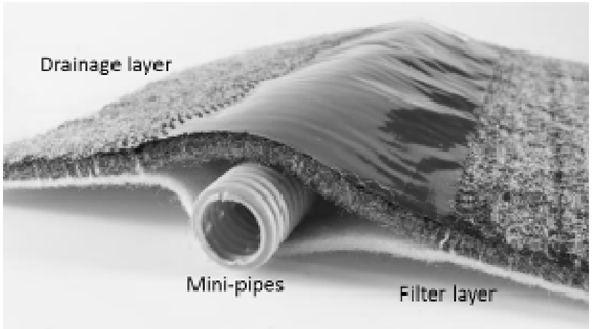

Figure 1 – General view of the Draintube material

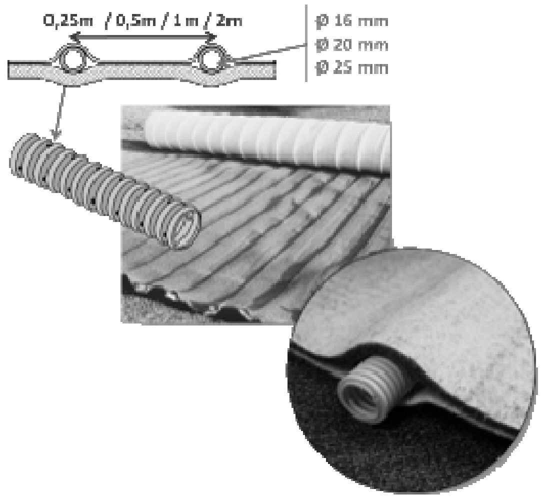

Figure 2 – Main dimensions acceptable for Draintube material

Depending on the task set, similar materials may be used, but considering the conditions where they are to be used. For example, if it is necessary to perform vertical drainage, the Alveodrain EV material can be used, which example is shown in Fig. 3. For this type of drainage, a special material made of thermoformed geotextile is used, through which water filters to the base of the vertical element and then enters to the polypropylene pipe, through which it is further drained outside the site.

To drain water out of drainage, the drainage pipes are connected to an interceptor through which water is drained outside the site. To connect the drainage pipes to the collector, a special connection (coupling) called Quick Connect is used, which enables to perform this work within a very short time. Figure 4 shows the Quick connect system for connecting drain mini-pipes to a collector.

Figure 5 shows examples of using up-to-date geosynthetic drainage materials at various sites.

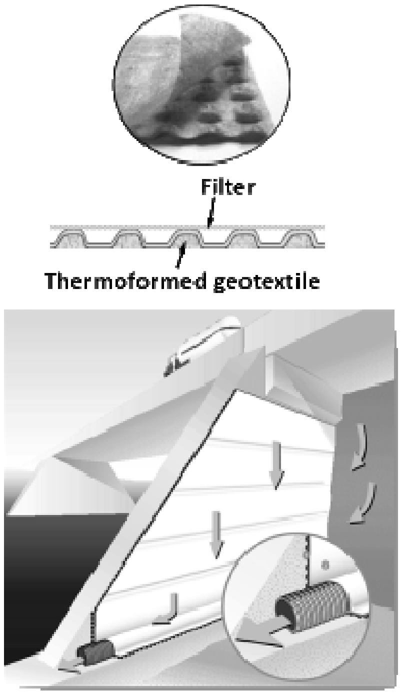

Figure 3 – An example of using Alveodrain EV material

Figure 4 – Quick connect system connecting drain mini-pipes to a collector

Figure 5 – Examples of using drainages made of geosynthetic materials

Compared with the traditional methods used in construction, the use of geosynthetic materials has several advantages. The main advantage is volume decrease of crushed stone and sand use as filtering materials. Depending on the conditions of drainages made of geosynthetic materials application, the economy of crushed stone reaches 60...70%, and under certain conditions it can reach 100%. At the same time, the pit depth is reduced (due to the reduction in the volume of used crushed stone), transportation cost of the worked-out soil and the delivery of crushed stone are reduced. In addition, the scope of works related to the construction of water depression is reduced. In certain cases, it is possible to use material where the sodium bentonite is used as the top layer (on contact with the construction), in such case this material can also serve as drainage and waterproofing.

On Figure 6 the options for using traditional solutions and drainage made of geosynthetic materials are compared.

To substantiate the possibility of using up-to-date geosynthetic materials as drainage and determine its parameters in the world, a number of specially developed software packages are used. One of such packages is the Lymphea software package, used by the company Afitex (France).

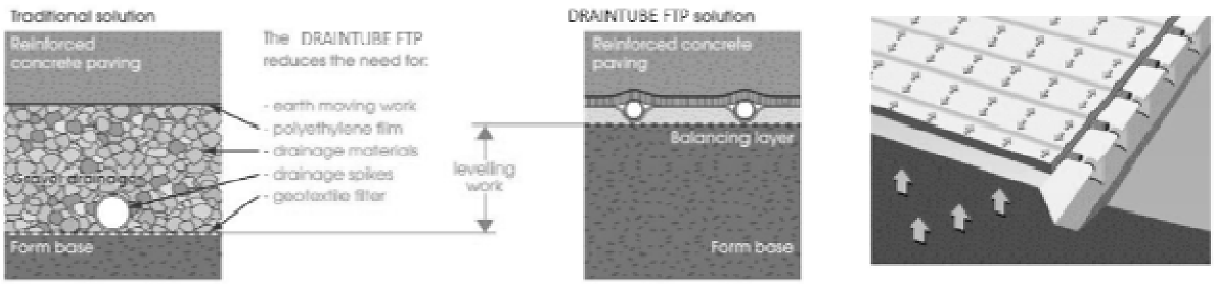

Figure 6 – Comparison of the traditional drainage design and drainage made of geosynthetic materials

The Lymphea software package is designed for performing hydraulic calculations to determine the parameters of drainage systems made of geosynthetic materials (geocomposites). This program is used in France. This complex is joint development of the Laboratory product of Regional Roads and Bridges from Nancy (France) and research laboratory in the field of geology and soils mechanics from Grenoble (France).

The software complex for solving the problems of determining the parameters of a drainage system made of geosynthetic materials is based on the application of the main law of filtration – Darcy's law.

The main parameters that are considered in this software package are the following:

– filtration coefficient of the drainage layer, taking into account the loads acting on it;
– length of drainage (dimensions);
– drainage slope;
– type of flow (complete flooding or pipe operation of partial cross section).

In the model that is formed for the calculation, the following prerequisites are used:

– the filtration flow is assumed constant (uniform), flowing from one side;
– two flow directions are taken: into the drain mat and into the mini-pipes.

The following parameters are considered in the calculation:
– pressure loss when water enters filter from the geotextile is acceptable;
– pressure loss when passing through the filter (geotextile) is acceptable;
– pressure loss when water enters the mini-pipe is acceptable;
– pressure loss when water flows through mini-pipes is acceptable.

During the calculation the pressure loss when passing from the ground to the filter is not considered. The drainage layer is water-saturated.

The specific consumption is assumed to be equal to

$$q = v_1 \cdot e_1 = k_{gf} \cdot i \,,$$

where q is the specific consumption, m^2/sec;
v_1 is the velocity of flow entering the drainage mat, m/sec;
k_{gf} is the geotextile filtration coefficient, m/sec;
i is the pressure gradient.

According to laboratory studies, which are consistent with the theory of filtration, pressure loss after water entering the mini-pipes is reduced. It is assumed that the mini-pipes are oriented in the direction of the pressure drop.

According to the results of laboratory studies, the expenditure that a mini-pipe can be calculated according to the formula

$$Q = q_d \cdot i = \alpha \cdot i^{n+1} \,,$$

where q_d is specific discharges of mini-pipes;
i is hydraulic gradient;
α, n are the values obtained experimentally.

Figure 7 shows the calculation scheme for determining the characteristics of hydraulic drainage made of Draintube material.

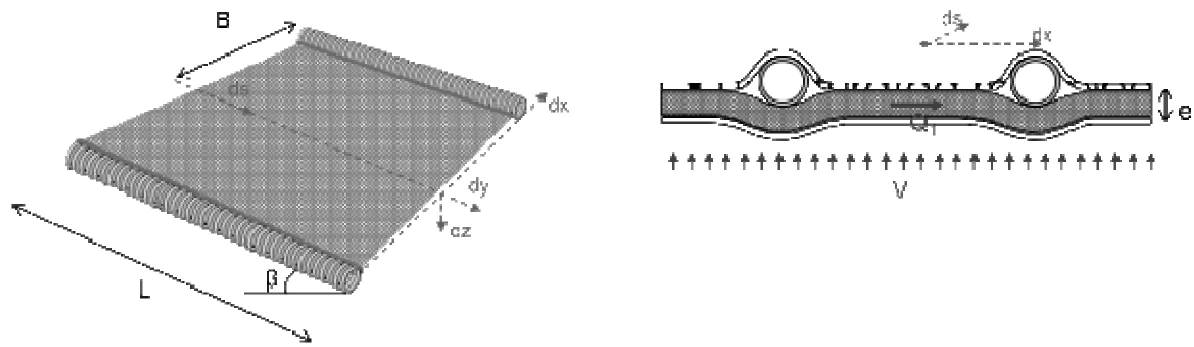

Figure 7 – Calculation scheme for drainage made of Draintube material

The maximum pressure in the drainage for the case when the mini-pipes work with the full cross-section is calculated according to the formulas:

$$h_{\max} = h_{geotextile} + \Delta_{hentransofm-p} + \Delta h_{m-p} \,;$$

$$h_{\max} = \frac{VB^2}{2k_{geitextile}} + C(VB)^b + \frac{n+1}{n+2} L^{\frac{n+2}{n+1}} \left(\frac{2VB}{\alpha}\right)^{\frac{1}{n+1}} + L \sin\beta \,.$$

For drainage, when the mini-pipes are operated with a partial section, the maximum pressure is calculated according to a formula

$$h_{\max} = \frac{VB^2}{2k_{geitextile}} + C(VB)^b \,,$$

where $h_{geotextile}$ is the loss of pressure;
$h_{hentransofm-p}$ is the loss of pressure at the inlet to the mini-pipe;
Δh_{m-p} is the loss of pressure on the length of mini-pipes;
V is the velocity of flow in the ground base;
B is half distance between the mini-pipes;
$k_{geotextile}$ is filtration coefficient of drainage layer (geotextile);
c, b, α, n are the values obtained experimentally.

For the case in which the mini-pipes are partially cross-sectional, the loss of pressure throughout their length is not considered.

Based on these equations, it is possible to determine the pressure in mini-pipes or the maximum drainage length. The solution of the equations is performed by selection, while different diameters of the pipes and the distances between them are taken into account in the calculations.

Figure 8 shows the results of comparing the experimental data obtained and the results of modelling (calculations). As a result of the comparison, we can conclude that the results of the calculations do not differ significantly from the experimental data. Figure 9 shows an example of visualization of calculation results.

The basis of the Lymphea software package contains the main provisions of the filtration theory, which are based on the Darcy’s law (the filtration law).

The theory of filtration studies the laws of motion of fluid, gas, or mixtures of these. The theory of filtration provides an opportunity to develop methods of filtration calculations of various kinds of structures during their design, construction and operation.

In the field of design, filtration calculations play an extremely important role, when solving issues related to flooding and filtration, it is possible to determine the structure and dimensions of constructions, as well as the construction of drains and various activities related to the protection of underground structures from flooding.

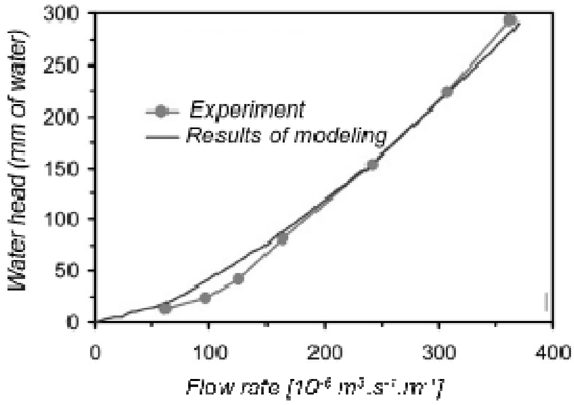

Figure 8 – Results of comparison of experimental data and modelling results

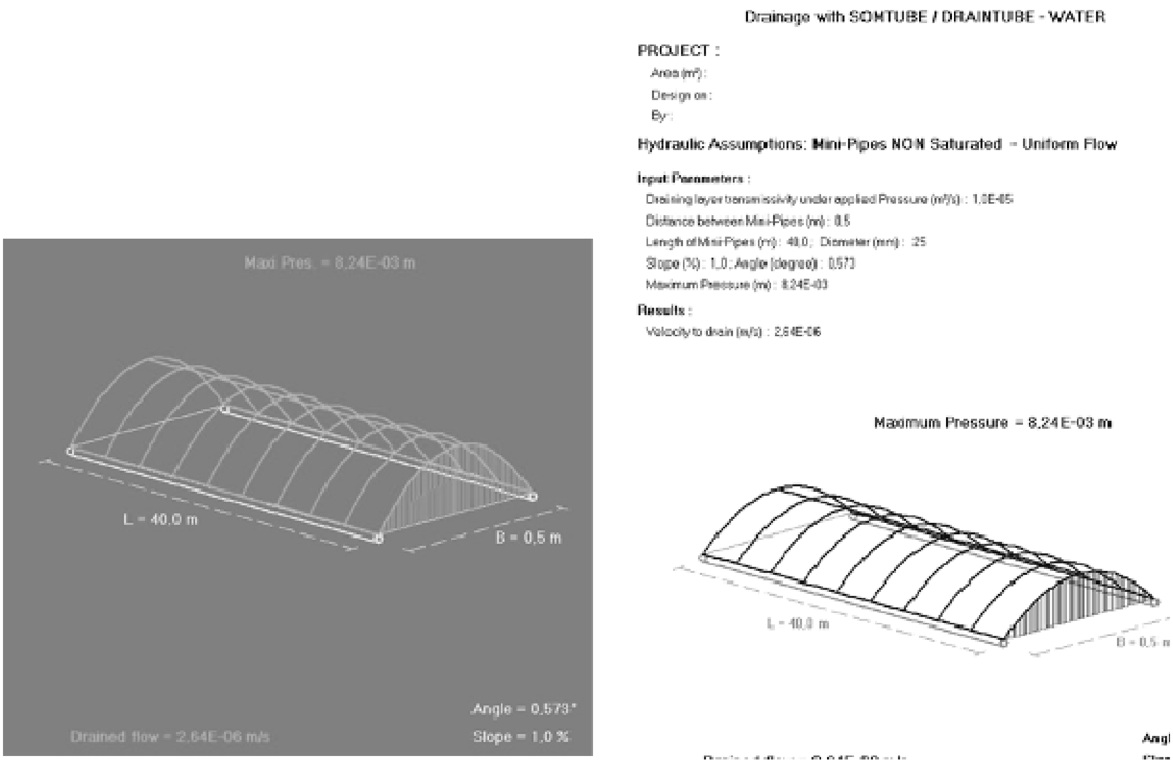

Figure 9 – Example of visualization of calculation results

The theory of filtration, which has wide practical application, has emerged relatively recently. The active development of the theory of filtration began in the second half of the twentieth century. The basis for the scientific development of filtration issues was the law of resistance in the fluid filtration.

In 1852–1855, the French engineer A. Darcy, having conducted a research on the study of filtration in sandy soils, established a linear relationship between the velocity of water filtration and the loss of pressure, which is called the law of filtration or the Darcy's law. According to the Darcy law, the pressure loss during filtration depends linearly on the filtration velocity. The presence of such a connection, as well as the development of hydraulics and hydromechanics, enables in the second half of the nineteenth century to develop theoretical dependency that can be used in filtration calculations.

The first theoretical studies of filtration, based on the linear law of filtration, were begun by J. Dupuis. However, the foundations of the filtration general theory were laid only in 1889 by M.E. Zhukovsky.

He derived differential equations of filtration. M.E. Zhukovsky introduced the concept of resistance force in filtration. In 1922, the theory of filtration was given a new boost owing to the work of M.M. Pavlovsky.

Further, the scientists of the world considerably expanded the theory of filtration in all directions and brought it to the level necessary for the design of modern constructions and their elements.

Conclusions. As a result of the approaches analysis used to determine the parameters of drainage made of modem geosynthetic materials (Draintube) manufactured by Afitex (France) in the Lymphea software package, it can be concluded that the theory of filtration applied throughout the world, including in Ukraine, are used in it. This software package enables to evaluate correctly the parameters of drainage, made of up-to-date materials, and to perform their design. The use of such drainage has become quite widespread throughout the world. There is a positive experience in their designing, construction and operation under different engineering and geological conditions on different continents. The requirements for such materials and the conditions for their application are described in the European norms.

These materials have good perspective of application in Ukraine with the appropriate justifying calculations, considering the normative documents in force in our country.

References

1. ДБН А.2.1-1-2008. (2008). *Інженерні вишукування для будівництва*. Київ: Мінрегіонбуд України.

2. ДБН В.1.1-24:2009. (2010). *Захист від небезпечних геологічних процесів. Основні положення проектування*. Київ: Мінрегіонбуд України.

3. ДБН В.1.1-25-2009. (2010). *Інженерний захист територій та споруд від підтоплення та затоплення*. Київ: Мінрегіонбуд України.

4. NF EN 9864 (2005). *Géosynthétiques – Méthode d'essai pour la détermination de la masse surfacique des géotextiles et produits apparentés*. Publiée, Comité technique: ISO/TC 221 Produits géosynthétiques.

5. NF EN ISO 12958 (2010). *Géotextiles et produits apparentés. Détermination de la capacité de débit dans leur plan.*

6. ISO TC221 WG6 N 181 TR 18228-4. (2017). *Design using geosynthetics. Part4: Drainage*.

7. Faure, Y.H., Matihard, Y., Brochier, P. & Suryolelono, K. (1993). Experimental and theoretical methodology to validate new geocomposite structures for drainage. *Geotextiles and Geomembranes,* 12(5), 397-412. doi:10.1016/0266-1144(93)90015-G

8. Sabiri, N., Caylet, A., Montillet, A., Le Coq, L. & Durkheim, Y. (2017). Performance of nonwoven geotextiles on soil drainage and filtration. *European Journal of Environmental and Civil Engineering*. doi:10.1080/19648189.2017.1415982.

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Flexible oneanchor retaining building models calculation results comparing with experimental data

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Based on the previously described mathematical model implemented in software complex PLASTICA, nonlinear calculation of one anchor sheet pile wall together with surrounding soil medium has been performed. To assess the results reliability with experimental data tests Lazebnik G.E., calculations in PLAXIS 2D and classic Coulomb method are compared. The basis of a mathematical model incorporated the theory of plastic flow with hardening, which is based on the principle of maximum Mises. In general, it can be assumed that the calculations results in the software package PLASTICA using the proposed nonlinear models showed satisfactory agreement as compared with the experimental data

Keywords:mathematical design, retaining wall, theory of plastic flow, experiment, mathematical pressure of soil, sensors, indicators of deformations, tensoresistors, epure of flexion moments

Порівняння результатів розрахунку моделей гнучкої одноанкерної підпірної споруди з експериментальними даними

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На підставі математичної моделі, яка реалізована в програмному комплексі PLASTICA, виконано нелінійний розрахунок шпунтової одноанкерної стінки спільно з оточуючим її ґрунтовим середовищем. Для оцінювання достовірності отриманих результатів їх порівняно з даними експериментальних випробувань Г.Є. Лазебника, розрахунками в PLAXIS 2D та з класичним методом Кулона. В основу математичної моделі закладено теорію пластичної течії зі зміцненням, яка базується на принципі максимуму Мізеса, що дозволяє врахувати процес складного навантаження і реальні властивості матеріалів конструкції та ґрунтів. Випробування здійснено в лабораторному лотку. Як засипку використано річковий кварцовий пісок. Тиск ґрунту на шпунтову підпірну стінку встановлено за допомогою датчиків для вимірювання малих тисків. Для визначення згинальних моментів застосовано індикатори деформацій і тензорезистори, встановлені на шпунтині-вимірнику. За результатами порівняння відзначено, що найбільші відхилення в епюрах пасивного тиску між експериментальними і розрахунковими в PLASTICA даними спостерігаються в нижній частині стінки, приблизно в два рази. Подібна різниця в епюрах активного тиску в області кріплення анкера склала 20%, в епюрах моментів – 23,5% в середній частині стінки. Показано, що результати розрахунку в програмному комплексі PLAXIS вийшли дещо завищеними порівняно з експериментальними даними. Доведено, що найбільші відмінності спостерігаються при порівнянні з результатами, отриманими за класичною теорією Кулона, особливо в епюрі пасивного тиску піску на стіну. У цілому, можна вважати, що результати розрахунку в програмному комплексі PLASTICA з використанням пропонованої нелінійної моделі показали задовільний збіг порівняно з експериментальними даними. Це дозволило зробити висновок про можливість використання запропонованої моделі в практиці проектування розглянутих споруд.

Ключові слова: математичне моделювання, підпірна стінка, теорія пластичної течії, експеримент, тиск ґрунту, датчики, індикатори деформацій, тензорезистори, епюра згинальних моментів

Introduction. Currently, there are a large number and variety of soil environment models, which continues to grow. Is is due to a variety of soil types and their work characteristics in a variety of conditions. Thus, any model describing phenomena characteristic subgrades should [1]: 1) to provide the laws and principles of solid mechanics deformable body; 2) reflect soil type characteristics, manifested by its behavior in various conditions; 3) determine the relationship among stress and strain.

For the design it is very important from the variety of existing models to make the most suiltable correct choice. It is known that the best proof of this selection correctness is to compare in each case the calculation results with natural or experimental data. This article is dedicated to this problem solution.

Analysis of recent research and publications of sources. Laboratory and field tests of retaining structures were made, for example, in [2 – 8]. The book [9] provides a detailed analysis and classification of the soil existing models, as well as references to work where they are described in detail. At [10] is described a mathematical model, which is embedded in the software complex PLASTICA used here, however in this work material exposure is lowered.

Isolation of previously unsolved aspects of the problem. In the article [11] a comparison calculation of the joint rigid retaining wall and its surrounding soil mass at software package PLASTICA have been already performed with experimental data obtained Z.V. Tsagareli by testing its towing model. In this paper, this comparison results example reliability for flexible oneanchor study wall is continued.

Purpose of the work. The aim of this study is to estimate the reliability of implemented in the software package PLASTICA mathematical modeling results of flexible retaining structures oneanchor by comparison with experimental data obtained by G.E. Lazebnik [5, 6] and a software package PLAXIS 2D and classic Coulomb method.

Main material and results. The basis of the mathematical model incorporated the theory of plastic flow with hardening, which is based on the principle of maximum Mises. It enables to consider the complex process of loading and the soils and construction materials real properties such as elasticity, plasticity and viscosity.

The algorithm incorporated in the software package PLASTICA uses numerical methods, which implement two problems: 1) sampling the area occupied by the model and the original equation; 2) the construction of an iterative process to determine the desired functions characterizing the stress-strain state model with preassigned accuracy.

The solution to the first problem enables to represent the problem in algebraic form, i.e. move from an infinite number of freedom model degrees to their finite number. It can be implemented using various projection methods, for example method of weighted residuals. The solution to the second problem enables to produce the original linearization of non-linear equations. In this case, the operation for adjusting their coefficients can be performed at each iteration solutions (method variables elastic parameters, the method of tangential inflexibility) or only on the first iteration or through their predetermined amount (modification of the method Newton -Kantarovich, which include, for example, the method of elastic and making initial stress method). Reviews of these methods, solutions have been found in many studies, for example, [12].

For discretization equations and the body region the finite element method (FEM) was used. Its advantages and drawbacks are detailed, for example, in [13]. One major advantage of this method is that it enables to use functions that approximate generalized solution, with domain within each finite element and enables to obtain the stiffness matrix symmetric and tape that makes the method very efficient. It is not superior to classical holds Bubnov-Galerkin and variational. FEM enables relatively easy to approximate the boundary conditions that cause serious difficulties in the finite-difference and variational-difference methods. Other benefits of FEM include its simplicity, versatility and clear physical interpretation.

The model of a flexible towing oneanchor retaining wall has been considered. Experiments were carried out on it in the tray size 5×2,5 m in plan and height 1,65 m using backfill river quartz sand of average particle size density ρ = 1,82 g/cm^3, and the internal friction angle φ = 36,5^o. Model sheet pile wall had a height of 1,6 m. It consisted of 10 individual sheet pile profiled section having a groove and a ridge. The model is made from an aluminum alloy durable D16 having a yield strength of 80 MPa and a moment of wall inertia of 54 cm^4. The tray was filled with sand up to the top wall.

Fig. 1 shows the towing pattern sheet pile after pile in subsoil and installs the anchor rods, a distribution of the beam.

To measure soil pressure on retaining sheet pile wall it was equipped with sensors for measuring low pressures. Their diagrammatic diametric section is shown in Fig. 2. Sensor at various stages of assembly is shown in Fig. 3.

Fig. 4 are schematic longitudinal sections of sheet piles models applied with established sensors. Also shown are the installation location indicators and strain gages on the sheet piling-meter bending moment. The mutual arrangement of piles models with sensors in the walls of the model is shown in Fig. 5.

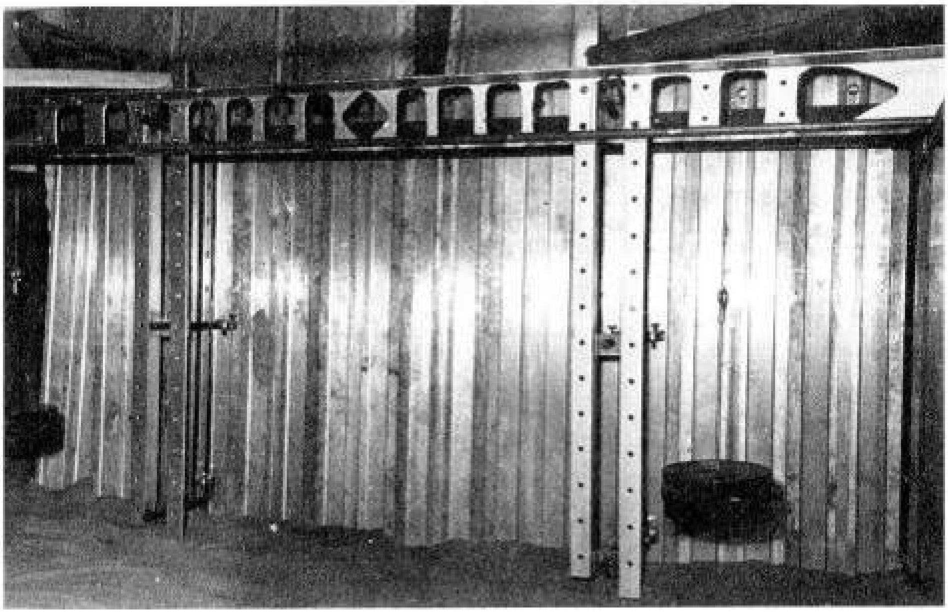

Figure 1 – Model of the sheet wall in the tray

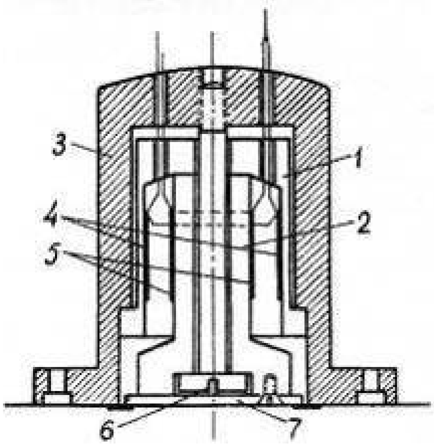

Figure 2 – A schematic diametrical sectional view
of the sensor for measuring small ground pressure:
1 and 2 – the outer and inner cups, respectively; 3 – a steel housing;
4 and 5 – strain gauges glued on the outer and inner cups; 6 – a tension screw;
7 – a steel lid that serves as a contact pad

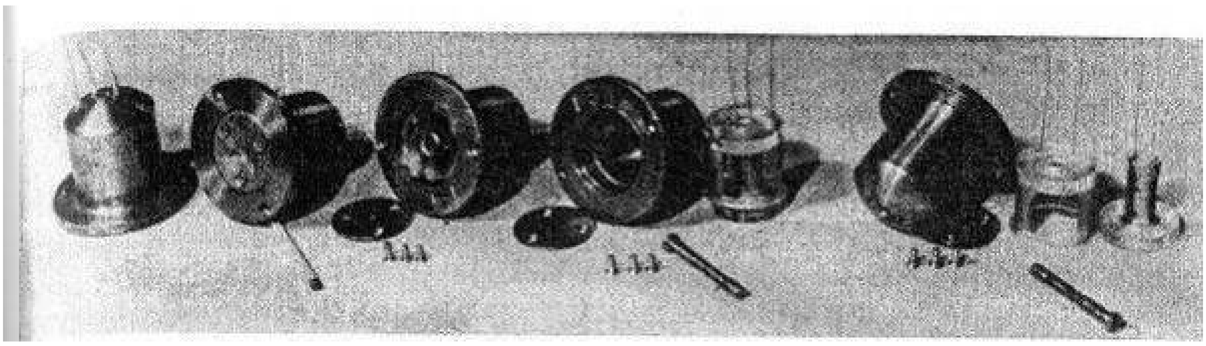

Figure 3 – Sensors for measuring low pressures at various stages of assembly

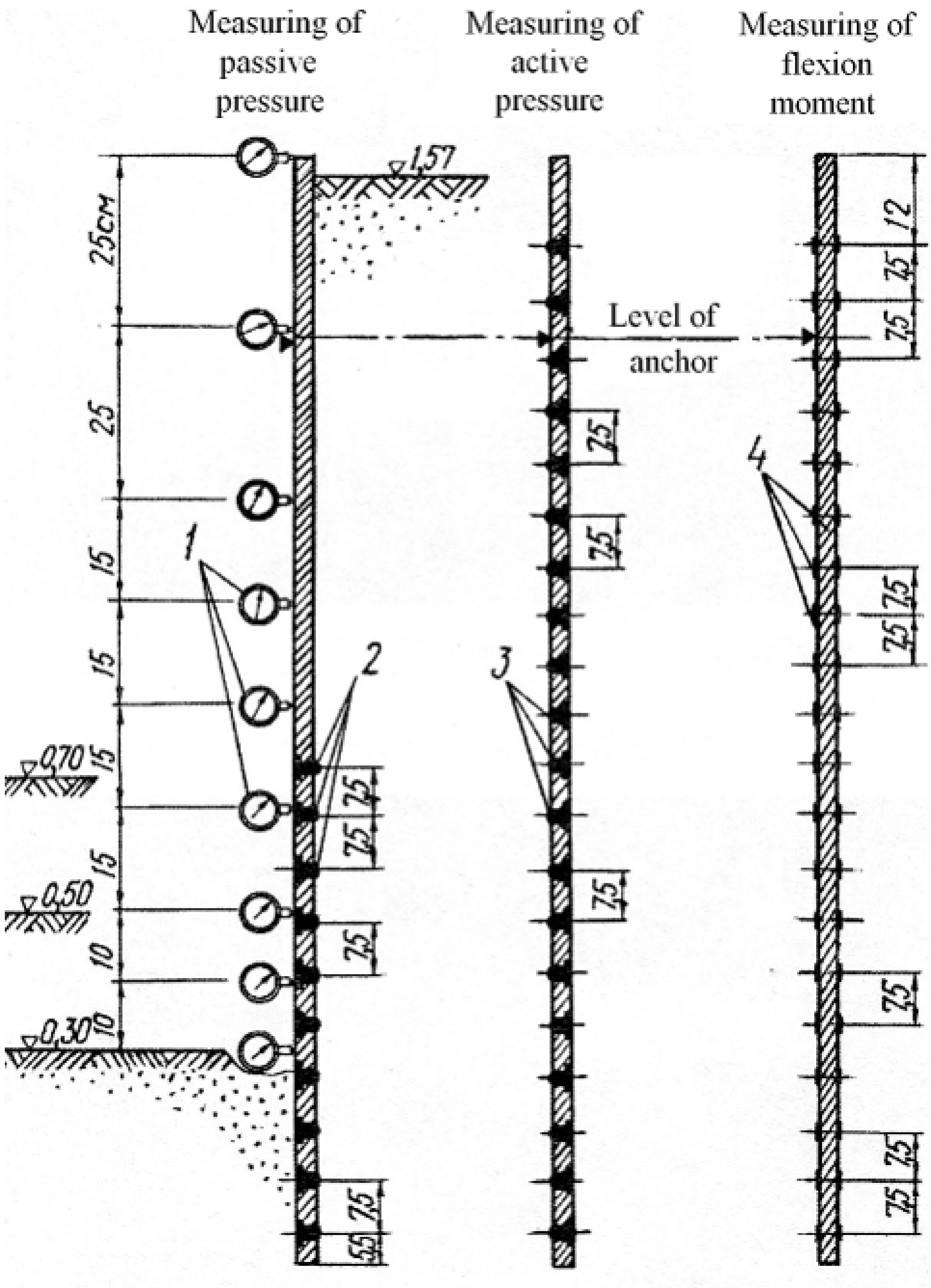

Figure 4 – Schematic longitudinal sections of sheet piles models with sensing devices
1 – strain indicators; 2 and 3 – active and passive sensors of pressure, respectively;
4 – strain gauges

Fig. 6, the solid line also shows the experimental data. Under the designation P shows diagrams of active and passive filling pressure on the wall and under the designation M bending moment arising in the wall. It can be seen that the greatest deflection in the Diagrams passive pressure between the experimental and calculated data in PLASTICA observed in the bottom wall and are 100% i.e. twice, and above – 9% (these places are shown in the figure by crosses). Such a difference in the Diagrams active pressure in the anchor mounting is 20%. At this moment diagram difference of 23,5% in the middle of the wall. Results of calculation in software PLAXIS complex turned somewhat exaggerated in comparison with experimental data. The greatest differences were observed when compared with the results obtained by the classical theory of Coulomb, especially in passive pressure sand diagram on the wall.

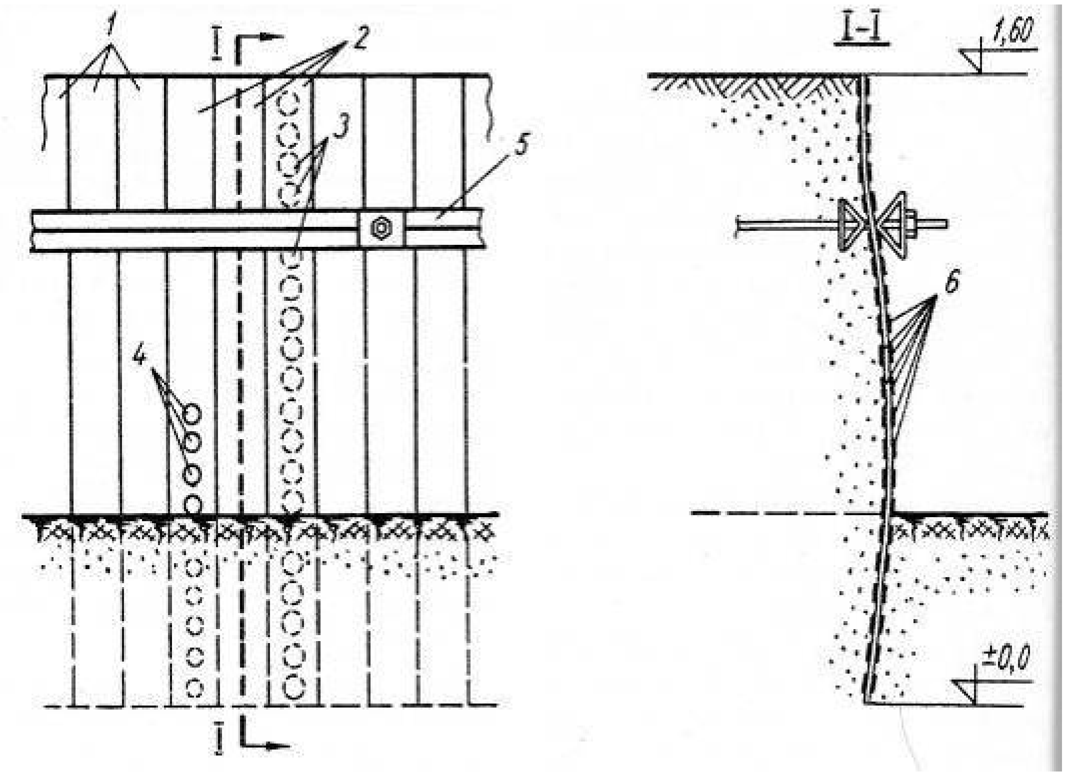

Figure 5 – The relative position of model piles, equipped with sensors
1 and 2 – respectively, are not equipped with sensors and equipped sheet piling;
3 – active pressure sensors; 4 – passive pressure sensors;
5 – distribution anchor beam; 6 – strain gauges

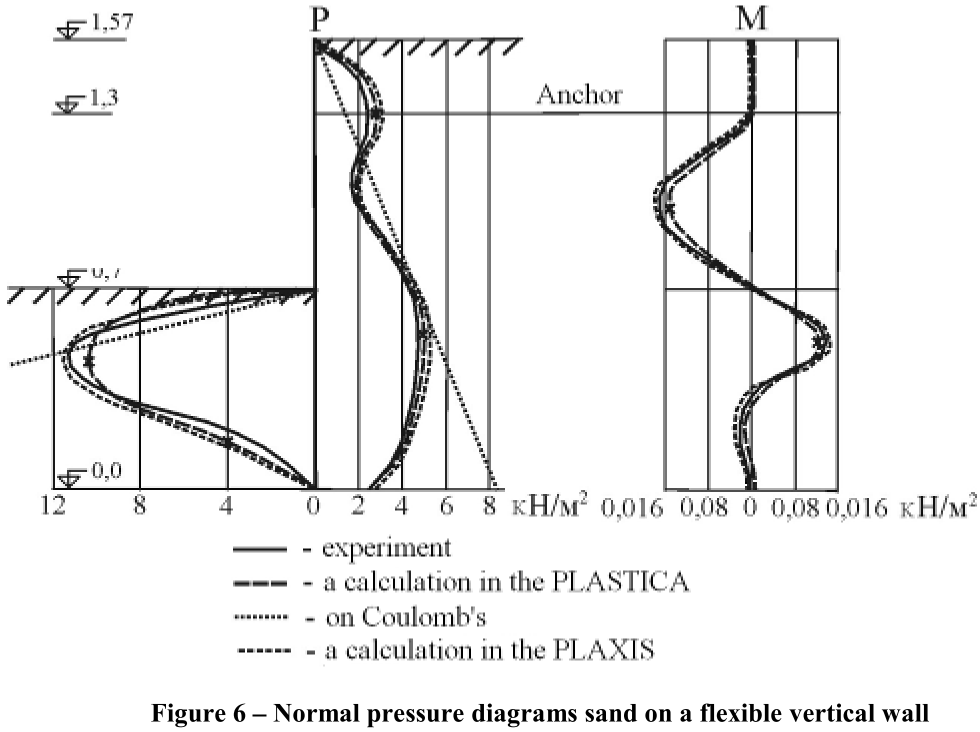

Figure 6 – Normal pressure diagrams sand on a flexible vertical wall and bending moment, resulting in the wall

Conclusions. According to the results of the comparison it can be noted that the calculations results in the software package PLASTICA using the proposed nonlinear models have shown satisfactory agreement as compared with the experimental data. It suggests the possibility of using this model in the practice of engineering structures under consideration.

References

1. Улицкий, В.М., Шашкин, А.Г. & Шашкин К.Г. (2010). *Геотехническое сопровождение развития городов*. Санкт-Петербург: Геореконструкция.
2. Дуброва, Г.А. (1963). *Взаимодействие грунта и сооружений*. Москва: Транспорт.
3. Ренгач, В.Н. (1970). *Шпунтовые стенки*. Ленинград: Изд-во лит-ры по строительству.
4. Цагарели, З.В. (1969). *Новые облегченные конструкции подпорных стен*. Москва: Стройиздат.
5. Лазебник, Г.Е. (2005). *Давление грунта на сооружения*. Киев: ППШВ.
6. Lazebnik, G.E. (1977). *Monitoring of Soil-Structure Interaction*. International Thomson Publishing.
7. Гуревич, Н.Б. (1969) *Речные портовые гидротехнические сооружения*. Москва: Транспорт.
8. Яковлев, П.И. (1964). Исследование работы разгружающих плит подпорных стенок. *Гидротехника,* 3, 69-85.
9. Винников, Ю.Л. (2016). *Математичне моделювання взаємодії фундаментів з ущільненими основами при їх зведенні та наступній роботі*. Полтава: ПолтНТУ.
10. Гришин, В.А., Руденко, С.В. & Гришин, А.В. (2017). *Математическое моделирование морских береговых оползневых склонов*. Херсон: Вид-во Гринь Д.С.
11. Гришин, А.В. & Сипливец, А.А. (2016). Сравнение результатов расчета модели подпорного сооружения с экспериментальными данными. *Моделювання та оптимізація будівельних композитів; матеріали міжнар. наук.-техн. семінару*. Одеса: ОДАБА, 22-24.
12. Zienkiewicz, O.C. (1986). *The finite element method*. Mcgraw-Hill Book Company (UK) Limited.
13. Owen D.R.J. & Hinton, E. (1980). *Finite elements in plasticity: theory and practice*. Pineridge Press Limited Swensea, U.K.

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Dynamic Calculation of the Pile Supported Wharf

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The article deals with the issues of designing and operating marine pile supported wharf in the seismically hazard areas considering various superstructures and overload equipment influence. Analysis of hydraulic structures seismic resistance erected in seismic regions of Ukraine shows that the actual seismic load on buildings significantly exceeds the estimated loads that are determined by the normative documents before 2006. Design of hydraulic structures should be done considering berths of reloading complexes, with proper scientific support. Berth construction calculationsof the ship repair yard No. 2 of the «Ilichevsk Ship Repair Plan» are given.

Keywords: hydrotechnical structures, seismic, ice and wave loads

Динамічні розрахунки причальних споруд естакадного типу

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Розглянуто питання проектування й експлуатації морських гідротехнічних споруд з урахуванням впливу різних надбудов і перевантажувального обладнання у сейсмічно небезпечних районах України. Сейсмічні впливи можуть задаватися як лінійно-спектральним методом, так і прямим динамічним методом, за розрахунковими акселерограмами землетрусу, які являють собою трикомпонентну функцію прискорення коливань у часі. Необхідність обліку спільної роботи комплексу «основа – споруда», обліку різних навантажень та впливів призводить до ускладнення розрахункового обґрунтування споруд. Аналіз сейсмостійкості гідротехнічних споруд, зведених у сейсмічних районах України, показав, що фактичні сейсмічні навантаження на споруди значно перевищують розрахункові навантаження, які були визначені нормативними документами до 2006 року. Наведено аналіз можливих видів руйнувань причальних споруд естакадного типу під час землетрусу, для аналізу використовувалися вітчизняні та іноземні літературні джерела. На основі аналізу визначено основні види руйнувань як окремих елементів, так і всієї конструкції, побудовано блок-схему та матрицю для визначення ймовірності безвідмовної роботи конструкції як для нормативних значень забезпечення надійності окремого елемента за окремими видами руйнувань, так і розрахункових. З'ясовано, що проектування гідротехнічних споруд повинно здійснюватися з урахуванням перевантажувальних комплексів за належного наукового супроводу. Наведенно можливі розрахункові схеми причалів естакадного типу, а саме: одномасову систему, двовимірні системи та загальний випадок просторової системи.

Ключові слова: гідротехнічні споруди, сейсміка, льодові та хвильові навантаження

Introduction. Ports development is one of the most important components of state economy development. In the era of globalization, the transformation of cargo flows, changes in their structure, port development planning issue, which implies port facilities balanced development, namely: sea zone development, port zone, and land zone, becomes especially important. In connection with the change in the normative intensity of seismic impact in DBN 1.1-12: 2014 in relation to SNiP II-7-81, for 4 marine ports in Ukraine, now the normative ball of the construction area is more by one ball on the MSK-64 scale, and 7 of the 13 ports are in areas with a glide above 7. It should also be noted that the territories, where ports are located, are characterized by complex engineering-geological conditions with a wide spread of highly porous loam and loamy sandy sediments related to the III-IV categories of seismic properties. According to the requirements of DBN B.1.1-12:2014 the calculated seismicity for ground conditions of this type should increase by one point relative to the normative one. Considering the fact that seismic impact can lead to destruction of port hydraulic structures that lead to disruption of port functioning, seismic impact assessment along with port upgrading is an important and urgent task.

Characteristic types of pile mooring structures destruction supported wharf with seismic actions *Port of San Fernando, Philippines*. On July 16, 1990, the Luzon earthquake in the Philippines with magnitude of M = 7.8 damaged the wharf № 1 in the port of San Fernando. Pier on reinforced concrete piles of a square section is 200 m long and 19 m wide. Vertical and inclined piles were used (Fig. 1).

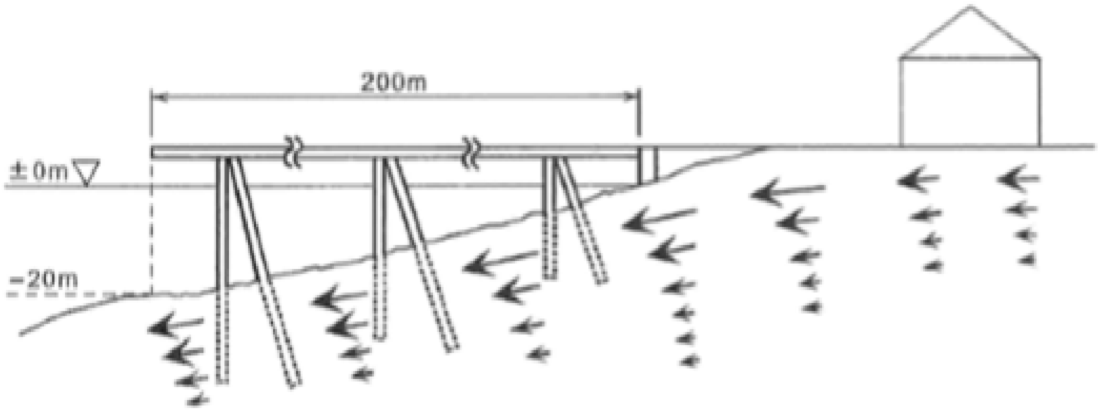

Figure 1 – Port of San Fernando, Philippines [10, 13]

The pier has received longitudinal displacements towards the sea [13, 14] as a result of shifts and bearing soil deformations. A lot of cracks arose in the longitudinal direction of the upper structure. The maximum opening of the cracks is 1.5 m. Also, cracks and fractures of pile heads were observed .

Port of Eilat, Israel. Earthquake on November 22, 1995 in Israel, the size of M = 7.2. The port of Eilat was at a distance of 100 km from the epicenter of this earthquake. The main berth had a total height of 13 m and a water depth of 10.5 m (Fig. 2). This open-type berth consists of prefabricated slabs and grillage on octagonal prestressed ferroconcrete piles with a diameter of 46 cm. After the earthquake, the pile was not damaged, but constant movements in the range of 5 to 15 mm led to jonts opening [5,10].

Port of Auckland, USA. The Loma Prieta earthquake in 1989, with a magnitude of 6.9 in California, caused serious damage to the terminal facilities in the port of Auckland [10, 13], which was 90 km north of earthquake epicenter. Acceleration in the Port of Auckland is 0,25 – 0,3g.

The most serious damage was inflicted on the Terminal in the vicinity of 7th Street (Fig. 3). The rarefaction of the backfill resulted in subsidence, coating lateral expansion and cracking over a large area. The nature of the damage was related to the fracture under tension as a result of embankment external pressure, indicating that liquefaction and accompanying transverse deformations were decisive factors.

Based on the analysis of trestle type moorage structures destruction during earthquakes, three main causes of trestle type moorage structures destruction during earthquakes can be identified [13]:

1. For berths built on strong ground it is the perception of inertia forces from the upper structure by piles (Fig. 3, a)
2. The maximum bending moment occurs in the headers of piles in the rear zone, since they have the smallest free length. In the event of significant shifts in the backfill, there may be movements of the upper structure towards the sea, which in turn lead to destruction, as shown in Fig. 3, b;
3. For berths built on weak soils, the appearance of piles movement towards the sea is characteristic, as shown in Fig. 3, c.

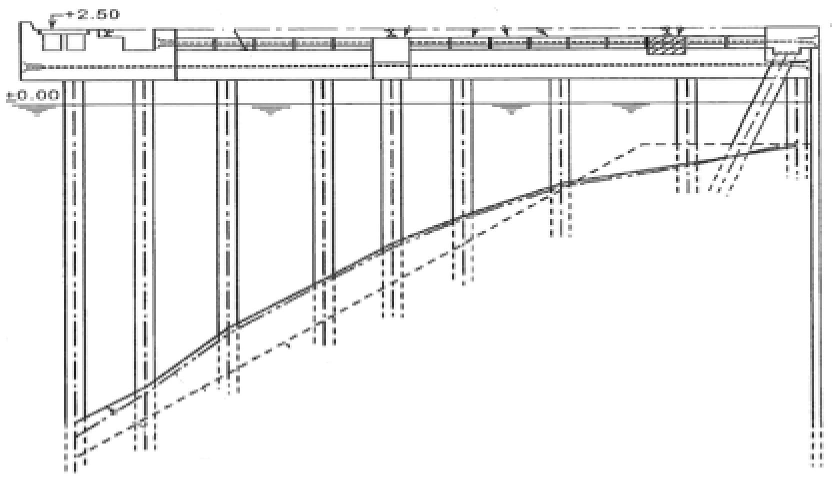

Figure 2 – Port of Eilat, Israel [10]

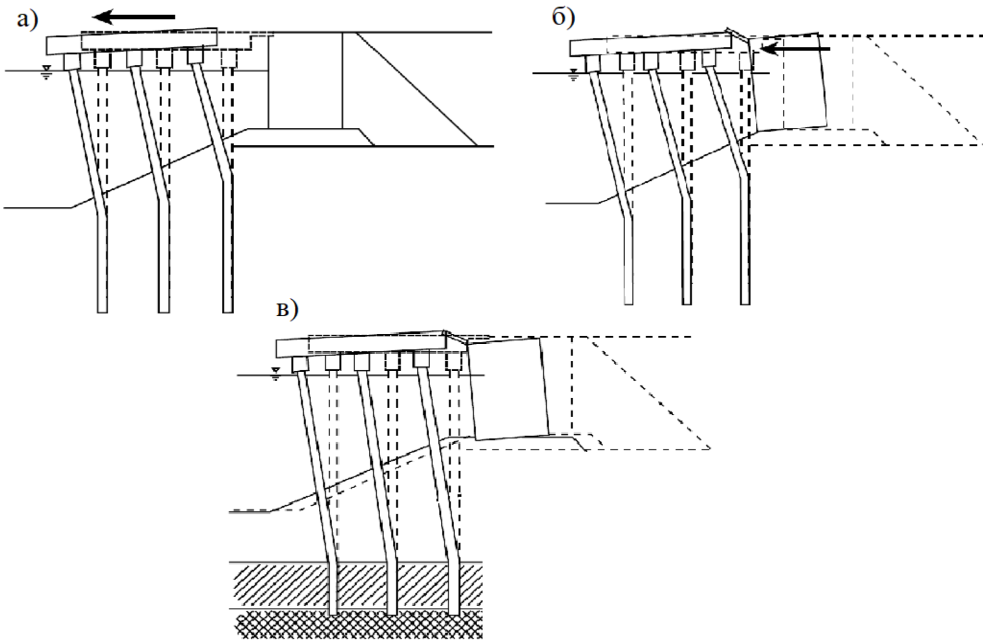

Figure 3 – Possible types of pile supported wharf during the earthquake destruction [13]:
a) deformation under the influence of inertia forces from the upper structure;
b) deformation in the event of significant shifts in the backfill;
c) deformation in weak soils

Design diagrams of pile supported wharf. Dynamic design schemes of pile supported wharf designed to determine seismic loads should be presented (Fig. 4, Fig. 5) [12, 16].

The most common and at the same time simple model is a single mass system (Fig. 4).

In modern design, a two-dimensional model is widely used to estimate the strength in the transverse direction-a multi-span frame (with rigid jamming and elastic springs modeling the base soil work (Figs 5, a, 5, b)) [13].

Recently, software complexes for the spatial calculation of complex structures have been developed. Figure 6 shows the calculated schemes of the pier:

– depending on the presence of links between sections, either as a chain of sections (see Figure 6 a), or as a separate section (see Fig. 6 b);

– depending on the presence of high-rise superstructures, or without add-ins (see Fig. 6a, b), or with superstructures (see Fig. 6 c, d) [13];

– depending on section upper structure deformation in the horizontal plane, or in the form of a hard disk (see Fig. 6, a, b, c), or in the form of a deformable structure (see Fig. 6, d), supported by elastic pile supports.

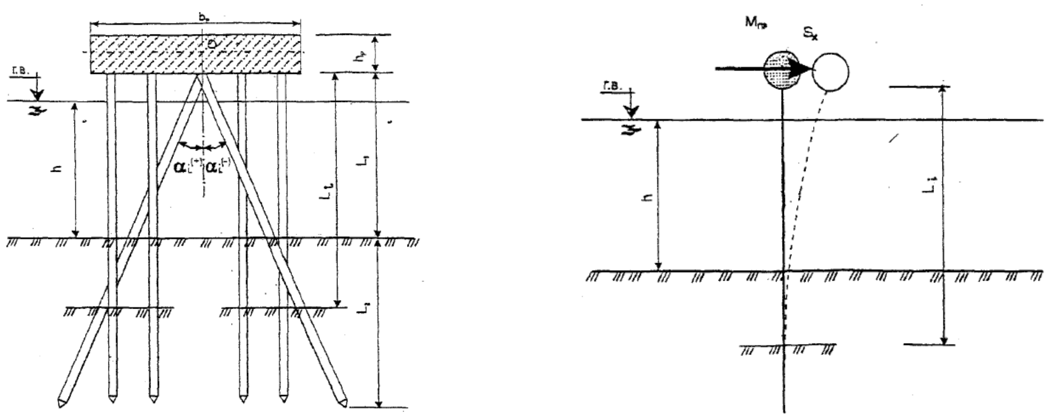

Figure 4 – Calculated (left) and reduced (on the right)

a) b)

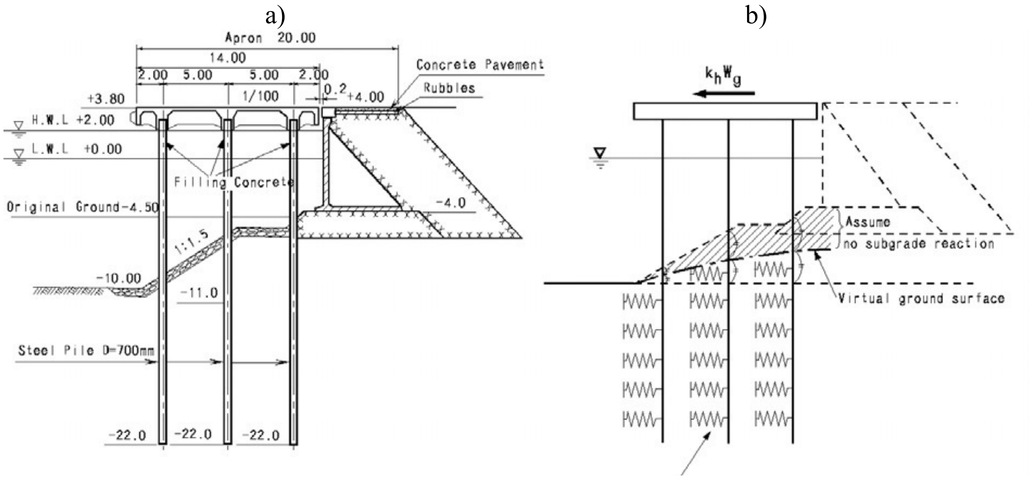

Figure 5 – Simulation of pile wharf:
a) rigid pinching; b) a system of elastic springs modeling the ground

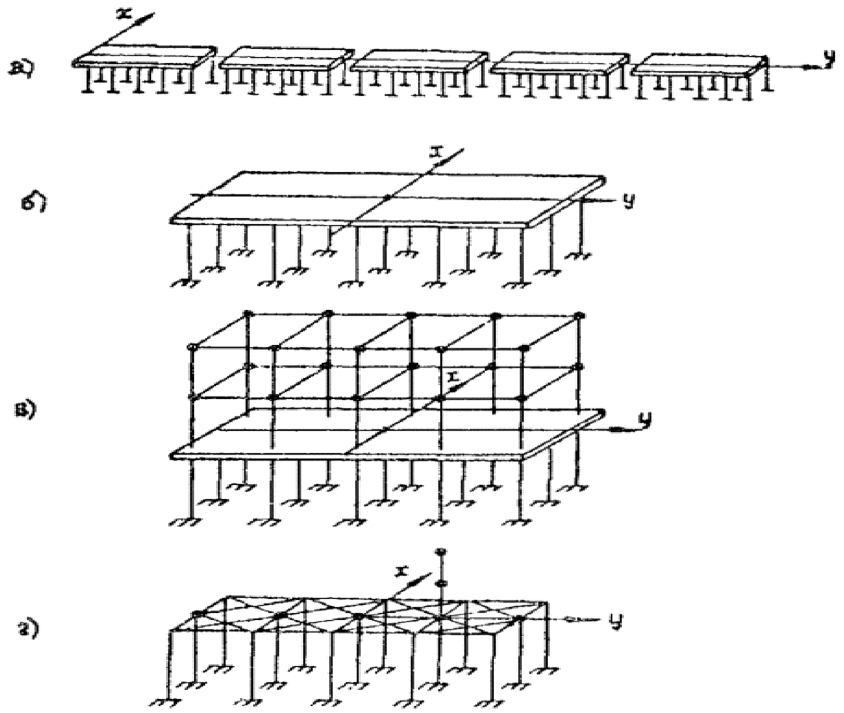

Figure 6 – Dynamic design schemes for pile supported wharf
a, b, c – the top structure in the form of hard disks; d – upper structure in the form of a deformable structure

The results of pile whatf calculation. Verification calculations were carried out for pier design of the ship repair pier No.2 « Ilyichevsky shipyard», considering design solutions for the reconstruction of the hydrotechnical part of the pier No.2 of the ISRZ for receiving Panamax type vessels for handling grain cargo. The calculation was carried out using the SCAD software, which implements the finite element method.

Loads and actions taken to calculate structures correspond to DBN B.1.2-2: 2006 «Loads and impacts» and SNiP 2.06.04-82 «Loads and impacts on hydraulic structures (wave, ice and ships).» [3]. Seismicity of the area for the proposed hydroengineering facility of responsibility class for the failure in operation of CC2-2 - 7 (seven) points.

It is supposed to perform a partial reconstruction of the berth, namely: to keep the depth on one side of the pier while installing the dredging on the other side, with the device of a subprime slope with a laying close to 1: 4, with matting of the BONTEX type. The settlement vessel of the type CH-50 (Panamax), the depth at the berth is 12.5 m from the «O» port. For the perception of mooring forces with reinforced concrete existing piles altered due to dredging, on feast two sides, the pallet openings, which are monolithic reinforced concrete superstructures, 1550 mm thick, joining 4 newly arranged piles of steel rolling profiles «Stainless Steel Pipes» along GOST 10704-91, with a diameter of 1020 mm, with a wall thickness of 14 mm. With the berth section standart length of 46.5 m, there are four slots for one section (2 on each side symmetrically). The pallets are joined in pairs by a monolithic reinforced concrete superstructure due to the promontory of the existing superstructure, including rubbing, 1550 mm thick. In the middle part of the joining plate, a pair of inclined piles are arranged, symmetrical to the longitudinal axis of the section, from pipes of the same diameter, with a 3: 1 laying. The mark of the piles bottom lowering corresponds to the roof of the IGE «Sand clayey» (abs-30.5), which corresponds to the length of the vertical element of 31.55 m and 33.26 m – the sloping one.

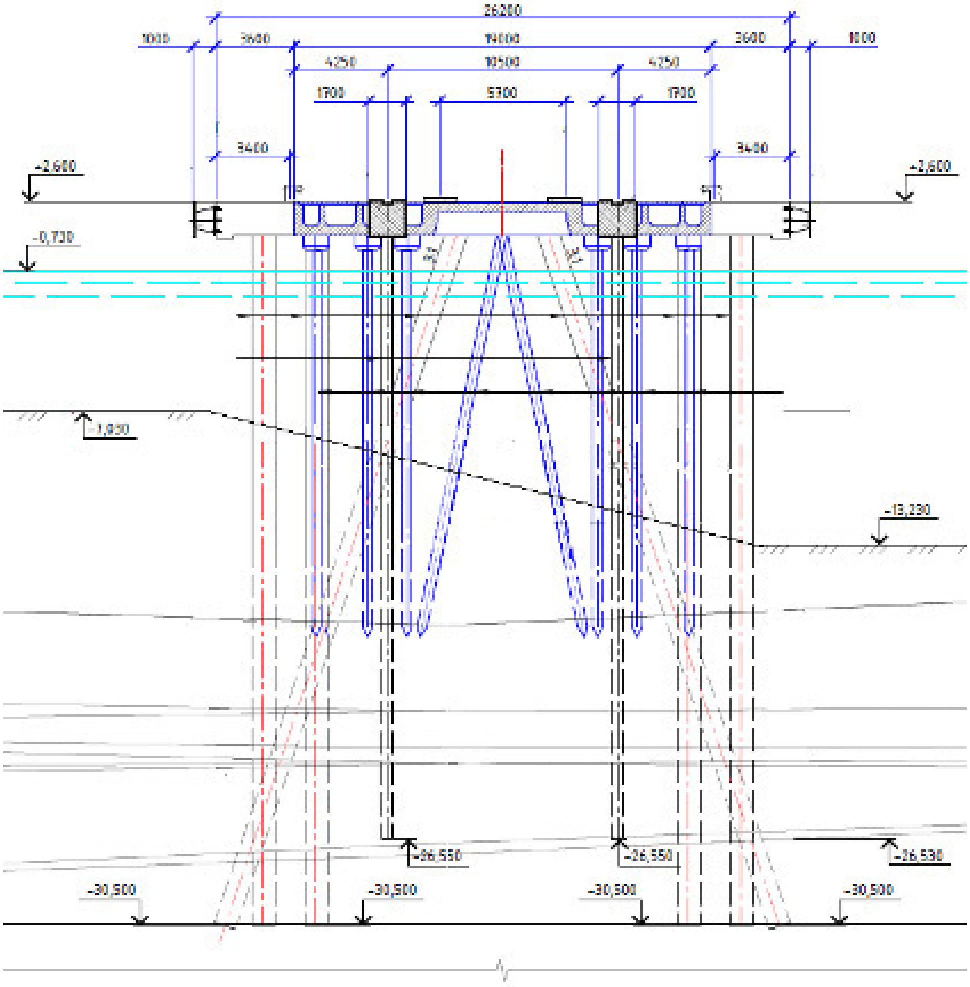

Figure 7 – Cross section

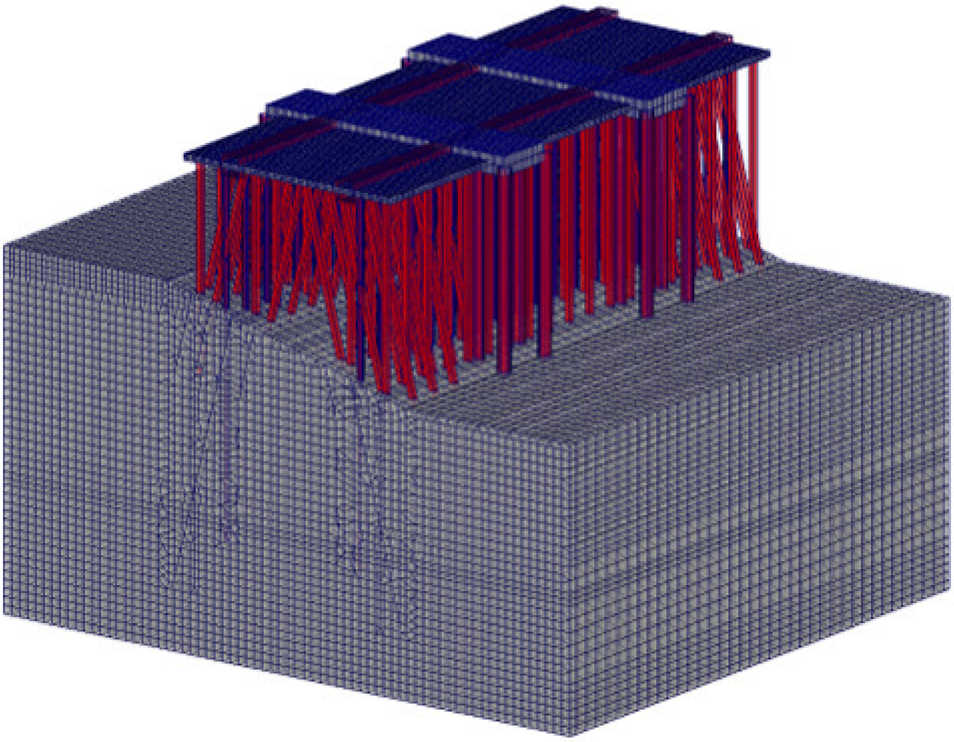

Figure 8 – three-dimensional design frame. General form

Re-profiling the berth for the processing of grain cargo provides for the installation of a loading machine with a capacity of 1200 t/h of the type «Neuero». The base is 10500 mm, the number of rollers is -4 (5), the load on the skating rink is up to 37 tons.

To absorb loads from the reloading machine, additional series of piles are arranged along the axis of crane beams from the steel rolling profile «Pipes made of straight steel» according to GOST 10704-91, with a diameter of 530 mm, with a wall thickness of 12 mm. The pile step corresponds to the 2.5 m moorage accepted for the original design.

The accepted cross-section of a monolithic reinforced concrete crane girder is 170x155 cm.

For cargo delivery to the reloading machine on the quay, a conveyer gallery assembly is provided. The gallery is made of metal structures. Supports pitch is vertical, in the longitudinal direction – 6 m, in the transverse direction - 5.7 m. The calculation scheme is shown in Fig. 8.

Considering feasibility study nature, the following loads and their combinations were used to assess structure performance:

– Load from the own weight of the structural elements (L1);
– Load from the ship loader (L2..L6);
– Load from transport conveyor gallery (L7);
– Load from mooring tension (L8);
– Payload on the berth territory (L9);
– Seismic impact intensity of 7 (seven) balls in the direction «perpendicular» to the longitudinal axis of the pier.

The results of the calculation are presented in Table 1.

Table 1 – Periods of oscillation

Load	Form	Eigenvalues	Frequency		Periods (seconds)
			1/sec	Gz	
10	1	0,308	3,252	0,518	1,931
10	2	0,221	4,517	0,719	1,39
10	3	0,205	4,881	0,777	1,287
10	4	0,068	14,78	2,354	0,425
10	5	0,066	15,229	2,425	0,412
10	6	0,055	18,15	2,89	0,346

Proceeding from the experience of designing such structures and in accordance with [9], the period of port piers and pile embankments natural oscillations (including the inclined pile) is in the range T = 0.19 – 0.44. This leads to the need to take the coefficient of dynamism on the ground β = 2.5 that determines the maximum horizontal seismic force on the port overpass.

The period of pier natural oscillation in accordance with the manual calculation is T = 0.4481 sec, and in the three-dimensional model T = 1.931 sec, which leads to an underestimation of seismic forces.

The calculations showed that the berth was designed for loading from the windfall at a wind speed of 20 ms, a design vessel of the Panamamax type.

The horizontal force from the windmill is equivalent to the seismic force at 7 balls.

Analysis of research results. The significant change in the technical regulatory framework during the period of Ukraine's existence as an independent state led to the situation when most of the port infrastructure facilities commissioned before 1991 and currently operated nowadays do not meet the requirements of these standards. For example - with the implementation of DBN B.1-1-12: 2006 and revision B.1-1-12: 2014, formally, none of Odessa and Ilyichevsk ports hydraulic structures, built before 2006, do not provide the required responsibility class, since their calculation and design were carried out, including without consideration possible seismic event onset.

In the process of designing marine hydraulic structures, many natural factors should be considred including hydrological, hydrographic, engineering-geological, geomorphological, and meteorological conditions of the construction area. Hydrological conditions include: sea wind wave, ice regime, level fluctuations, sea currents, tsunami waves. The hydrographic conditions include water depth, seabed and adjacent coastline topography. Of particular importance are the engineering-geological and geomorphological data on seabed structure, bottom soils physical and mechanical properties, and sediments migration. The main meteorological factor is the wind regime (speed, direction and duration). Also, during the design of offshore structures, seismic calculations must be performed. In this case, it is necessary to consider structure structural features and construction area existing engineering and geological conditions [3, 4].

Calculations of hydraulic structures strength, landslide slopes stability, structures located on them and bank protection structures in Odessa region should be carried out considering seismic loads. These effects can be specified either by linear-spectral method or by direct dynamic method, according to the calculated accelerograms of earthquake which represent oscillation acceleration three-component function in time.

The need to consider joint work of the «foundation-construction» complex, taking into consideration the various loads and impacts, leads to design basis for the structures complication. Calculation of emergency (seismic) impact using calculated accelerograms, considering nonlinear physical properties, considering the stages of construction, complicating the task of calculating and designing marine hydraulic structures many times, and at the same time making it possible to obtain rational solutions that provide specified operational properties with regulated reliability and safety parameters.

Conclusions. The issues of port hydraulic structures calculations, design, maintenance, repair, inspection and reconstruction have traditionally been regulated by the departmental requirements (Ministry of the Maritime Fleet) regulatory documents. There are no updated versions of these documents that correlate with state building regulatory documents.

Regardless of the current strategy for the port industry development, a systemic state approach is required to create modern technical regulatory framework and effective methods for monitoring compliance, especially for strategic port infrastructure facilities.

When designing hydraulic structures, it is necessary to considere a number of factors and requirements; compliance with them ensures structure effective operation, reliability and durability.

Based on the analysis of surveys results and existing regulatory documents requirements, it is necessary to designate the design parameters of natural, including seismic, impacts on the designed structures, considering their service life.

Design of hydraulic structures should be carried out considering transshipment complexes at the berth, with proper scientific support.

The use of new constructive solutions requires appropriate experimental studies, including in-situ and laboratory conditions.

During project implementation, it is necessary to comply with the regulatory documents requirements that ensure construction and installation works proper quality.

At present, it is necessary to improve the methods for calculating the stress-strain state of the soil foundation under hydraulic engineering structures, considering alternating effects.

References

1. ДБН В.2.4-3:2010. (2010). *Гідротехнічні споруди. Основні положення*. Київ: Мінрегіонбуд України.
2. ДБН В.1.1-12: 2014. (2014). *Строительство в сейсмических районах Украины*. Київ: Министерство строительства, архитектуры и жилищно-коммунального хозяйства Украины.
3. СНиП II-7-81*. (1985). *Строительство в сейсмических районах*. Москва: Минстрой РФ.
4. Немчинов, Ю.И. (2008). *Сейсмостойкость зданий и сооружений*. Киев: Будівельник.
5. Немчинов, Ю.И., Марьенков, Н.Г., Хавкин, А.К. & Бабик, К.Н. (2012). *Проектирование зданий с заданным уровнем обеспечения сейсмостойкости*. Київ: Будівельник.

6. *Пособие по определению несущей способности эксплуатируемых в сейсмических районах морских гидротехнических сооружений*. (2005). Москва.

7. Патынский, В. (2012). Состояние портовых гидротехнических сооружений Украины. *Порты Украины,* 5(117). Взято з: https://ports.com.ua.

8. Пустовитенко, Б.Г., Кульчицкий, В.Е. & Пустовитенко, А.А. (2006). Новые карты сейсмического районирования территории Украины. Особенности модели сейсмической опасности. *Геофизический журнал,* 28(3), 54-77.

9. Nozu, A., Ichii K. & Sugano, T. (2004). Seismic Design of Port Structures. *Journal of Japan Association for Earthquake Engineering*, 4(3). doi.org/10.5610/jaee.4.3_195.

10. Borg, R.C. (2007). *Seismic performance, analysis and design of wharf structures: a comparison of worldwide typologies* / A Dissertation Submitted in Partial Fulfilment of the Requirements for the Master Degree in Earthquake Engineering.

11. *Seismic Design Guidelines for Port Structures.* (2001). International Navigation Association. Tokyo: Balkema Publishers.

12. Egan J., Hayden R., Scheibel O. & Seventi G. (1992). Seismic repair at Seventh Street Marine Terminal. *Grouting, Soil Imporovement and Geosynthetics, Geotechnical Special Publication*, 30, 867-878.

13. Chen Wai-Fah, Scawthorn Ch. (2003). *Earthquake engineering handbook.* CRC Press LLC.

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Marine transportation-technological systems safety and development

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The article deals with marine hydraulic structures design and operation, considering the influence of various superstructures and reloading equipment. The analysis of seismic resistance of the hydraulic structures erected in the seismic regions of Ukraine has shown that the actual seismic loads on the structures significantly exceed the design loads being determined by regulatory documents prior to 2006. In the era of globalization, transformation of cargo flows, changes in their structure, the issue of ports planning development, implying port capacities balanced development, namely: sea zone port zone, and land zone development, becomes particularly important. The design of hydrotechnical structures should be carried considering the transshipment complexes at the quay with proper scientific support.

Keywords: hydrotechnical structures, reloading equipment, seismic, ice and wave loads

Безпека і розвиток морських транспортно-технологічних систем

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У статті розглянуті питання проектування і експлуатації морських гідротехнічних споруд з урахуванням впливу різних надбудов і перевантажувального обладнання. Аналіз сейсмостійкості гідротехнічних споруд зведених в сейсмічних районах України показав, що фактичні сейсмічні навантаження на споруди значно перевищують розрахункові навантаження, які були визначені нормативними документами до 2006року. Проектування гідротехнічних споруд повинно здійснюватися з урахуванням знаходяться на причалі перевантажувальних комплексів, при належному науковому супроводі. Світовий досвід дозволяє визначити розвиток портів як одну з найважливіших складових розвитку економіки держави. В епоху глобалізації, трансформації вантажних потоків, зміни їх структури особливо важливим стає питання про планування розвитку портів, що припускає збалансований розвиток портових потужностей, а саме: розвиток морської зони, портової зони, сухопутної зони. Так, за статистичними даними, загальна тенденція зростання перевалки вантажів у всіх портах Чорноморського регіону до 2016 року, для портів України в 2016 році показала різке скорочення перевалки транзитних вантажів з Росії і Білорусі - нафти і нафтопродуктів, вугілля, руд, металів при нарощуванні переробки імпортно -експортних вантажів - контейнерів і зерна. Інфраструктура морських портів являє собою складний симбіоз будинків, будівель, споруд, механізмів, конструкцій, розташованих на території і (або) акваторії морського порту і забезпечують роботу транспортної інфраструктури країни в цілому. Сам по собі морський порт залежний елемент транспортної інфраструктури. Вирішення питання стратегічного планування комплексного розвитку складної інфраструктури можливо тільки на підставі аналізу та прогнозу довгострокового розвитку вантажної бази. В українських портах працює більше 500 портальних кранів, середній вік яких становить понад 37 років, деякі портальні крани працюють по 50 років. Зазначений вік перевищує європейські стандарти практично в два рази. Виробничі потужності портів представлені в основному парком портальних кранів імпортованих за програмою Мінморфлот СРСР, балансова вартість яких через багатократні індексації значно перевищує ринкову

Ключові слова: гідротехнічні споруди, перевантажувальне устаткування, сейсмічні, льодові та хвильові навантаження

Introduction. World experience enables to define ports development as one of the most important constituents of governmental economic development. In the era of globalization, transformation of cargo flows, their structural changes, the issue of ports development planning becomes especially important that intends balanced development of port capacities, namely marine zone, port area and ground zone development. Thus, by static data, general growth trend of goods transhipment in all the ports of Black Sea region till 2016 year, showed drastic transhipment reduction for Ukrainian ports in 2016 for goods from Russia and Belarus, such as mineral oil and petrochemicals, coal, metals, when increasing processing of import-export cargo as in containers and grain.

Materials and methods. For Ukrainian marine industry, there are two mutually exclusive approaches to issue solution about its future development. The official position is ports privatization and concession, which is apparently "an only possible way of Ukrainian marine industry development". According to this position, marine industry reform is one of current government activity priorities and among first steps there is a formation of special marine administration and administration reforming of Ukrainian ports.

According to representatives of sea trading ports enterprises, in case of privatization, private investments would be used in building port facilities which are currently involved by no more than 65%. In this case, the port infrastructure owned by the government would decline. It means that it is possible to develop strategical infrastructure objects only by the government sector income of the port industry economy that generates up to 8 billion UAH per year. Although, the government should reject the privatization and increase the share of capital investments in the infrastructure of ports, which in 2015 amounted to no more than 5% of their income.

The maritime ports infrastructure represents a complex symbiosis of buildings, structures, edifices, mechanisms, constructions, located on the territory and (or) the water area of the maritime port and provides the work of the government transport infrastructure system in general. The maritime port itself is a depended element of the transport infrastructure. The solution for the strategical planning issue for complex development of a complicated infrastructure is possible only if based on analysis and forecast of long-term cargo base development.

Figure 1 – Combination of buildings, structures, edifices, mechanisms constructions located on the territory of maritime port

In accordance with the law of «maritime ports of Ukraine» from 17.05.2012, reloading equipment refers to the objects of ports infrastructure. In order of «The development strategy of maritime Ukrainian ports until 2015» approved by the Ministers Cabinet of Ukraine on July 16, 2008. №1051 (Expired after signing the order of July 11, 2013 №548-р) the port infrastructure was defined as a specialized property complex, as well as storage facilities and grounds, loading and unloading mechanisms and other property. According to the strategy of 2008, the attraction of investments in the development maritime ports were planned through public-private partnership (PPP). However, during the eight years work of different ministries and administrations the procedure for the PPP has remained at the same level. For the specified period of time it was possible to modernize the infrastructure of the main Ukrainian ports and become competitive subjects of the international transport system.

After 2013-year reform assets of every seaport were divided into two groups:

– Strategic objects (water area, docks, other hydrotechnical constructions, communications) are assigned to administration of sea ports (ASP) with main office in Kiev and branches in every port;

– Nonstrategic objects (including storage areas and other main facilities in the rear of the berths) are left on balance of governmental stevedore companies (GSC).

It should be noted that strategic objects are usually objects that require "long" investments, payback period of which is measured with decades.

Overload capacity of ports are usually attributed to nonstrategic objects, i.e. subject to privatization.

Nowadays Ukraine has 13 seaports, excluding five Crimean ports. According to state property fund, infrastructure of ports is worn out on 70÷80%, which means that it is necessary to refresh or modernize handling equipment of Ukrainian ports.

There are above 500 gantry cranes working in Ukrainian ports, average age of which is above 37 years, some gantry cranes are working for 50 years. Said age exceed European standards almost twice. Ports production capacity are usually presented as park of gantry cranes imported under the program of the ministry of the navy of the USSR, book value of which far exceeds market price due to multiple indexing.

The institute Chernomor NII Project analysis of berthing facility operating time showed that by now the operating time of most berthing facilities is 30-40 years and it is getting close to standard or exceeds standard. Based on the same source, the amount of constructions, that are being exploited for 30 years and more, is about 70%. [5].

These numbers testify that most of the handling equipment and berthing of ports and former Ukrainian shipyard do not meet the modern requirements. Besides, as seen on practice, the technical exploitation of the port hydrotechnical structures is often carried out with significant deviation from regulatory requirements for different reasons. There are facts of reduced security level of port hydrotechnical equipment exploitation, among which there should be noted:

– Unsatisfactory work of tally services and department of technical exploitation on compliance with modes of berths cargo operation, including assessment of overloads impact on structures technical state Permissible loads on piers are almost not revised;

– Maintenance work and overhaul are held out of time;

– Carrying out repair work not in full, often with an involvement of organizations without any experience of working with marine hydrotechnics;

– Untimely carriage of engineering inspection and a hydrotechnical structures survey;

– Absence (or presence not in full) of technical documentation;

– Design operation mode is being changed arbitrarily on active docks (re-profiling, increase in design depths and others);

– Projecting, survey, diagnosing, certification are being carried out without reconciliation with the state territorial organization that is responsible for direction «sea transport».

Cases of scientific and technical products output (prospecting, projects, surveys with the assessment of technical condition, certification and recommendations for change of exploitation mode) by different unspecialized structures have become frequent, often with the violation of current standards and legal system of Ukraine at low technical and engineering level, excluding the perspective of development of enterprises.

Research results and analysis of them. A significant change of technical normative base, for the period of existence of Ukraine as an independent country, led to a situation when the majority of port infrastructure objects, that were commissioned in times of former USSR and are exploited nowadays, do not meet the requirements presented by these standards. For example, with implementation of the building code V.1-1-12:2006 and the reduction V.1-1-12:2014 «Building in seismic districts of Ukraine», formally none of the hydrotechnical constructions of ports in Odessa and Illichevsk, built until year 2006, does not provide the required class of responsibility, since their calculation and designing were conducted without inclusion of a possible seismic event happening.

Wherein strategic objects of the port infrastructure, specifically docks, protective and bank-reinforcing structures, the water area, underwater channels that provide port activity and so on, are less vulnerable, from a strategic planning perspective, in long-term planning for the macroeconomic risk factor connected with a sudden change of the world market conjuncture, structure of the freight traffic, decrease in investment activity and others.

Figure 2 – Deformation of the berth due to overloading

In the process of marine hydrotechnical constructions projecting, it is necessary to consider a lot of natural factors. These include hydrological, hydrographic, engineering geological, geomorphologic and meteorological conditions of the building district. Hydrological conditions include the sea wind, ice conditions, level fluctuations, sea currents, tsunami waves. Hydrographic conditions include the depth of water, the seabed and adjoining coast topography. Engineering geological and geomorphologic data about the seabed structure, physical and mathematical properties of bottom soils and sediment migration has a special significance. The main meteorological factor is the wind regime (speed, direction, duration). Also in the process of marine constructions, it is necessary to perform seismic calculations. Herewith constructive features of a structure and existing engineering geological conditions of the building area should be considered. [6].

The present technical condition of hydrotechnical constructions that were built in soviet times, is unsatisfactory and in some cases emergency for many reasons.

First of all, the main ones are insufficient considering of the factors above; violation of building technology in hydraulic structures; using substandart building materials and products.

Ukraine nowadays is one of the most dangerous countries in the world in terms of man-made disasters. It is extremely densely saturated with industrial infrastructure of various purposes, in most of the cases extremely worn out. For many years, any attention was not paid to ensuring the safe operation of enterprises. It led to the fact that the number of potentially dangerous objects reached several thousands and continues to grow. And some of them, such as the Odessa Port Plant, in case of seismic catastrophe, can harm a millionaire city with its poisonous emissions.

Odessa is a part three most seismically dangerous regions of Ukraine (Crimea – 6-9 points, Odessa – 6-9 points, Transcarpathian region – 6-8 points).

For many years, major repairs of structures were not carried out, as a result of which the life of structural strength is almost exhausted, which threatens to lose it even with weak earthquakes. Vulnerability grows every day. Currently, intensive development of territories for the constructions is under way. Not only of residential buildings, but also large, unique structures, which destruction can lead to significant economic and social losses from seismic influences (Figure 3-5).

The development of the Black Sea shelf, creation of environmentally hazardous industries - oil terminals and oil pipelines, occurs because of seismicity increasing on the coast. And today, with the scientific substantiation of the strategy for port infrastructure development, special attention should be paid to issues of ensuring the reliability and safe operation of both newly constructed and exploited and reconstructed facilities.

Figure 3 – Significant deformations of the berth territory from the earthquakes action

Figure 4 -- Tilting the crane due to the action of seismic forces

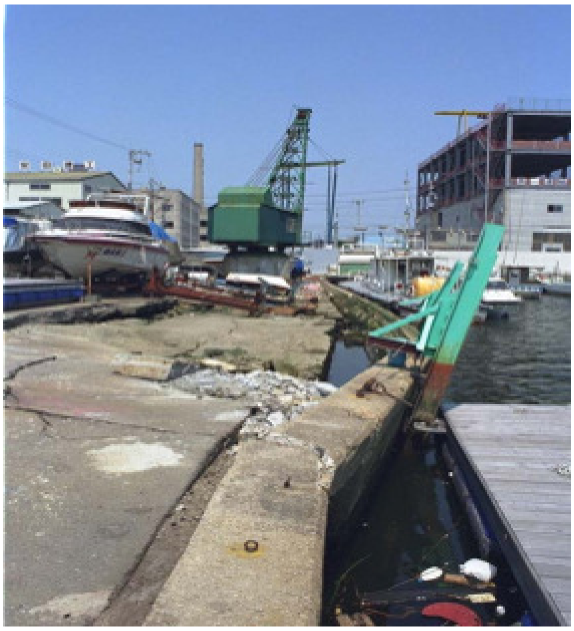

Figure 5 – Deformations of the berthing facility cordon line

Earlier, before 2007, structures calculations and their bases located on landslide slopes were made in accordance with SNIP 2.01.07-85* «Loads and impacts» and SNIP II-7-81* «Construction in seismic regions», and in accordance with these SNIPS, the seismic hazard in the south of Ukraine was equal to 6 points, so calculations for seismic impacts were not required.

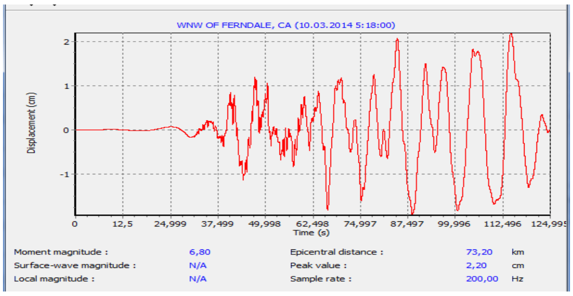

Figure 6 – Calculated accelerogram

But since 2007 DBN B.1.1-12:2006 «Building in seismic regions of Ukraine» has been put into operation. According to the OCP-2004-A map, the Odessa region is in the zone with a seismicity of 7 on the EMSH-98 scale.

Strength calculations of hydraulic structures, landslide slopes stability, structures located on them and coast protection structures in the Odessa region should be carried in accordance to seismic loads. These effects can be specified either by the linear-spectral method or by a direct dynamic method, according to the calculated accelerograms of the earthquake which represent a three-component function of oscillations acceleration in time.

The necessity of calculation joint work of the «foundation-construction» complex and various loads and impacts, leads to the complication of the structures basis design. Calculation of emergency (seismic) impact using calculating accelerograms, accounting nonlinear physical properties, accounting construction stages, complicates the task of calculating and designing marine hydraulic structures, at the same time makes possible to obtain rational solutions that provide specified operational properties with regulated reliability and safety parameters.

Conclusions. Calculating, design, technical operation, repair, inspection and reconstruction of port hydraulic structures issues have traditionally been regulated by the requirements of departmental (former Ministry of the Navy of the USSR) regulatory documents. There are no actualized versions of these documents that correlate with state building regulatory documents.

Regardless of port industry current strategy development, a systemic state approach is required to create a modern technical regulatory framework and effective methods for monitoring compliance, especially for strategic port infrastructure facilities.

While designing hydraulic structures, it is necessary to account several numbers of factors and requirements that would ensure an effective structures operation, reliability and durability.

According to the analysis basis of surveys and the requirements existing regulatory documents results, it is necessary to designate the design parameters of natural and seismic impacts on the designed structures, considering their life service.

The hydraulic structures design should be carried out considering the transshipment complexes at the wharf with proper scientific support.

The usage of new constructive solutions requires appropriate experimental studies, including in-situ and laboratory conditions.

During the project implementation, it is necessary to comply the requirements of regulatory documents that ensure the proper quality of installation and building works.

At the present, it is necessary to improve the calculating methods of the stress-strain state of the soil foundation under hydraulic engineering structures, considering alternating effects.

References

1. DBN V.2.4-3:2010. (2010). *Hydrotechnical building. Basic statements*. Kyiv: Minregionbud Ukraine.
2. DBN B.1.1-12:2014. (2014). *Construction in seismic regions oa Ukraine. The official version*. Kyiv: Minregionbud Ukraine
3. Kendzera, A., Iegupov, K., Semenova, Y., Iegupov, S. & Lisovyi Y. (2018). Use of seismological information for the design of multistory buildings. *16th European conference on earthquake engineering (Thessaloniki)*.
4. Nemchynov, Ju. (2008). *Seismic buildings and structures.* Kyiv.
5. Nemchynov, Ju., Maryenko, N.G., Khavkin, A.K. & Babik, K.N. (2012). *Design of structures with a given level of seismic resistance*. Kyiv.
6. Patinski, V. (2012). Status port hydraulic structures of Ukraine. *Ports of Ukraine Journal*, 05(117), 5-9.
7. Nemchinov, Y., Havkin, D., Marenkov, M., Dunin, V., Babik, K., Yegupov, K., Kendzera, A., Yegupov, V. (2013). Practical aspects of the dynamics of buildings. *Scientific and production magazine Building Ukraine,* 6, 6-21.
8. Pustovitenko, B., Kulchitsky, V. & Pustovitenko, A. (2004). New data on seismic danger of the city of Odessa and Odessa region. *Construction designs. Mechanics of soil, geotechnics, foundation engineering,* 61, 388-397
9. Borg, R.C. (2007) *Seismic performance, analysis and design of wharf structures: a comparison of worldwide typologies* / A Dissertation Submitted in Partial Fulfilment of the Requirements for the Master Degree in Earthquake Engineering.
10. *Seismic Design Guidelines for Port Structures*. (2001). International Navigation Association. Tokyo: Balkema Publishers.
11. Egan J., Hayden R., Scheibel O. & Seventi G. (1992). Seismic repair at Seventh Street Marine Terminal. *Grouting, Soil Imporovement and Geosynthetics, Geotechnical Special Publication*, 30, 867-878.
12. Chen Wai-Fah, Scawthorn Ch. (2003). *Earthquake engineering handbook.* CRC Press LLC.

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Soil cement as a constructive material for anaerobic bioreactor corps

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One of the promising directions of modern alternative energy is anaerobic biotechnology for the production of biogas (so-called sewage gas). The main problem of hampering widespread use is biogas plants high cost. A new design of an anaerobic bioreactor for the production the biogas is proposed, where soil cement is used as a constructive material of continuous and monolithic construction of the corps bottom and walls. A structural and logical scheme for designing a bioreactor from soil cement for specific capacities of pig farms in geotechnical and climatic conditions has been developed and presented. Also the technological features of object construction by drilling and mixing technology are presented.

Keywords: soil cement, bioreactor, corps ,chink, biomass, substrate, drilling and mixing technology, modifier, climatic parameters, fermenter

Грунтоцемент як конструктивний матеріал корпусу анаеробного біореактора

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Подано нову, енергоефективну й компактну конструкцію анаеробного біореактора каскадного типу, що поєднує в собі ефекти: максимальної площі поверхні для виходу газу; забезпечення умов для механічного перемішування (раціональна форма камер бродіння без глухих кутів); мінімальних енергозатрат на підігрів (мінімальна довжина зовнішнього периметра); можливості централізованого виведення осаду з усіх камер бродіння. Охарактеризовано взаємозв'язок закономірностей щодо прийняття співвідношень діаметра і глибини закладання в ґрунт майбутньої споруди. З'ясовано, що взаємозв'язок між цими параметрами обумовлюється технологічними особливостями виходу біогазу, умовами його перемішування, кліматичними параметрами – промерзання ґрунту (мінімізації витрат на підтримання необхідної температури для анаеробного процесу), технологічними й економічними параметрами влаштування корпуса біореактора буровим верстатом, геологічними та гідрогеологічними параметрами будівельного майданчика й ін. Розроблено і представлено структурно-логічну схему проектування біореактора з ґрунтоцементу для конкретних потужностей свинокомплексу, геотехнічних та кліматичних умов. З метою поліпшення гідратаційних процесів, зниження усадки ґрунтоцементу, зменшення тріщиноутворення та підвищення морозостійкості запропоновано модифікувати ґрунтоцемент ферментним препаратом «Дорзин». Також наведено технологічні особливості зведення об'єкта за бурозмішувальною технологією.

Ключові слова: ґрунтоцемент, біореактор, бурозмішувальна технологія, модифікатор, кліматичні параметри, ферментатор

Introduction. In the context of price constant increases and ecological situation deterioration, more and more attention is being paid to alternative sources of energy supply. Modern alternative energy is represented by a wide spectrum of tools and sources. Their analyze due to the climatic conditions of Ukraine and Kyrovohrad region in particular is very important; the greatest prospect of the biogas technologies and solar energy spread is obvious.

Available in each farm or farmstead, large volumes of organic waste [1] are expedient to be utilized with the maximum total effect of solving complex problems: energy, ecological, farm supply with fertilizers, groundwater contamination prevention etc.

Biological wastes of agricultural production, as a rule, are exported outside the territory of farms and are accumulated for the purpose of natural utilization (degradation). Domestic waste (so-called solid household waste SHW) is exported to landfill solid waste. Both of these types of waste causes pollution, oxidation of the soil, makes it unpropriate for use without the use of high-value measures for the rehabilitation. Fissile material products penetrate into groundwater, lead to emissions of greenhouse gases and many other negative consequences

In the case of household organic waste, the problem of utilization is solved by sorting and subsequent processing. The situation on the agro-industrial enterprises or farms is simpler: sorting is not necessary there because the waste can immediately be recycled by bioreactor.

Analysis of recent sources of research and publications. Biogas reactors in Ukraine have not become widely distributed due to their considerable cost, the capriciousness of methane fermentation technology, and high energy consumption [1 – 4].The deviation of 3°C from the optimum for the process has already slowed down the methanogenesis. Significant influence on the processes of biogas production has environment mobility, stagnant zones absence, thermostabilization. During the cold period, it is necessary to minimize heat loss for substrate heating and equally distribute heat throughout the reactor volume. Practically there are no automation means of biogas production process. But it shoul be remembered that the reactor stop per hour can lead to energy problems in the company, because the process of fermentation is associated with the life of anaerobic bacteria and they restart their activity only for a few days or weeks. The process needs to be adapted to the conditions of Ukraine and permanent automatic monitoring. For Ukrainian conditions, cheap and reliable reactors are needed that is equally effective both in warm and cold seasons.

Anaerobic bioreactor for biogas and organic substrate production is a a closed container, filled with biological waste without air access at a predetermined stable temperature (from +25 to +55°C). As a result of methanogenesis, biogas and an ecologically safe organic substrate that fall out as solid precipitate, organic fertilizer can be used in agriculture as a material for the body bioreactor traditionally used concrete, metal, polymers.

Selection of previously unsettled parts of the general problem. The disadvantages of known bioreactors are their relatively high cost and lack of concrete structure of a complex curvilinear form techniques installing that provide the best conditions for thermal insulation, mixing, substrate surface crust destruction, providing the maximum area for biogas output from the substrate, periodic removal of solid precipitate, and maintenance. Also, a significant disadvantage is the insufficient durability of bioreactor body, due to corrosion instability.

An option to solve the problem of reducing the bioreactor cost and at the same time to provide body corrosion resistance, soil cement is used as a material for both bottom and walls [5]. There are some advantages for soil cement use in bioreactor body [6–10]:

– positive experience of using soil cement as a material for the construction of sludge cesspool for toxic waste;
– high water resistance W12-14;
– low cost of manufacturing due to natural soil use from the foundation pit;
– high compressive strength, 2 MPa;
– resistance to aggressive components (chemical resistance);
– longevity, lifetime of more than 300 years;
– soil cement is environmentally safe;
– frost resistance within M25;
– the soil cement has ability to become more solid with time.

Problem statement.

1. To propose a new, energy-saving and technological considering ease of operation in Ukraine, the anaerobic bioreactor has been constructed using such constructive material as soil cement, modified soil cement.
2. To develop the structural-logical scheme of designing a bioreactor from soil cement for specific household conditions, in particular, the pig complex LLC «Liga» in the village Company of Kirovograd region.
3. To develop the technological process of building a body bioreactor from soil cement, using. drilling and mixing technology.

Main material and results. The proposed construction of a bioreactor [5] relates to the technology of biogas production by anaerobic decomposition of various biological waste (livestock, poultry, plant growing, food industry, solid household waste, sewage), and can be used in agriculture, food industry, urban solid landfills household waste, sewage treatment plants, organic fertilizer manufacturing plants, such as mulch, substrate.

The purpose of the bioreactor proposed design is to improve the anaerobic bioreactor, ferment-gasholder by installing a solid and monolithic body from the soil cement, the reliability and durability of the bioreactor work, its tightness increases. Moreover the cost reduction and the term of construction under the conditions of natural resources rational use highlights obvious benefits of proposed design.

Fig. 1 shows a top view of a bioreactor installation consisting of loading and pumping unit and directly the bioreactor itself. Where 1 – receiving capacity for biological waste products, 2 – pump, 3 – capacitance - dispenser. Capacities 1 and 3 are formed by walls and bottom of soil cement.

An anaerobic bioreactor for the production of biogas and organic substrate consists of a body 4 of a cylindrical shaped (see fig. 1) with three internal vertical partitions 5, which are arranged relative to one another at an angle of 120 ° and form three cameras for anaerobic fermentation, respectively 6, 7 and 8. Outside of the bioreactor is a collector for mass that brooded 9 formed by the walls and bottom of the soil cement. The cylindrical body 4, the three internal partitions 5 and the joint bottom 10 (fig. 2) are made in a «monolithic» version - as a whole from the soil cement. On the surface of the interior partitions 5, heat exchangers 11 are provided with a polymer tube connected to a heat pump (not shown in fig.). In the fermentation chamber 6, by means of a traverse 12, a biomass mixer is installed, including a hydromotor 13, a reducer 14 with a vertically-mounted shaft, on the bottom of which is a short-base single-shaft polishing screw 15 fixed, and a trapezoid frame with grids 16 for the destruction of the surface crust is fixed in the upper part. In partitions 5 there are openings for overflow pipes 17. The scheme (scanning of bioreactor chambers) of biomass circulation is shown in fig. 3.

Gas accumulation and its initial temporary storage takes place in the cavity of the gasholder 18 formed by a gas cap 19, which is tightly and hermetically installed in the grooves of the soil cement walls of the body 4. In the upper part of the cap 19, which is made of a dark color material for additional heating of the bioreactor from the sun, a means of control and automatic control 20, as well as an outlet gas pipe 21, is installed.

In the lower part of the bioreactor there is provided a reinforced concrete pipe 22 with installed in it, by means of supporting bearings, long-base auger 23 for the removal of a precipitate (substrate). Herewith, the surface of the bottom 10 of the body 4 of the bioreactor in the fermentation chambers 6, 7, 8 and the collector of the fused mass 9 has internal deviations to the apertures with the hydraulic shutters 24 installed therein, through which, periodically and in a definite sequence, the precipitate (substrate) enters the inner cavity tube 22 with screw 23.

The anaerobic bioreactor for the production of biogas and organic substrate works in the following way. By tape conveyor, tractor and loader, the raw material of biological origin is fed into a tank 1, there they add water and mix with a screw mixer to the required consistency. Subsequently, the pump 2 supplies the biomass to the dosing tank 3 and controls the required temperature of the mixture (within +25…+55°C) and maintains the required biomass level in the tank 3. Herewith, the excess biomass (fig. 3) through pipeline 17 enters the lower part of the bioreactor in a fermentation chamber 6 where it is heated by a biomass heater 11 and mixed with a short-base empty auger 15. The fermentation process begins. The superficial crust of solid elements formed on the surface of the biomass in the chamber 6 is destroyed by trapezoidal grids 16, which allows gas bubbles to enter the cavity 18 of the gas cap – gasholder 19 without interference.

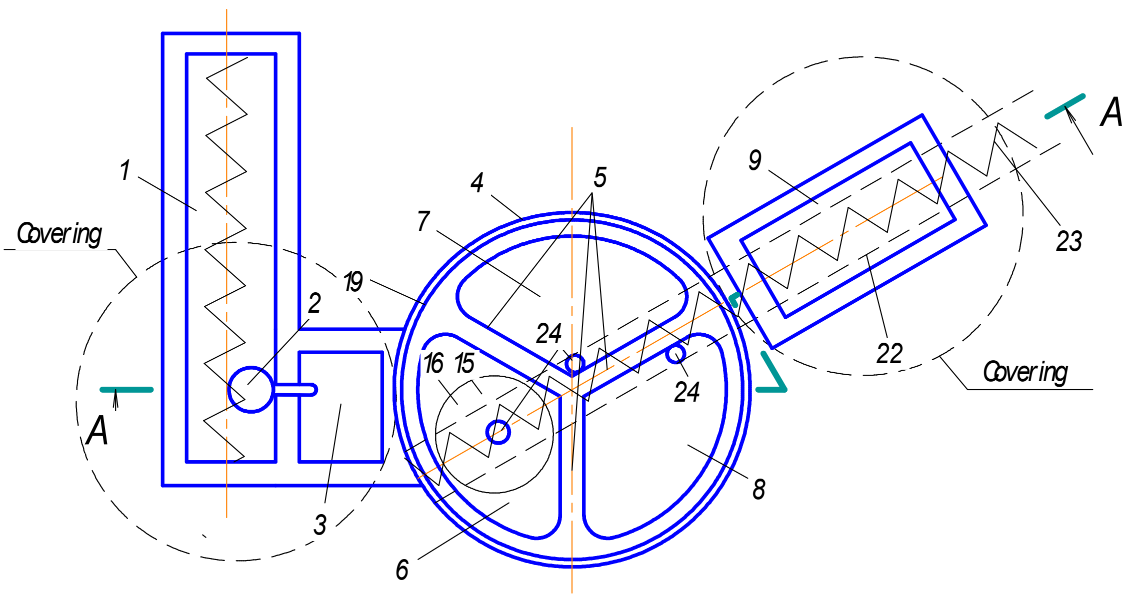

Figure 1 – Bioreactor installation (top view) [5]

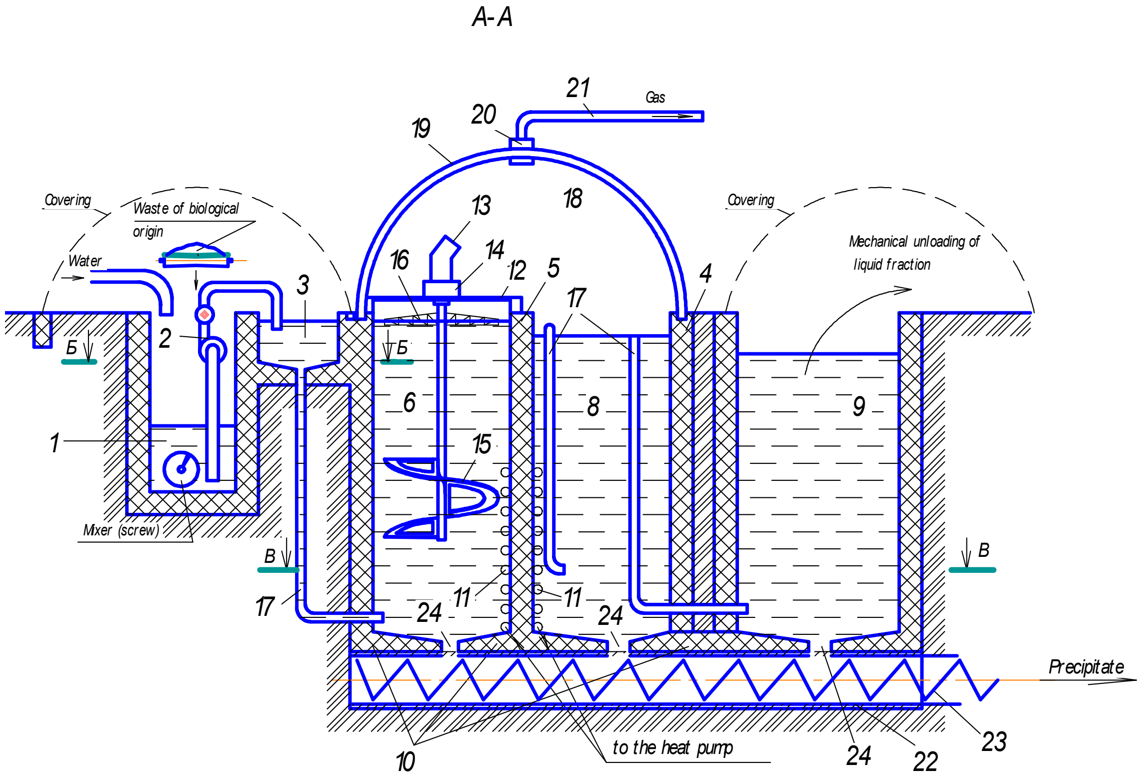

Figure 2 – **Slash A-A bioreactor from Fig. 1 [5]**

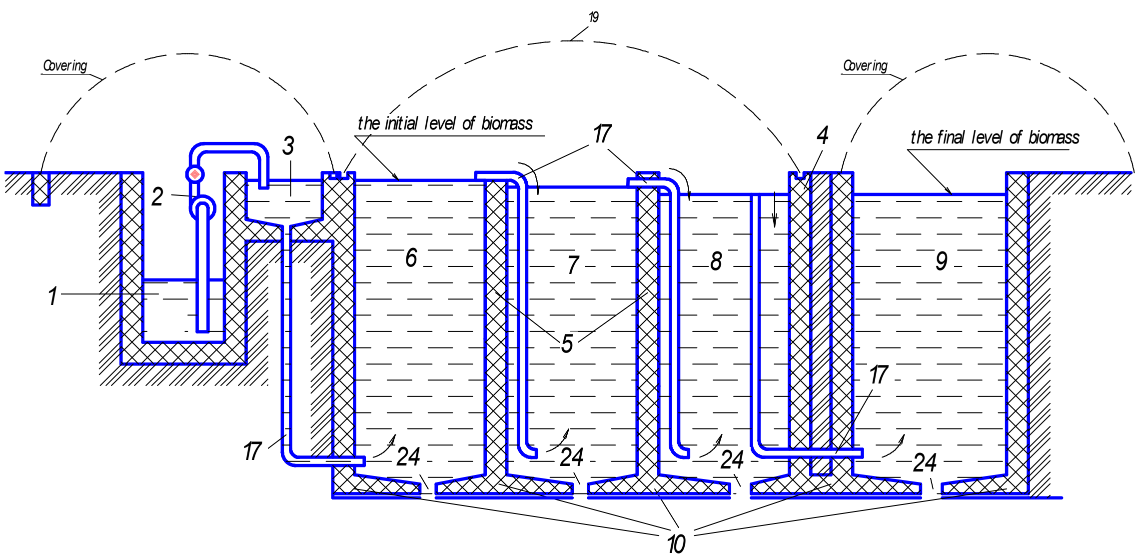

Figure 3 – **Scheme (bioreactor chamber scanner) of biomass circulation [5]**

Partially burnt out, the biomass that takes position in the upper part of the fermentation chamber 6 enters the transient pipe 17 into the lower part of the intermediate fermentation chamber 7, stirring and heating from the partition 5 with heaters 11. There is a continuation of the fermentation of biomass in the intermediate chamber of fermentation 7 and biogas production, which freely flows into the cavity 18 of the gas cap of the gasholder 19. As the chamber 7 is filled, the biomass from the upper part enters through the intermediate pipe 17 between the chamber 7 and the chamber of final fermentation 8 in its lower part, mixing and heating from the partition 5 with the heaters 11. There is a final fermentation of biomass in the chamber 8 with emission of biogas into the cavity 18 of the gas cap – of the gasholder, 19 where the gas is stored, it's kept for a certain period and, by means of automatic devices 20, given up by pipe 21 to places of longer storage, purification or direct use.

From the chamber of final fermentation, biomass, through the transfer tube 17, enters the collector of mass that brooded 9, from which it is mechanically removed by a clamshell, a noria, etc.

At the same time, the process of fermentation and the location of biomass in chambers 6, 7, 8 and collector of fermented mass 9 is accompanied by the fall of a biomass precipitate (substrate). The sediment accumulates at the bottom of 10 chambers 6, 7, 8, and 9, and due to internal deviations it gets more concentrated in a joint with a concrete pipe 22 openings with hydro shutters 24 and is given by a screw 23 to the place of collection, packaging. The frequency, sequence, opening time of the hydraulic shutters 24 and the activation of the screw 23 may occur, both manually and automatically, in accordance with the bioreactor control program.

An anaerobic bioreactor for the production of biogas and organic substrate is designed to work both continuously and discretely at any time of the year.

Bioreactor designing. Depending on the initial conditions, namely: the type and daily output of biological waste, calculate the internal volume of the reactor.

Suppose the farm has 10 heads of cattle, 20 pigs and 35 chickens. In the course of the day, the excrement will be: 55 kg from 1 cattle; 4,5 kg from 1 pig; 0,17 kg from 1 chicken. The mass of daily waste is:

$10 \times 55 + 20 \times 4{,}5 + 0{,}17 \times 35 =$

$= 550 + 90 + 5{,}95 = 645{,}95$ kg.

The optimum moisture content of raw materials for loading in a bioreactor is 85%.

Humidity of fresh excrement of pigs, cows is 86%, and chickens excrement 75%. To bring the moisture content of chickens excremant to 85%, add about 0,5 liters of water (about 0,5kg). Then the daily load of the reactor will be 646 kg.

The daily loading of the reactor is allowed to be no more than 10% of its full load.

Then the complete loading of the reactor:

10×646 kg $= 6460$ kg $\approx 6{,}5$ ton

For LLC «Liga» smt. Kompaniyivka Kirovograd region the calculation is carried out for 5,3 thousand heads (the industrial capacity of the enterprise is 6 thousand heads.) The reducing number of livestock may be explained by the presence of piglets, seasonal variations, etc.

Then the mass of daily waste will be:

$m = 5300 \times 4{,}5 = 23850$ kg.

Full load of the reactor:

$m = 10 \times 23850$ kg $= 238500$ kg $= 238{,}5$ t.

According to [11], the bulk weight ρ of fresh pigs excretes varies within $\rho = 1013\ldots1400$ kg/m^3.

Reactors volume: $V = \dfrac{238500}{1200} = 198{,}75 м^3$

The result of the previous calculations is that reactor must be with a useful volume of 200 m^3.

The issues of the ratio of diameter and depth of laying to the ground of the future construction remains. The interconnection between these parameters is determined by the technological features of the biogas output, the conditions of its mixing, the climatic parameters - freezing of the soil (minimizing the cost of maintaining the required temperature for anaerobic process), technological and economic parameters setting up of the bioreactor with a drilling machine, geological and hydrogeological parameters of the construction site and other. This is a complex optimization task.

It should be noted that the diameter of the bottom of the reactor is the greater part of the conditional foundations of the future structure (fig. 4), and the accepted value of the outer diameter of the container must satisfy the condition $P_{ум} \le R$ (the second group of boundary states, the calculation is carried out according to the classical methods). The adoption of the minimum value of the outer diameter of the reactor body is based on calculations for the designing of foundations [12, 13] and depends on soil, hydrogeological, climatic conditions and loads from the structure and substance in it. An increase in the diameter of the tank will positively affect the reduction of the pressure under the $P_{ум}$ structure, the gas outlet area and the liquid *grad p* pressure gradient, but will increase the outer perimeter, which, in the presence of overlying soils in the upper layers will cause «frost heave». Minimal effects are the occurrence of additional internal uneven tensions in the body of the soil cement tank, which can lead to loss of integrity and tightness. To solve this problem, it would be advisable to create anti-friction collars at the depth of the seasonal freezing, insulation of the upper perimeter, the arrangement of drainage.

Placing the reactor vessel below the level of the daily surface, using three radially installed through 120° internal partitions and filling the inner cavity by the substrate can minimize the load on the walls and bottom. In the working and long-term position, walls and bottom of the reactor, passive pressures from the substrate $P_{суб}$ and the external natural soil of the undisturbed structure of the $P_{гр}$ are perceived, which can be assumed to be equal in the first approximation. The thickness of the bottom and walls will be determined by the value of: the filtration coefficient; passive pressure on the wall in the sectors between the partitions; technological possibilities of mixing working equipment to provide a continuous and homogeneous enclosure structure from soil cement. A value that satisfies all conditions is accepted.

The main criterion for calculating the thickness of the bottom and walls of the reactor vessel is the value of the filtration of moisture from the substrate, which is assumed to be zero. The well-known formula A. Darcy may be adopted as a basis: $Q = k_ф \dfrac{\Delta H}{\Delta l} F$, that, taking into account the coefficient of permeability for different substances, k (P.G. Nutting) $k = d_{eф}^2 Sl(m, \varepsilon)$ will take the form: $v = \dfrac{k}{\mu}\dfrac{\Delta p}{\Delta l}$ and will represent the equation of L.S. Leibenzon:

$$\nu = \frac{d_{e\phi}^2 Sl(m,\varepsilon)}{\mu} \frac{\Delta p}{\Delta l},$$

where $d_{e\phi}$ – is the effective diameter of the particles (the diameter of the particles of equivalent fictitious soil, the hydraulic resistance of which is equal to the hydraulic resistance of the real breed); $Sl(m,\varepsilon)$ – is the Slighters number (dimensionless), as a function of the coefficient of porosity m and the structure of the porous space ε (under the structure of the porous space understand the shape and size of individual pores, their quantitative ratio and compatibility); μ - is the dynamic coefficient of viscosity of the liquid.

The establishment of climatic parameters in the design of a future bioreactor can be carried out according to the proposed method [14] for determining the climatic loads at a given geographic point must be used data of the local network of weather stations (fig. 5).

The area equation, which reflects changes in the climatic factor Z in the vicinity of the design point, can be written as:

$Z = A + BX + CY$,

where X, Y – coordinates of the meteorological station or the design point;

A, B, C – Parameters determined according to the network of meteorological stations.

Parameters A, B, C are determined by the least squares method according to the data from all weather stations by the point of the design point, which can be implemented in any environment of any computing complex, in particular, Microsoft Excel. The coordinates of the meteorological stations and the design point X, Y can be given in the form of rectangular coordinates in kilometers relative to the conventionally selected center, or the values of longitude and latitude of the terrain in degrees published in meteorological guides, determined from maps of Google Maps or other cartographic systems.

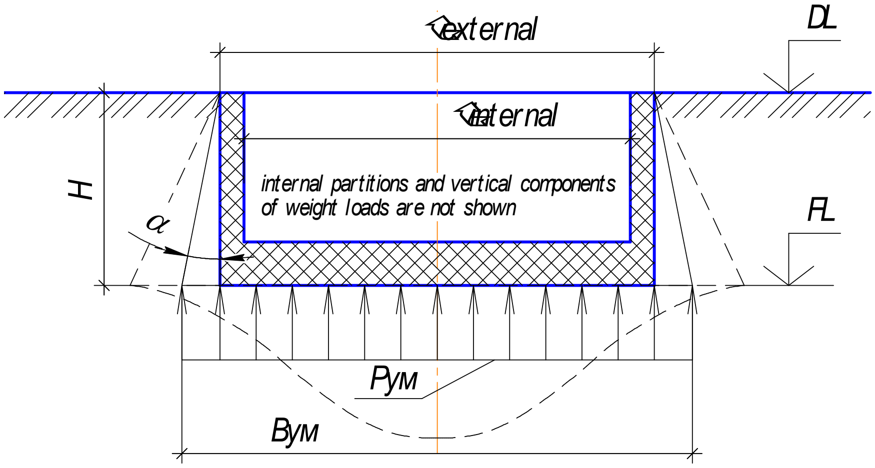

Figure 4 – The scheme of pressure transfer to the ground due to the resistance of the soil on the side surfaces and under the bottom of the fermentor

Figure 5 – Network of weather stations in the region [14]

Geological parameters of the construction site. Ideally, the soil for laying the bottom and walls of the soil cement tank should have at least 25–30% of cohesive colloidal particles (that should passing through a sieve with a cell size of 0,07 mm) and contain a minimum of large granular particles such as gravel. That is, it should be clayey or dusty soil: clay, or loam. Modification of soil cement by the introduction of the enzyme preparation «Dorsin» [15] in the most optimal amount (to 0,05%), improves hydration processes, significantly reduces shrinkage of soil cement, reduces cracks formation, increases frost resistance. A well-known positive experience with the use of enzyme modifier for clay soils Dorsin in road construction.

The enzyme preparation «Dorsin» is a composition of substances that were mainly formed during the cultivation of microorganisms (yeasts of the genus Saccharomyces) in a complex nutrient medium with some additives. The basis of the nutrient medium is beet molasses.

Developed, and in fig. 6, a structural-logical block diagram of the design of a bioreactor from soil cement is presented.

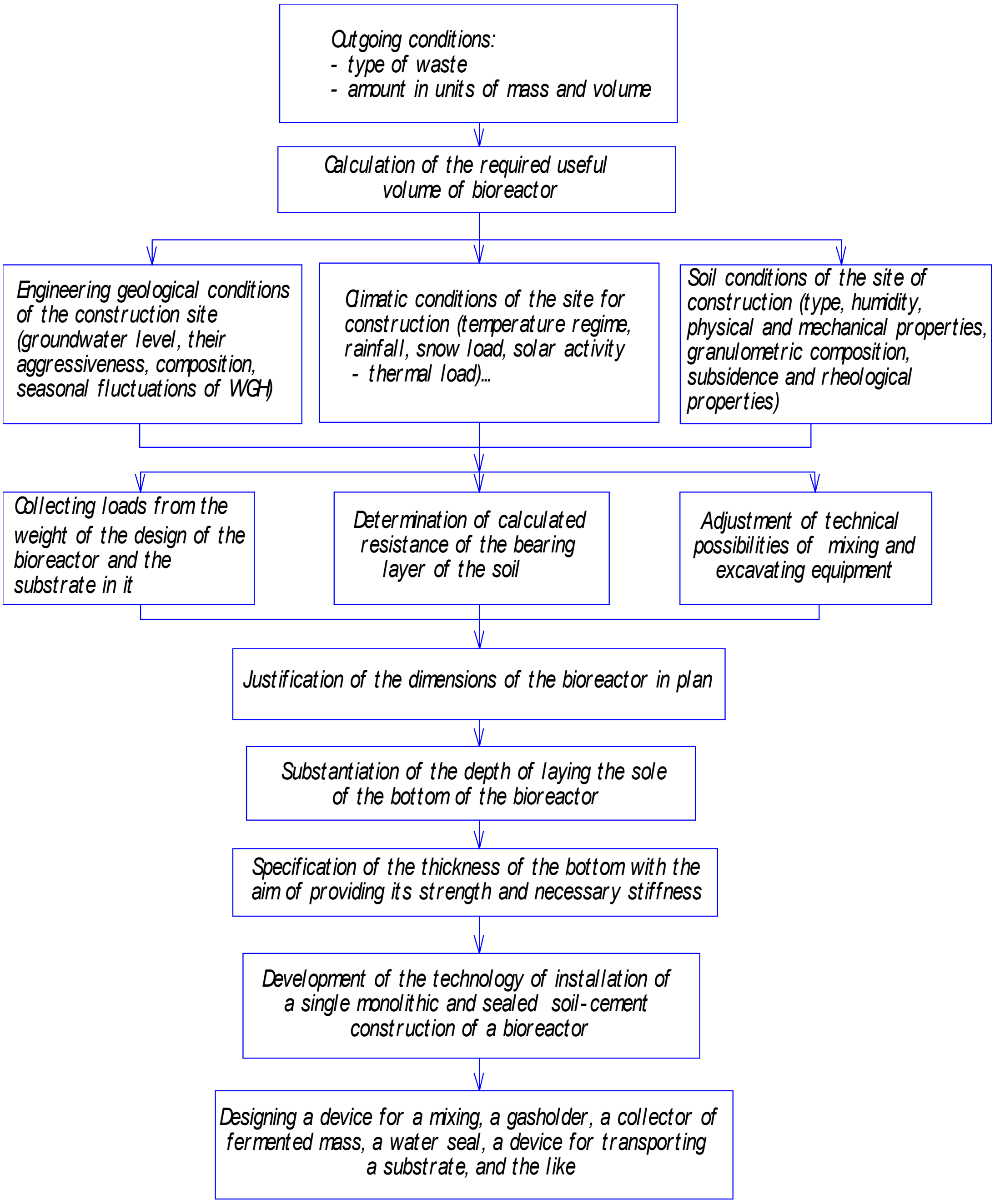

Figure 6 – Structural and logical block diagram of designing bioreactor from soil cement

Execution of bore-mixing works can be carried out by the drilling machine BM-811м on the basis of the car URAL [16, 17]. Cylindrical construction is non-technological in terms of convenience, accuracy of the positioning bore-mixing body and the displacement of the drilling machine. One solution, adopt a linear scheme of the reciprocating motion of the drilling machine, shown in fig. 7. In this case, it is necessary to arrange 682 wells with a depth of 4,0 m, a diameter of 0,5 m with a distance between adjacent wells 0,4 m. The main lines show the wells where the injection of cement occurs throughout the depth of the well, auxiliary - thickness of the bottom of the reactor - 0,9 m. Works on soil reinforcement are performed in three changes within 10 days. The average length of installation of one borehole is about 25 minutes. As a normative source, a document of the RF (Code of the rules of joint venture CP 291.1325800.2017 Structures of cement reinforced, designing rules was used. [18]. Cement outgoings ranges from 500 to 700 kg/m^3 and depends on the type and granulometric composition of the soil. The set of strength of the soil cement acording to the project lasts for 28 days, after which by the excavator with the equipment «reverse shovel» or «grab» holdings perform excavation of soil from three chambers and then perform a manual cleaning. The total duration of works is 40 days.

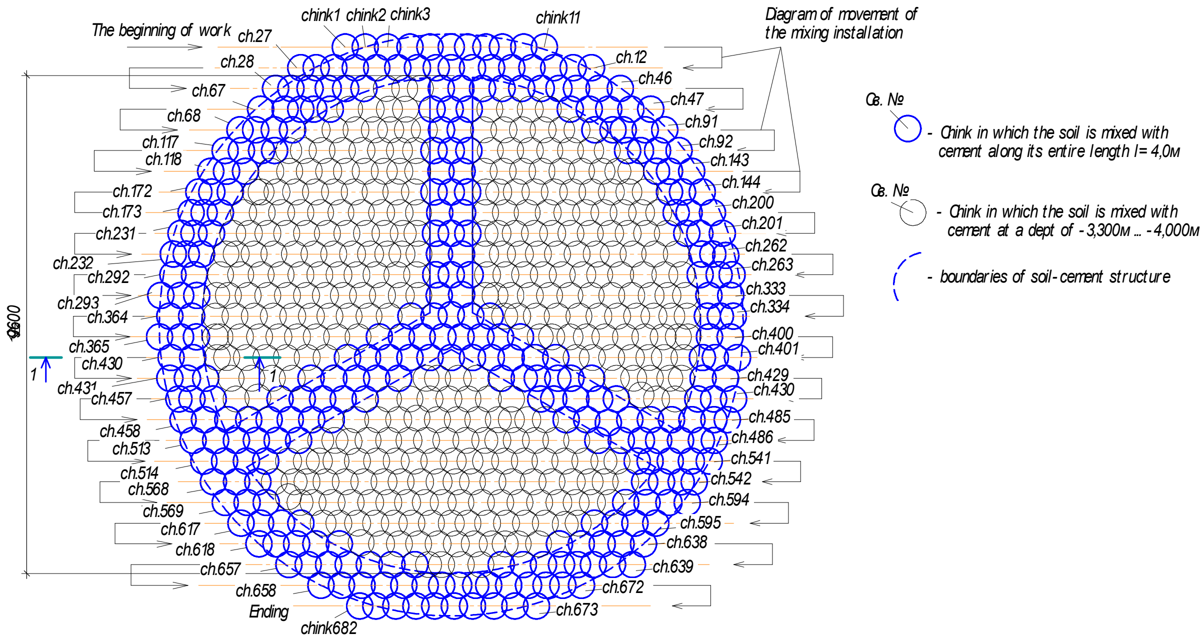

Figure 7 – Technological sequence of drilling and mixing works

Conclusions:

1. Expediency of soil cement use for the arrangement of ferment anaerobic bioreactor that produce biogas and organic substrate is proposed and determined.

2. A new, energy-efficient and compact anaerobic bioreactor design is introduced, which combines such effects as maximum surface area for gas outlet; provision of conditions for mechanical mixing (fermentation chambers rational form without deafening angles); minimum energy consumption for heating (minimum length of the outer perimeter), the possibility of sediment centralized outflow from all fermentation chambers.

3. Bioreactor design proposed design application enables to solve a number of significant problems: the rational use of natural, including renewable energy sources; ecological utilization of wastes; rational use of natural resources for construction; obtaining environmentally friendly fertilizers for agrocomplex.

4. The structural-logical block diagram of bioreactor designing from soil cement has been developed. The starting data are the type of biological waste and the daily amount of waste in units of mass and volume.

5. It was designed and given the recommendations for the bioreactor location for a particular company in the data engineering geological and climatic conditions.

6. Recommendations on the technological sequence of fermentor placement from soil cement cylindrical elements by the bore-mixing technology have been developed.

7. Perspective is to establish the appropriateness of soil cement use for constructing the bottom of artificial reservoirs and as separate foundations for solar cells.

References

1. *Державне агентство з енергоефективності та енергозбереження України*. Режим доступу: http://saee.gov.ua/uk/bioenergy.
2. Ратушняк, Г.С., Джеджула, В.В. & Анохіна, К.В. (2010). *Енергозберігаючі відновлювальні джерела теплопостачання*. Вінниця: Вінниц. нац. техн. ун-т.
3. Едер, Б. & Хайнц, Ш. (2008). *Біогазові установки. Практичний посібник*.
4. *Компактні біогазові установки контейнерного виконання виробництва Польщі.* (2016). Режим доступу: http://atagos.com.ua.
5. Карпушин, С.О., Клименко, В.В. & Шиндер, А.В. (2018). Патент України 124712. Київ: Державне патентне відомство України.
6. Зоценко, М.Л. & Тимофєєва, К.А. (2012). Патент України 71256. Київ: Державне патентне відомство України.
7. Azadi, M., Taghichian, A. & Taheri, A. (2017). Optimization of cement-based grouts using chemical additives. *Journal of Rock Mechanics and Geotechnical Engineering*, 9(4), 623-637. Retrieved from https://www.sciencedirect.com.
8. Fan, J, Wang, D. & Qian D. (2018). Soil-cement mixture properties and design considerations for reinforced excavation. *Journal of Rock Mechanics and Geotechnical Engineering*, 10(4), 791-797. Retrieved from https://reader.elsevier.com
9. Вагидов, М.М. & Зоценко, Н.Л. (2012). Грунтоцементные основания и фундаменты. *Вестник Дагестанского государственного технического университета. Технические науки: сб. научн. трудов*, 26, 94-102. – Взято з https://cyberleninka.ru.
10. Винников, Ю.Л. & Ярмолюк, О.І. (2010). Будівельні властивості ґрунтоцементу за наявності у його складі органічних речовин. *Строительство, материаловедение, машиностроение. Серия: Инновационные технологии жизненного цикла объектов жилищно-гражданского, промышленного и транспортного назначения,* 56, 97-103. Взято з http://www.irbis-nbuv.gov.ua.
11. *Свойства компонентов навоза.* [Электронный ресурс]. Взято з http://myzooplanet.ru
12. ДБН В.2.1-10-2009. (2009). *Основи та фундаменти споруд. Основні положення проектування: Основи та фундаменти будинків та споруд*. Київ: Мінрегіонбуд України, Державне підприємство «Укрархбудінформ».
13. ДБН В.2.1-10-2009. (2011). *Основи та фундаменти споруд. Основні положення проектування. Зміни 1. Пальові фундаменти*. Київ: Мінрегіонбуд України, Державне підприємство «Укрархбудінформ».
14. Пашинський, В.А., Карпушин, С.О. & Пашинський, М.В. (2018). Методика визначення кліматичних навантажень у заданій географічній точці. *Вісник Одеської державної академії будівництва та архітектури : зб. наук. праць*., 71, 68-72. Взято з http://nbuv.gov.ua.
15. Дмитриева, Т.В. (2011). *Стабилизированные глинистые грунты КМА для дорожного строительства.* (Автореф. дисс. канд. техн. наук). Белгород.
16. Зоценко, М.Л., Винников, Ю.Л., Омельченко, П.М. & Суходуб, О.В. (2016). Досвід вирішення геотехнічних проблем при реконструкції будівель і споруд. *Вісник Одеської державної академії будівництва та архітектури : зб. наук. праць*. 61, 123-129. Взято з http://www.irbis-nbuv.gov.ua
17. Yoshizu, T. (2014). Developmen of excavating agitator in deep mixing soil method. *AIJ Journal of Technolodgy and Design*. 20(44), 25-28. Взято з https://www.researchgate.net.
18. СП 291.1325800.2017. (2017). *Конструкции грунтоцементные армированные, правила проектирования*. Москва: Минстрой России. Взято з: http://docs.cntd.ru/document/456081631.

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The high-rise building foundation with developed stylobate part design features using piles tests data

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The paper presents two structural solutions for the high-rise building pile foundation where stylobate and high-altitude parts are not separated by a contraction joint studying results. The main difference between these structural solutions is in the use of piles of different lengths: in the first version all piles have the same length (implemented in practice) and the second version (perspective) is use of different lengths of piles and their support various engineering-geological elements (EGE): the longer piles support by stronger EGE under the altitude part and the shorter piles support by weaker EGE under the stylobate part. In calculations of the adopted versions piles tests results and geodetic observations have been considered for the used calculation methods verification.

Keywords: bored pile, «base – foundation – above-foundation building part» system calculation, static jacking load test, stylobate

Особливості проектування фундаментів висотної будівлі з розвиненою стилобатною частиною з використанням даних випробувань паль

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Подано результати дослідження двох конструктивних рішень пальового фундаменту для висотної будівлі, в якої стилобат та висотна частина не розділені деформаційним швом. З’ясовано, що основна відмінність конструктивних рішень полягає у використанні різних довжин паль: перший варіант – усі палі однакової довжини (реалізований у натурі); другий варіант (перспективний) – використання різної довжини паль і їх обпирання на різні інженерно-геологічні елементи: під висотну частину – довші палі з обпиранням на міцніший ІГЕ, стилобатна частина – коротші палі з обпиранням на слабший ІГЕ. При розрахунку прийнятих варіантів ураховано результати випробування паль та дані геодезичних спостережень для верифікації використаних розрахункових методів. Результати розрахунку показали, що більш економічним та раціональним з точки зору роботи конструкцій виявився другий варіант. У середньому матеріалоємкість фундаментів та технологічних процесів зменшилася на 10%, а величина осадок і їх нерівномірність у 2,5 раза порівняно з першим варіантом фундаментів. Крім того, підтверджено можливість використання розробленої методики з визначення осадок будівлі з використанням даних випробувань паль. Розходження виміряних осадок будівлі та прогнозних розрахункових у середньому складало не більше 10%, що є цілком прийнятним для прогнозування осадок будівель на стадії проектування.

Ключові слова: буронабивна паля, розрахунок системи «основа – фундамент – надфундаментна частина будівлі», випробування статичним вдавлювальним навантаженням, стилобат.

Introduction. The high-rise buildings construction requires the stylobate part ensuring the comfortable stay of people and meeting their needs developed. As a rule, parking lots, engineering, social facilities and amenities etc. are arranged in stylobates. The situation is rather common when the architectural requirements to the internal space organization in the building stylobate and high-altitude parts cause the organizing impossibility a complete contraction joint for separating building altitudinal and stylobate parts. It is because the contraction joint arrangement requires paired columns installing, which leads building internal space deviation from the original version envisaged by an architect.

Therefore, a complicated task of the pile foundation designing for two building spaces high-altitude part including (with significant loads on the columns) and low-rise stylobate part (with light loads on the columns) arises before the designer. For this reason, to avoid significant internal forces due to nonuniform deformations in building structures occurrence it is necessary to construct a foundation where deformations non-uniformity between the stylobate and high-altitude parts do not limit value exceed.

Thus, high-rise buildings with a developed stylobate part foundations various structural solutions studying is relevant with regard to obtain the optimal economic and structural design.

Review of the latest research sources and publications. Different lengths piles using supported by various engineering-geological elements, that is, under the altitude part the longer piles supported by harder EGE are arranged and under the stylobate part the shorter piles supported by weaker EGE are arranged.

Such approaches implementation to the foundations design proved to be effective.

For example, in time of the underground parking arranging under the Goethe Square in Frankfurt/Main, piles of different lengths were used for the central column and the columns along the perimeter. The piles lengths under the central part were larger in twice as under the peripheral part and deformations non-uniformity was within the permissible limits [1].

Moreover, the same approach was used in tall buildings construction in Berlin, e.g. SONY-CENTER and Treptowers [2].

The authors justifying calculations carried out of such foundations considering computer modeling and design parameters monitoring during the construction process.

Definition of unsolved aspects of the problem. However, the settlements and their irregularities were forecasted without consideration the «load – settlement» dependence of the pile deformation during its testing by a static jacking load that enabled to approximate maximally the settlements value theoretical calculations to the received ones at the construction site.

The main difference between the static jacking loads tests carried out in Ukraine [3] and foreign countries [4 – 5] is in compliance with Ukrainian regulatory documents; according to them each load increment is held to the chosen level of settlements stabilization depending on the soil type at the base. Thus, the settlements values prognosis with action of one pile on another one considering (cluster effect) is more accurate when the test data carried out by the procedure used in Ukraine applying.

Problem statement. The versions of the building pile foundations structural solutions should be considered for not to exceed the recommended limit value of deformations non-uniformity in the building stylobate and high-altitude parts in order to avoid the occurrence of significant internal forces in their structural elements that enables to make economic decisions for the building structures.

Basic material and results. To solve the tasks set forth in the paper, by way of illustration, it is reasonable to take a multistory building (26 floors) with a stylobate part (4 floors) where the high-altitude and stylobate parts are not separated by a contraction joint. The building computer model is presented on Fig. 1.

As the building foundation, a pile foundation is considered. The bored piles were of an 820-mm diameter. Two types of pile foundation were adopted: 1) in the first type of a foundation the lengths of piles were the same (22.0 m) for the high-altitude and stylobate parts with their resting on the same engineering-geological element (EGE) (this type of foundations was implemented at the construction site); 2) in the second type the lengths of the piles were different (22.0 m and 34.50 m) and the shorter piles were arranged under the stylobate part and longer ones – under the multistory part. The piles were supported by the different EGEs. Such piles lengths were chosen by the EGEs with different deformation characteristics presence. At the same time these piles were tested. The «load – settlement» dependence for two piles is shown on Fig. 2. The experimental piles seating on the engineering-geological section is presented on Fig. 3.

Fig. 3 approves that Kiev Marl EGE-6 (which has general deformation module E=50 MPa) forms the base for the 22.0-m long piles, and the fine sand EGE-7 (E=69 MPa) is the base for piles of 34.5 m length.

The pile field plan (the first type) with the same piles lengths is shown at Fig. 4. The pile field consists of 449 piles and the piles concrete volume is 5235 m^3.

The pile field plan (the second type) with different lengths of piles is shown in Fig. 5. The pile field consists of 177 piles by 34.50 m length with the piles concrete volume 3236 m^3 and of 155 piles of 22.0 m length with the piles concrete volume 1807 m^3. Total piles concrete amount is 5043 m^3.

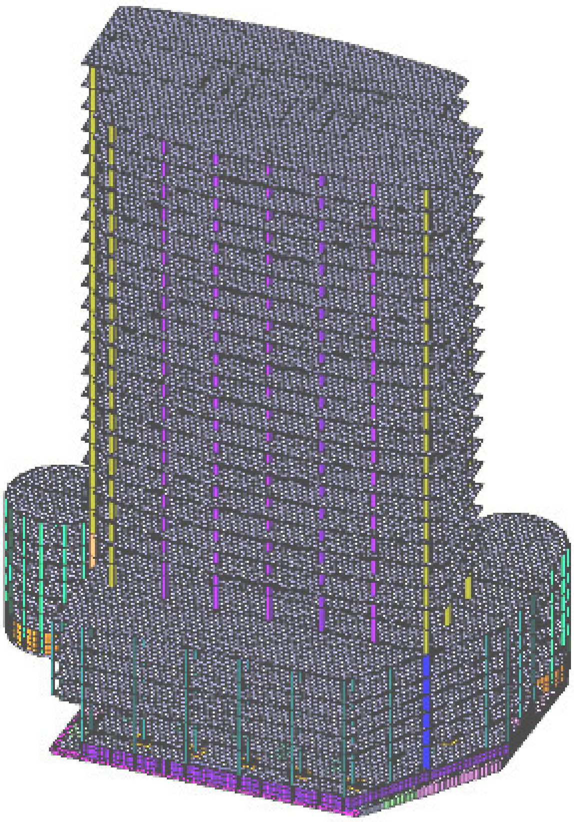

Figure 1 – Considered building computer model (piles are not shown)

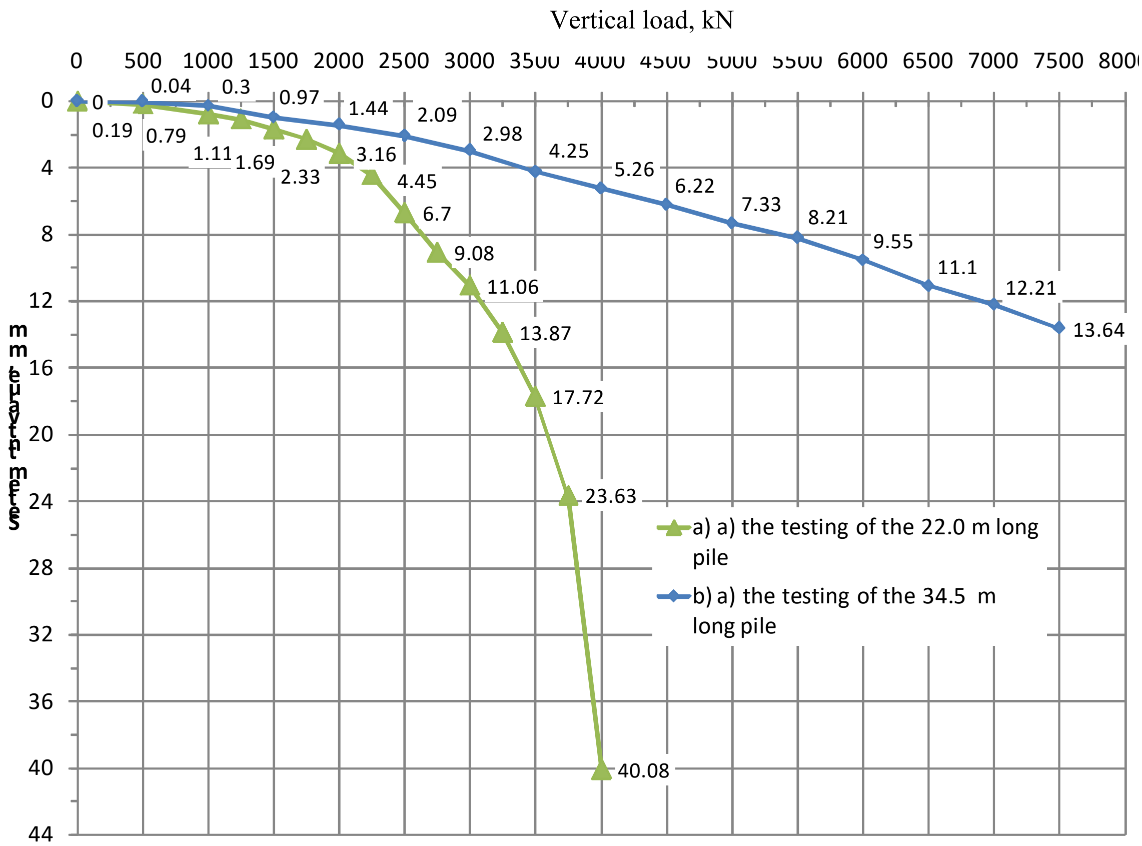

Figure 2 – Test of a pile with a diameter of 820 mm:
a) length of 22.0 m; b) length of 34.5 m

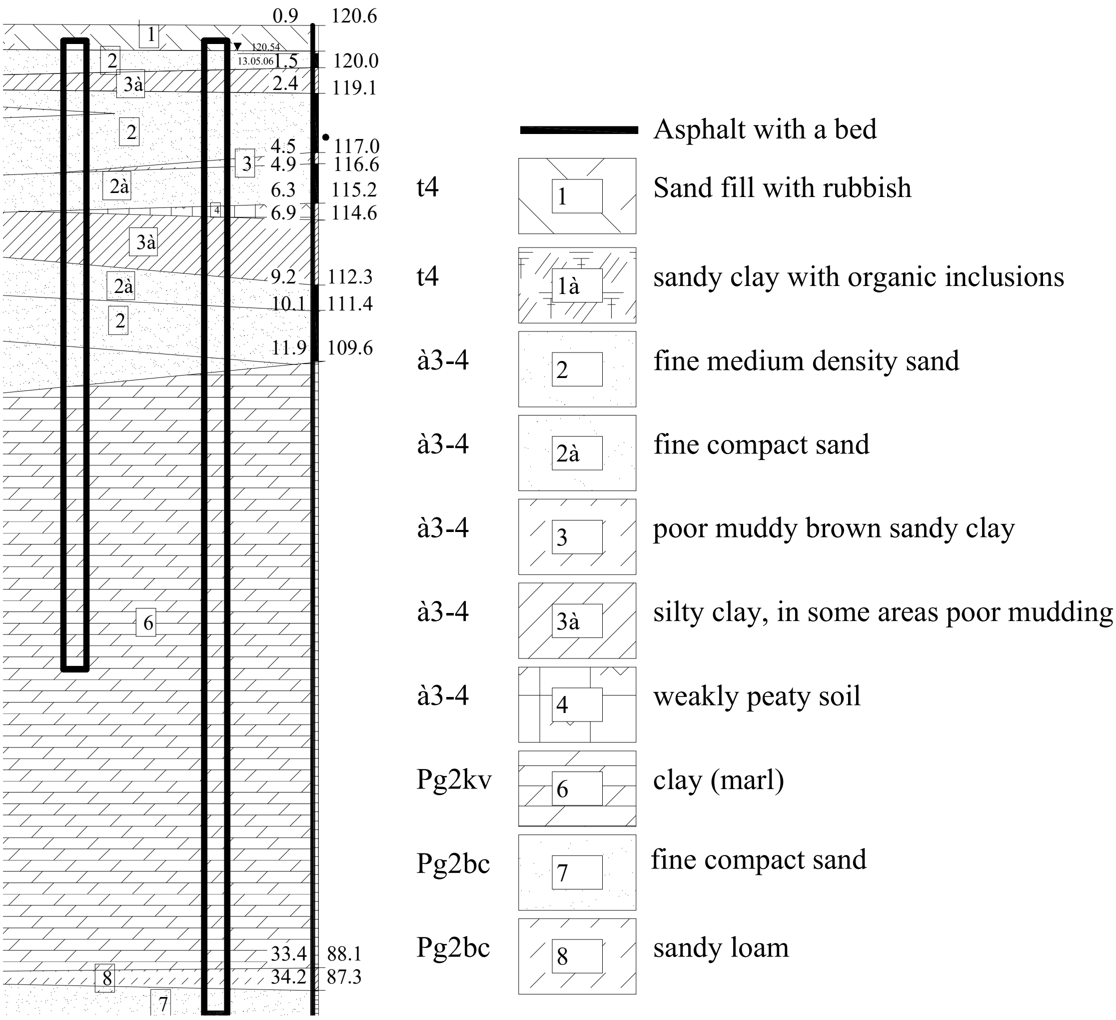

Figure 3 – Experimental pile spacing in the engineering-geological section

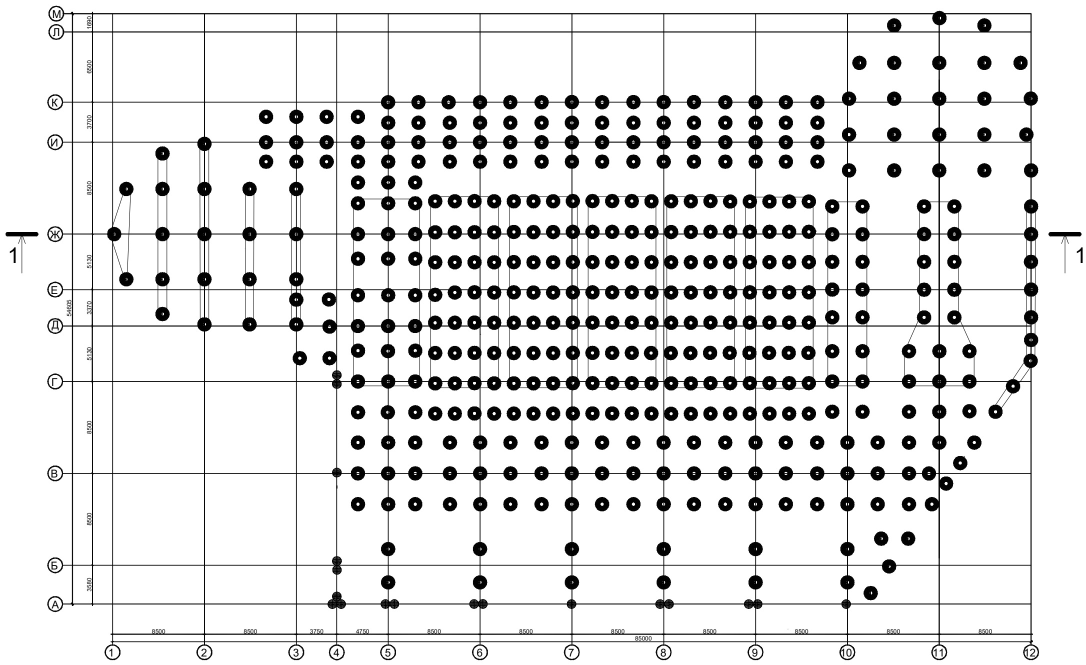

Figure 4 – Plan of the pile field with equal pile lengths – Type 1

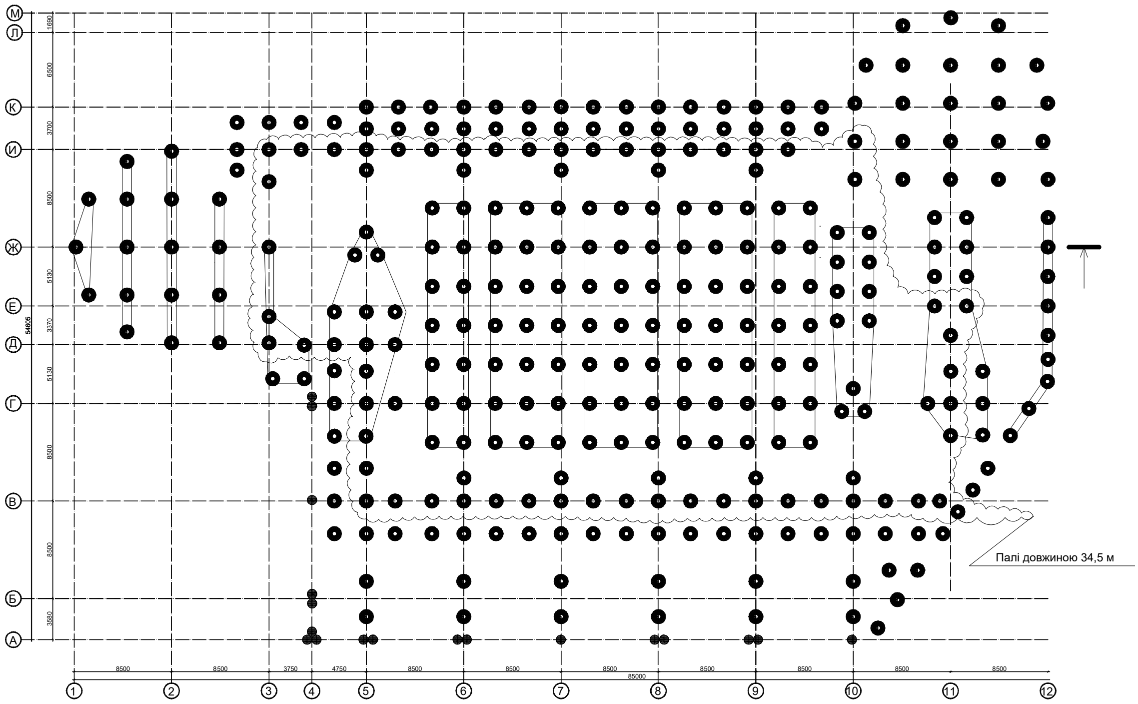

Figure 5 – Plan of the pile field with various pile lengths – Type 2

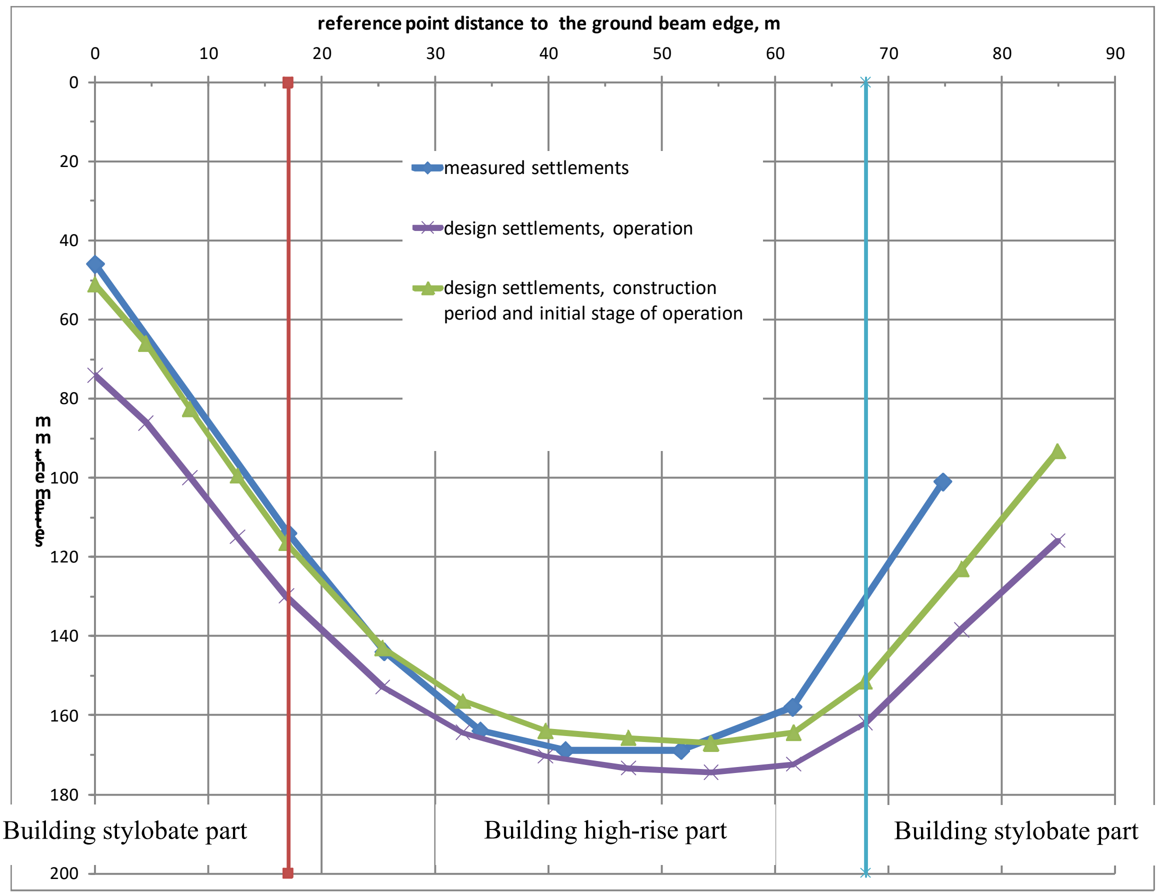

Figure 6 – Measured and calculated building settlement for foundation of type 1 implemented at the construction site

The pile foundations with piles of different lengths calculation using the methods presented in the regulatory document [6] is complicated by the fact that the «load – settlement» diagrams for the different lengths piles differ significantly (Fig. 2), and when presenting the surveys results of the total deformation modules values, as a rule, can be substantially underestimated. Therefore, for the calculation of the "base – foundation – above-foundation building part" system, the modified calculation procedure presented in works [7 and 8] is used. The calculation procedure details can be found in paper [6]. The procedure summary is as follows:

1) the carrying out of the «base – pile foundation – above-foundation building part» system first calculation (according to requirements [5]) considering the conditional foundation dimensions for each pile, which, according to the test data, are determined by the static jacking loading. The first calculation is performed with the same significant value of stiffness at the pile base where the extra forces from nonuniform deformations do not occur in the above-foundation structures. This calculation gives the loads on the piles $N_{II,i}$ and their settlements s_{ij}, with piles actions on each other being considered, which results in the bedding value C_{zi}^{K} for each pile, where index «K» means that the pile settlements are obtained with an allowance for the cluster effect. In addition, the result of the calculation is the correlation between the bedding values of each pile and other ones;

2) the building piles settlements determination without considering loads mutual influence received in the previous paragraph. As a result of this calculation, the bedding value C_{zi} is obtained for each pile;

3) the determination of the cluster effect value as the ratio $K=C_{zi}^{K}/C_{zi}$ for each foundation pile. In this case, for the deformations non-uniformity assessment it is possible to determine each pile settlement value by means of multiplying the settlement value received by the pile during load tests in accordance with paragraph 1 by the individual pile cluster effect value not considering the building stiffness (figures 7 and 8);

4) the cluster effect average value determination for all piles

$$K_m = \sum_{i=1}^{n} K_i \qquad (1)$$

where n is a number of piles, pieces;

5) the determination is based on the experimental pile "load - settlement" curve, the experimental pile settlement $s_{i,e}$ due to the loads received in accordance with para. 1 of this procedure;

6) the determination of the piles settlements average value $s_{m,e}$ obtained in accordance with para. 5 of the procedure,

7) the determination of the «base – pile foundation average value iteratively recalculated above-foundation building part» system settlement by means of the individual pile settlement average value $s_{m,e}$ multiplication according to para. 6 by the cluster effect value K_m obtained according to para. 4, that is

$$s_m^k = s_m \times K_m \qquad (2)$$

8) the iterative recalculation of the bedding values for the "base – pile foundation – above-foundation building part" system considering the average settlement value obtained according to para. 7 and the bedding values ratio obtained according to para. 1. As a result, the non-uniform settlements of the «base – pile foundation – above-foundation building part» system are obtained considering the stiffness of the above-foundation building part and the piles test with a static jacking load results.

An advantage of discussed procedure is that the cluster effect load and coefficient are known for each foundation pile, thus, it is possible to get the each pile settlement value that it receives without consideration the above-foundation structures stiffness, using the "load – settlement" dependence for the experimental pile. It enables to assess the deformations non-uniformity value.

The verification of the proposed procedure is performed on the first type foundation by comparing the measured and design settlements. At Fig. 6 the settlements values measured at the construction site (observation lasts over 10 years) and settlements design values calculated for two cases (the first case refers to the period of construction completion and the operation initial stage, the second case includes the operation period) are shown. The data are given for the section 1-1 (Fig. 4). Fig. 6 shows that on the average, the relative difference between measured and designed settlements values does not exceed 10% that indicates acceptable accuracy with regard to the prognosis of the settlement value using the proposed method. It should be noted that the better fit of the design settlement values was found in the calculations for the construction period and the operation initial period, rather than for the operation period. It is caused by the fact that the load between the piles of the stylobate and high-altitude parts was redistributed due to the cracks formation in the slab ground beam, but not because of the overloaded periphery piles settlement, provided that the ground beam operated without cracks.

Now the calculation results of two foundations types should be considered in more detail. At Fig. 7 the cluster effect values distributions in section 1-1 (figures 4 and 5) for foundations both types are shown. Fig. 7 shows that within the stylobate parts of two foundations types the cluster effect values differ not substantially and vary from 1.3 up to 8 with increase from the periphery to the center, which is quite logically explained by the piles interaction with decrease in the distances between them. At the same time, there is a difference in the building high-altitude part for two foundation types: the cluster effect is 10 ... 11.3 (mean value is 10.7) for the foundation of type 1 and 8 ... 9.9 (8.9) for the foundation of type 2. It also corresponds to the process physics, since the cluster effect decreases with the distance increase between piles.

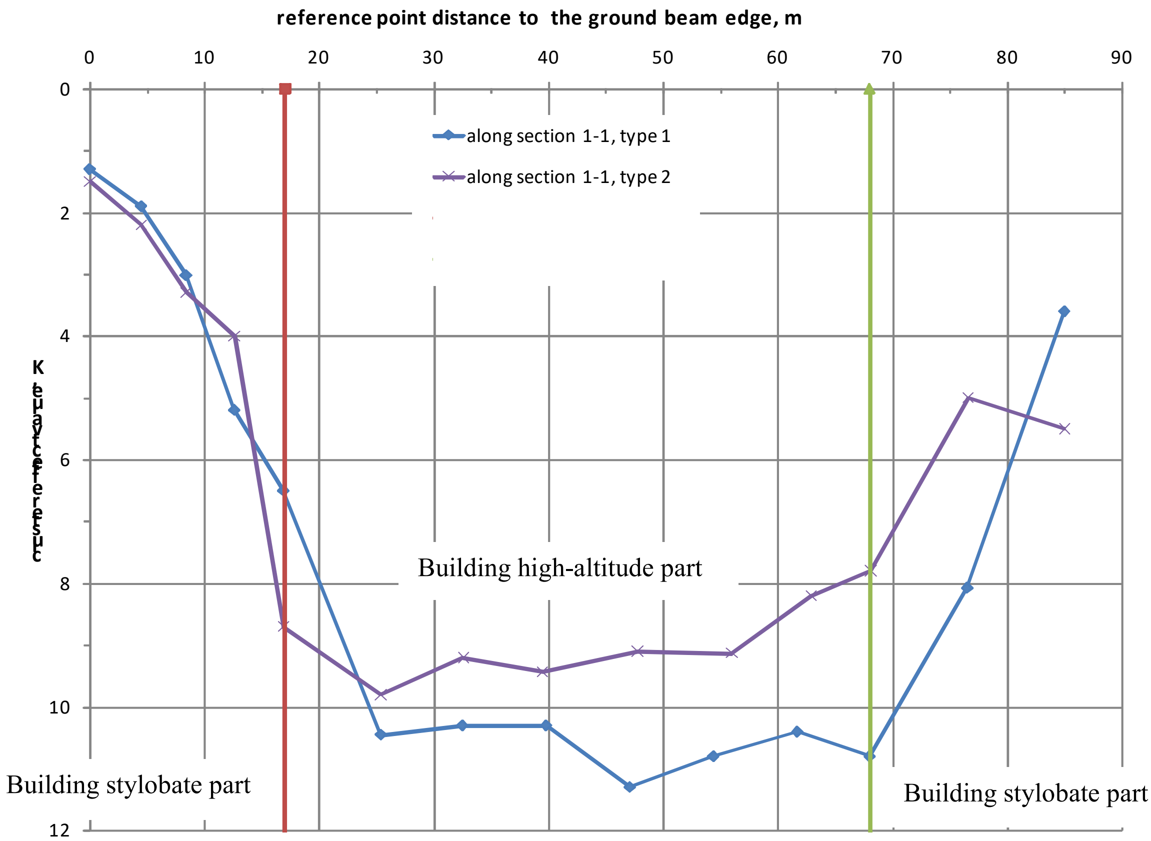

Figure 7 – Distribution of the cluster effect value along the larger side of the ground beam (in section 1-1, figures 4 and 5) for two types of foundations

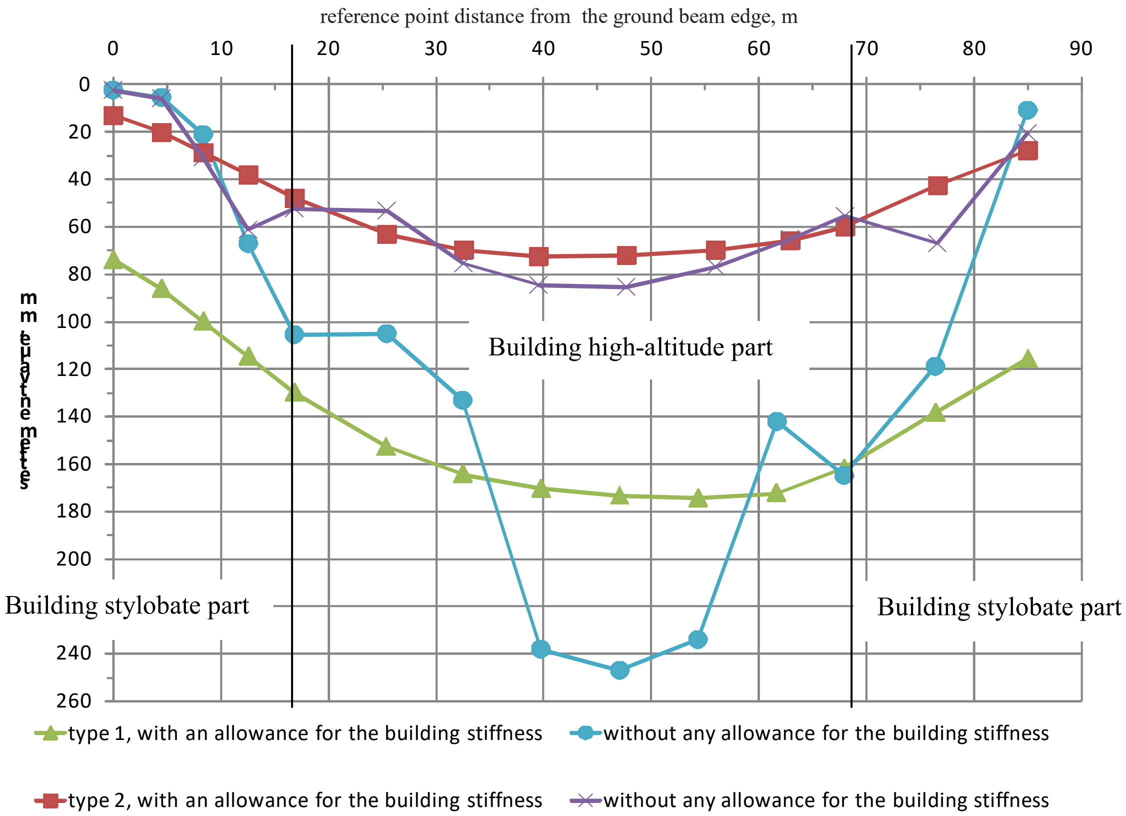

Figure 8 – Slab ground beam settlements for two types of foundations considering the above-foundation structures stiffness and without its consideration (in section 1-1, figures 4 and 5)

However, the data presented in Fig. 8 are more interesting. It is evident from Fig. 8 that the first type of foundation (all piles are the same) features the foundations significant irregularity between the stylobate and the high-altitude parts. Thus, according to the diagram without consideration the building stiffness, the stylobate part settlement varies within the range of 1.5 ... 100 mm, while the central part settlements are in the range of 140 ... 245 mm. This irregularity is caused by the fact that the loads on the central part piles were on average 3450 kN, but according to the test diagram (Fig. 2), in this range the pile works in a zone of nonlinear deformations. At the same time, within the stylobate part, where the pile works in the conditionally linear dependence of "load – settlement", the average load on the piles is 2500 kN (Fig. 2). With such a significant non-uniformity the significant force occur in the building part above the foundation. These forces redistribution can be carried out in two ways as follows:

1) if the building part reinforcement above the foundation is chosen considering the internal forces due to nonuniform deformations, the building redistributes the loads between piles without the cracks occurrence, which ensures its further normal operation. At the same time, more loaded piles are unloaded, but less loaded ones get more loading;

2) if the building part reinforcement above the foundation is selected without consideration the internal forces due to nonuniform deformations, then the building redistributes the loads between the piles with the cracks appearance (in fact, building is divided into separate blocks), and its further normal operation does not be ensured within the period of operation prescribed by the standards. The most dangerous in that case is that if the load on the piles (usually within the stair-lift block) exceeds pile based on the soil basis properties bearing capacity, then an emergency situation arises as the building subjects to the deformations of significant values.

A completely different picture is observed for the second foundations type (Fig. 8). Since the pile works in a conditionally linear phase throughout the entire load range (Fig. 3), the building does not undergo significant nonuniform deformations (Fig. 8), even considering the difference between the cluster effects in the stylobate and high-altitude parts (Fig. 7). So, it can be stated that the second type of foundations is more rational with regard to the structures above the foundation work and is more economical foundation. The comparison of two foundations types is presented in Table 1.

Table 1 – Comparison of two foundations types under the building with high-altitude and stylobate parts without a contraction joint

Parameter	Foundation of type 1	Foundation of type 2	Note
Average settlement of the high-altitude part, mm	150,0	70,0	
Average settlement of the stylobate part, mm	110,0	50,0	
Maximum non-uniformity of deformations	0,006	0,002	For the second type of a foundation the nonuniformity does not exceed the recommended value of 0.002
Values of bending moments in slab ground beam, kN x m	-4530…16600 ()	-2760…11000	The reduced range of force variation. For the second type of a foundation the enforcement necessity is 1.5 times less than for the first type of a foundation
Quantity of piles with a length of 22.0 m, pieces with a length of 34.5 m, pieces	449	155 177	
Piles concrete volume	5235	5043	For the second type of a foundation the pile concrete saving is 192 m^3
Total length of a pile borehole, r.m.	9878	9516	For the second type of a foundation the pile saving of a pile borehole drilling is 362 r.m.

Conclusions

The results of executed research enable to make the following conclusions:

1) the piles testing results use by the static jacking loading in accordance with the proposed procedure for the pile foundation settlements determination have showed a high convergence of the measured settlements values with the design ones. On average, the design settlements exceeded the measured ones by 10%, which proves a sufficient accuracy of calculations;

2) for high-rise buildings with a developed stylobate part, when the absence of a contraction joint between the stylobate and high-altitude parts is necessary, the use of foundation where under the high-altitude part the longer piles resting on the harder EGE and under the stylobate part the shorter piles being supported by the weaker EGE are arranged, is more rational with regard to the design and the structures performance. On average, there was the material consumption decrease for foundations and technological processes by 10%, and settlements values and their irregularity by 2.5 times compared with the first version of the foundations.

References

1. Dunaievskyi, R.A. (2008). «TOP-DOWN» method of construction. *New technologies in construction*. 1(15). 53-56.

2. Katzenbach, R. (2006). *The experience of geotechnical design: Presentation*

3. DSTU B V.2.1-1-95. (1997). *Soils. Field tests methods by piles*. Kyiv: Minregionbud Ukraine.

4. ASTM D 1143/D 1143M-07. (2007). *Static Axial Compressive Load. Standard Test Methods for Deep Foundations Under Static Axial Compressive Load*. Barr Harbor Drive, PO Box, USA.

5. Sisa, T. (2014). Static load test analysis for 600 mm bored pile in stiff soil. *Proceding of the 23rd European Young Geotechnical Engineers Conference*. Barcelona.

6. DBN V.2.1-10-2009. (2011). *Amendment №1. Bases and foundations of buildings.* Kyiv: Minregionbud Ukraine.

7. Kovalskyy, R. (2016). The settlements determination during the calculation of the «base – pile foundation – building» system based on the calculation scheme of the conditional foundation taking into. account the data of piles tests. *Civil structures,* 83, 28-37.

8. Kovalskyy, R. (2014). The determination of the «base – pile foundation – building» system settlements taking into account the data of piles tests. *Svit Geotekhniky,* 4(44), 26-30.

Збірник наукових праць. Серія: Галузеве машинобудування, будівництво

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Jet and jet-mixing grouting

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Analysis of ground cement elements production technologies by jet, jet mixing and mixing technologies with the consumption of various quantities of cement needed for their manufacture is carried out, their applicability is determined. It has been defined that design of bases and building bases grouted by ground-cement elements on weak and subsidence soils requires scientific approach. Mixing and jet-mixing technologies are more cost-effective ones and do not require pulp utilization; the material (ground cement) obtained during soils jet grouting for determining change in its characteristics in time and under the influence of various factors should be studied.

Keywords: ground cement, ground cement elements, cementation, fixing solutions, building bases, technologies

Струминне та струминно-змішувальне закреплення ґрунтів

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Приведено аналіз технологій виготовлення ґрунтоцементних елементів за струминною, струмино-змішувальною та змішувальною технологіями з витратою різної кількісті цементу на їх виготовлення, визначено їх можливості використання. Встановлено, що проектування основ і фундаментів підсилених ґрунтоцементними елементами на слабких и просадочних ґрунтах потребує наукового підходу. Основною відмінністю відомих технологій виготовлення ґрунтоцементу є спосіб змішування ґрунту з закріплює розчином і тиск подачі розчину. Спосіб струменевої цементації грунтів, що заснований на здатності високонапірного струменю руйнувати закріплювальним розчином ґрунти досить великий міцності, утворюючи при цьому пульпу, що складається з ґрунту геологічного розрізу і закріплювального розчину, який подається з сопла малого діаметру під тиском 45-60 МПа. Роботу з руйнування ґрунту і його перемішування виконує високонапірний струмінь. Дуже близькі за способом виготовлення грунтоцементу бурозмішувальна і струменева-змішувальна технології. Бурозмішувальна технологія передбачає руйнування ґрунту бурозмішувальним долотом і змішування зруйнованого ґрунту з закріплювальним розчином механічним способом, який є в'язким композитом. При струменево-змішувальній технології додатково до механічного перемішування, виконується гідравлічне перемішування вже зруйнованого механічно ґрунту струменем закріплювального розчину під тиском в 0,2-0,4 МПа. Змішувальна і струмино -змішувальна технології більш економічні за витратою матеріалів і не вимагають утилізації пульпи; потрібні дослідження матеріалу (грунтоцементу), одержуваного при струменевому закріпленні ґрунтів для визначення зміни його характеристик в часі і при впливі різних факторів

Ключові слова: ґрунтоцемент, ґрунтоцементні елементи, цементація, закріплюючі розчини, фундаменти, технології.

Introduction. The regulatory document «Change 2 to DBN B.2.1-10-2009 (base and building bases)» gives design wider opportunities, using efficient, reliable and economical technologies for foundations arrangement. Soil cement is increasingly used as the building basis and in some cases as a base, which gives significant economic effect to construction, and buildings long-term exploitation when at their construction soil cement was used, confirms this technology reliability. The analysis of recent research and publications, containing the solution of this problem is given in a number of works. The technology of soil cement manufacture for different technologies and its use are described in the works [2 – 6].

The main difference between the known technologies of soil cement manufacturing is the method of mixing the soil with fixing solution and the pressure of solution injection The method of soils ink cementation is based on the ability of high-pressure jet to destroy the soil fixing solution of quite high strength, while forming a pulp consists of geological section soil and fixing solution, which is injected from a small diameter nozzle under pressure of 45-60 MPa.

The work on the soil destruction and its mixing is performed with a high-pressure jet. Blasting and jet mixing technology are very similar to the method of soil cement manufacturing. Mixing technology involves soil destruction with a mixing bit and the mixing of damaged soil with a fixing solution by a mechanical method, which is a viscous composite. Under jet mixing technology, in addition to mechanical stirring, the hydraulic mixing of the mechanically ground with a jet of fixing solution under pressure at 0.2-0.4 MPa is carried out.

Actual scientific researches and issues analysis. Numerous studies of conducting the work on the soil cement manufacturing by various technologies and in various engineering-geological conditions enable determining their capabilities and explicabilities. The main parameters of construction work performing are the speed of their execution, quality, safety, reliability and cost. At the same time, the change in characteristics of soil cement strength, manufactured by different technologies, is studied insignificantly. In general these works are performed at Poltava Technical Yuri Kondratyuk University, under the supervision of ScD, Professor N.L. Zotsenko [7]. It is noted there, that soil cement, manufactured by mixing technology has a strength which increases with time. Information on the the material obtained during soil fixation flow is scarce.

Setting objectives. to analyze possibilities of ground cement elements manufacturing technology by different means are analysed, soil cement changes with time are evaluated, their applicability is determined.

The main material and results. It is important for a designer to determine possibility of applying a certain technology to prepare basis of a specific section at a particular engineering geological structure. It is known that under complicated engineering and geological conditions, the value of the buildings foundations may amount to 25% or more of all construction.

The given article is not aimed at analyzing of what is better – the piles, which will cut the weak part of the base, or reinforcing this part of the base with a ground cement; we suggest leaving it for further objective research, and in turn we will perform a comparison of technologies for soil cement production. Thus, according to the reports and publications, we will analyze consumption of cement as the main component of fixing solution.

It is noted in [3] that consumption of cement is equal to the amount from 200 to 270 kg per 1 m^3 of soil cement while manufacturing soil cement by jet mixing technology. In such a case, the excess of soil cement pulp is from 0.1% to 0.5%, depending on the length of the cement element and the geological structure.

Under mixing technology [7] cement consumption is from 170 to 200 kg per 1 m^3 of soil cement, with an excess of pulp from 0.1% to 0.5%. As can be seen, these technologies are very close to materials cost. The consumption of cement under the jet technology is equal to the amount [8] from 800 to 1000 kg per 1 m^3 of soil cement. The excess of technological slurry is from 30 to 70%.

While performing works on soil cement elements manufacturing in sections made of subsidence soils, and especially in the built-up areas, it is necessary to exclude the possibility of such soils soaking. Soaking of these soils will have a negative impact on the adjacent building, and in some cases, catastrophic effects.

A series of experiments [3] was carried out to determine the influence distance of the cement element manufacturing by jet-mixing technology on adjacent soils, the purpose of which included determining the radius of forest soils humidification while manufacturing soil cement elements by the given technology. To perform this, the pit was carried out, the walls of which were secured by the jet mixing technology, and soil samples were taken from the walls to determine the change in the soil humidity for laboratory studies. It is known that forest soil, due to its structure, is better for pure water filtering, and impurities that are there in aqueous solution are delayed. In this case, it can be said that the distribution of water occurs at a greater distance than the solution. According to these data, it is established that increasing in humidity of cement element adjacent to the soil at a distance of 10 cm is nowhere in evidence.

Carrying out that and other field number and laboratory studies enable to arrange disconnecting screens from ground-cement elements in various engineering-geological conditions Fig. 1.

Figure 1 – Protective screen from ground cement elements

Such screens are used to exclude the influence of the building base in the newly erected building on the nearby shallow laying building bases in existing buildings.

Very close data are also obtained in the manufacture of soil cement elements by mixing technology. It indicates soil cement elements safe placement possibility on the above-mentioned technologies in forest soils.

In the paper [4] it is noted that when arranging the cement constructions in their manufacture by the jet technology in conditions of heterogeneous layering weak water-saturated dust-clay soils, strength soil cement strength value on compression practically does not depend on the type of soil, since most of the soil is placed on the surface in the composition of the pulp; the aggregates of the soil lower layer, when exiting the surface, together with the pulp, are mixed with the working balls set up above, and thus the body of the pile turns out to be practically homogeneous in composition of the height of the palm), which at one time was marked by A.G. Malinin [6]. In fact, the strength of the soil cement turns out to be approximately equal to the strength of the cement stone for a given water-cement ratio.

In addition, the results determining the change in the specific resistance to immersion of the cone probe at different distances from the manufactured cement-based element are given here. The criterion for assessing the changes in soil properties was ground soils static sounding results, performed before the work start on the fence installation and building base excavation pit.

According the paper, the resistance of the cone immersion varies greatly, and at the distance of 2.5D from the edge of the soil cement element strength adjoining soil increases by an average of 20-30%, which is a sign of the penetration solution at a considerable distance. In general, the diameter of the cement-based elements, which are made by the jet technology, is about 1000 mm. If the strength of the adjoining soils increases at a distance of 2500 mm, then the soaking occurs at a much greater distance. In forest soils, soaking up entails sedimentation. It must necessarily be taken into account when designing grouting base. Avoid it, due to manufacturing technology peculiarities of ground cement elements by the jet technology, it is impossible.

In addition, as noted above, there is a significant removal of the pulp, consisting of soil and cement mortar, and requires utilization. The amount of pulp, depending on the type of soil, is from 30 to 70% cement element amount.

As for all fixing solutions preparation using cement, it is necessary to understand the basic processes occurring in solutions preparation and solidification processes.

In the research course, which determined the optimal composition of the cement amount in the soil cement, samples were made, where the cement was 90-95%. The samples were exposed to weather conditions. When viewed, they were once again discovered that their destruction was occurring, as can be seen in Fig. 2, 3.

Consider the processes that occur when preparing a cement solution.

After adding to the water cement, a solution is formed which is supersaturated with respect to calcium hydroxide and contains ions:

Ca2 +, SO42-, OH-, Na +, K + (1)

From the solution, hydrosulfate acuminate and calcium hydroxide are precipitated as primary tumors. At this stage, the system is not strengthened; the hydration of minerals is like a hidden nature. The second period of hydration (tussing) begins in about an hour with the formation of initially very thin crystals of calcium hydro silicates.

Figure 2 – Material sample fragment made by jet technology

Figure 3 – Material bundle under weather conditions influence

Hydro silicates and calcium hydrosulfate aluminates grow in long fibers form that penetrates the liquid phase in the form of bridges that fill the pores. A porous matrix is formed, which is gradually strengthened and filled with products of hydration. As a result, the mobility of solid particles is reduced and the cement paste is gripped. It is the first high-porosity low-strength structure that determines the tensile strength, consisting mainly of products interacting with water and gypsum. During the third period (hardening) pores are gradually filled with clinker minerals hydration products, there is a consolidation and strengthening of cement stone structure, as calcium hydro silicates increasing quantity formation result.

In the final form, the cement stone is an inhomogeneous system a complex conglomerate of crystalline and colloidal hydrates of formations, not activated residues of cement grains, finely divided water and air. It is sometimes called micro concrete.

The structural and mechanical properties of the cement dough increase as hydration of cement. For example, the maximum stress of the shift of the cement dough, according to E.E. Segalov. Measured after its manufacture, amounted to 0.01 MPa; before the beginning of the seizure, it increased to 0.15 MPa (i.e., 15 times), and by the end of the decade it reached 0.5 MPa (increased 50 times). Consequently, the cement paste differs with the ability to change quickly the rheological properties within 1 - 2 hours.

Portland cement hardening process is very complicated and is not yet fully understood at the same time. There are two main hypotheses that explain the transition of liquid cement mortar into a solid state. The crystallization hypothesis, initiated by Le Chatelier, explains the ability of the liquid cement mortar to solidify and solidify by the fact that Portland cement clinker source minerals have significantly higher solubility than their connection with water.

In the final form, the cement stone represents an inhomogeneous system - a complex conglomerate of crystalline and colloidal hydrates of formations, cement grains non-activated residues, finely divided water and air. It is sometimes called micro concrete.

As can be seen from the samples, their cracking occurs due to the presence of non-activated cement grains, which, after the end of the solidification time, are reacting and the stresses that arise at the same time, lead to the bundle of the material.

From all of the above, the following **conclusions** are drawn:

– The design of building base and building base with soil cement elements on weak and subsidence soils requires a scientific approach;

– Strengthening of weak and subsidence soils by ground-cement elements by mixing and jet mixing technologies can be used to create bases for slab and separately located building base;

– Mixing and jet mixing technologies are applicable for strengthening the existing base and buildings base and eliminating dissolution occurrence;

– Mixing and jet mixing technologies are more cost-effective materials and do not require pulp use;

– Material (ground cement) study obtained during the jet grouting to determine the change in its characteristics in time and under the influence of various factors should be conducted.

Refernces

1. ДБН В.2.1-10-2009. (2009). *Основи та фундаменти споруд. Основні положення проектування. Зі змінами №1,2*. Київ: Мінрегіонбуд України, ДП ДНДІБК.

2. Зоценко, М.Л. (2008). Прогресивні методи підготовки основ та будівництва фундаментів. *Будівельні конструкції: міжвідомчий науково-технічний збірник,* 71, 245-253.

3. Крысан, В.И. (2004). Перспективнык направления применения технологий бурения в строительстве. *Науковий вісник Національного гірничого університету,* 5, 80-82.

4. Крысан, В.И. & Крысан В.В. (2014). *Использование грунтоцемнта в строительстве*. Материалы международной научно-технической конференции «Научно-технический прогресс в строительстве и архитектуре». Баку.

5. Ланько, С.В. (2013). *Влияние грунтоцементных конструкций на деформируемость ограждений котлованов в условиях городской застройки.* (Автореферат дис. канд. техн. наук). СПбГАСУ, Санкт-Петербург.

6. Малинин, А.Г., Жемчугов, А.А. & Гладков, И.Л. (2010). Определение физико-механических свойств грунтов в ходе натурных иследований. *Известия ТулГУ. Науки о земле,* 1, 325-330.

7. Зоценко, М.Л. & Винников, Ю.Л. (2007). До підсумків міжнародної науково-технічної конференції «Проблеми механіки грунтів і фундаментобудування в складних грунтових умовах». *Світ геотехніки,* 1, 30-31.

8. Ланько, С.В. (2012). Влияние технологии струйной цементации на механические свойства окружающего массива грунта. *Вестник гражданских инженеров*, 3(32), 159-163.

Збірник наукових праць. Серія: Галузеве машинобудування, будівництво
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Mathematical modelling of soil massifs strained-deformed state under soil water level decreasing

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The analytical solutions for the determination of vertical displacements at any point of single-layer and multilayer soil compositions under filtration water flow influence, saline solutions presence and filtration considering soil changing filtration and deformation characteristics have been obtained. The mathematical models of soil filtration and the stress-deformed state from water-saturated ground massifs and bases deformations forecast under internal volumetric forces influence (hydrodynamic forces of the filtration flow, changes in the soils own weight) have been developed and substantiated. Numerical solutions of the corresponding boundary filtration problems and SDS of soil in regions with time-varying curvilinear boundary have been obtained for these mathematical models. They have enabled to perform water-saturated soils and bases deformations forecast under the change of hydrogeological conditions and man-made factors effect.

Keywords: pressure, displacement, conformal mapping, hydrodynamic net

Математичне моделювання напружено-деформованого стану ґрунтових масивів при зниженні рівня ґрунтових вод

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Досліджено вплив зміни гідрогеологічних умов та дії техногенних факторів на деформації водонасичених ґрунтових масивів і основ. Наведено результати експериментальних досліджень впливу концентрації сольових розчинів на фільтраційні та деформаційні властивості ґрунту. Отримано емпіричні залежності у вигляді поліномів коефіцієнта фільтрації, модуля деформації й коефіцієнтів Ламе від концентрації сольових розчинів, які дозволили вдосконалити математичні моделі фільтрації та напружено-деформованого стану ґрунту з урахуванням нелінійних фільтраційних і деформаційних процесів, що відбуваються у ґрунтових масивах за наявності та фільтрації сольових розчинів. Отримано аналітичні розв’язки з визначення вертикальних зміщень у будь-якій точці одно-і багатошарових ґрунтових масивів при дії фільтраційного потоку води, наявності та фільтрації сольових розчинів з урахуванням змінних фільтраційних і деформаційних характеристик ґрунту. Розроблено й удосконалено математичні моделі фільтрації та напружено-деформованого стану ґрунту з прогнозування деформацій водонасичених ґрунтових масивів і основ при дії внутрішніх об’ємних сил (гідродинамічних сил фільтраційного потоку, зміни власної ваги ґрунту). Отримано чисельні розв’язки відповідних крайових задач фільтрації та напружено-деформованого стану ґрунту для плоских областей зі змінною в часі криволінійною межею. Наведено результати числового моделювання вертикальних зміщень ґрунтових водонасичених масивів і основ у процесі їх осушення, наявності водозабірних свердловин та водонапірних споруд.

Ключові слова: напір, переміщення, конформне відображення, гідродинамічна сітка

Introduction. The change in the hydrogeological conditions on Earth is taking place more and more rapidly. The reason for such changes is the development of the last thousand years and the excessive increase in the beginning of the millennium of a new, previously unknown geological agent. This new geological agent is human activity and E.M. Sergeyev [1] considers this to be driving geological force, which not only changes earth surface, but also makes significant changes in the upper part of earth crust, which for scales and consequences coincide with geological processes.

Hydro geological conditions change in soils and presence of man-made factors lead to the formation of hydrodynamic forces of the filtration flow, changes in the soil own gravity, etc.

These factors growth magnitude and intensity can be significantly changed, which leads to earth surface subsidence occurrence. These deformations complicate normal operation, and in some cases lead to accidents in buildings and structures and can bring significant economic damage.

An analysis of the latest sources of research publications. In the paper [2] the frequent earth surface surveying of was performed for the direct study of vertical relief displacements. These studies require a lot of time and considerable material costs. An alternative to this may be the mathematical modeling of soil surface vertical displacement in its drain process [3 – 5].

Existing methods for soil massifs deformations and bases working assessing in the conditions of varying groundwater levels and man-made factors effects have been developed for the case of stabilized groundwater level still do not fully reflect their state [6 – 10].

So, the study of this problem is relevant.

The problem solution is greatly simplified with the development of mathematical methods for water-saturated granular massifs stress-strained state modeling.

Setting objectives. The problem of lowering the groundwater level in the soil mass as a result of water pumping from horizontal drains should is considered (Fig. 1).

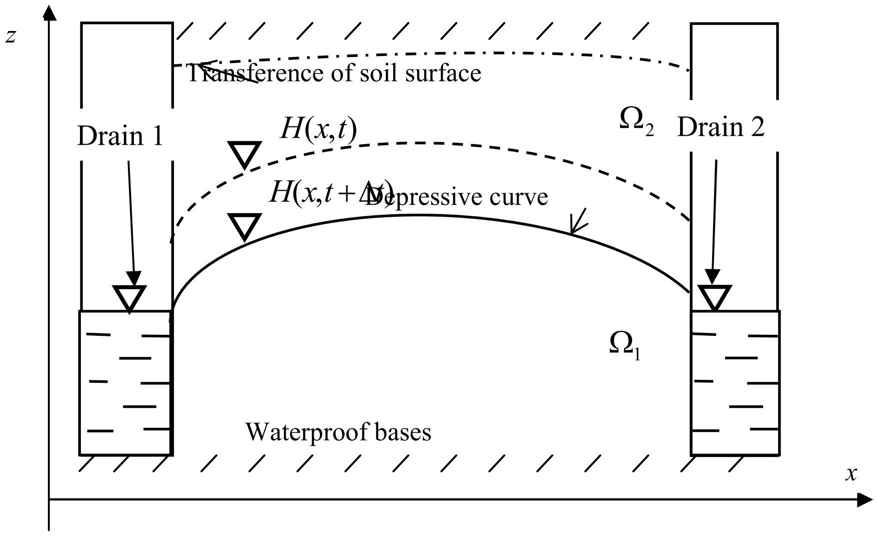

Figure 1 – Scheme of soil massif during its drainage

Due to the drainage of soil upper part and intensive filtration forces, the stress-deformed state of the soil massif changes and it leads to soil surface transference.

In order to find the transference of the soil surface, the pressure at all points of the soil massif at each time moment should be known.

For head determination in the changeable area $\Omega_1 = \{(x,z,t) \mid x \in (0,l), z \in (0,h(x,t)), t > 0\}$ solution of differentiated equation should be found:

$$\frac{\partial H}{\partial t} = \frac{kh_{col}}{\mu}\left(\frac{\partial^2 H}{\partial x^2} + \frac{\partial^2 H}{\partial z^2}\right) \quad (1)$$

under such boundary conditions:

$$H(x,z,0) = H_1(x,z), \quad (2)$$

$$H(0,z,t) = H_0(0) - V_0 t, \quad (3)$$

$$H(l,z,t) = H_0(l) - V_l t, \quad (4)$$

$$\frac{\partial H(x,0,t)}{\partial z} = 0, \quad (5)$$

$$H(x,h(x,t),t) = h(x,t), \quad (6)$$

where *H (x, z, t)* – water head at the time moment *t* in the point *(x, z)* of soil massif; *k* –filtration coefficient; h_{col} – capacity of filtration flow; μ –water drainage coefficient; $H_1(x, z)$ – division of water heads at the time moment; $H_0(x)$ – the height of soil water impending at the primary time moment (known function); *h (x, t)* – the height of soil water placement at the time moment *t*.

Symbols are introduced:

$$a^2 = \frac{kh_{col}}{\mu}. \qquad (7)$$

Then the equation (1) will look in the following way:

$$\frac{\partial H}{\partial t} = a^2\left(\frac{\partial^2 H}{\partial x^2} + \frac{\partial^2 H}{\partial z^2}\right). \qquad (8)$$

For the solution of this problem it is necessary to find *h (x, t)* та $H_1(x, z)$. That is why let's consider two additional problems.

Problem 1. In the area $\Omega = \{(x,t) \mid x \in (0,l), t > 0\}$ to find the solution of differential equation

$$\frac{\partial h}{\partial t} = \frac{kh_{col}}{\mu}\frac{\partial^2 h}{\partial x^2} \qquad (9)$$

under such boundary conditions:

$$h(x,0) = H_0(x), \qquad (10)$$

$$h(0,t) = H_0(0) - V_0 t, \qquad (11)$$

$$h(l,t) = H_0(l) - V_l t. \qquad (12)$$

Problem 2. In the area $\Omega = \{(x,z) \mid x \in (0,l), z \in (0, H_0(x))\}$ to find the solution of differential equation

$$\frac{\partial^2 H_1}{\partial x^2} + \frac{\partial^2 H_1}{\partial z^2} = 0 \qquad (13)$$

under such boundary conditions:

$$\frac{\partial H_1(x,0)}{\partial z} = 0, \qquad (14)$$

$$H_1(0,z) = H_0(0), \qquad (15)$$

$$H_1(l,z) = H_0(l), \qquad (16)$$

$$H_1(x, H_0(x)) = H_0(x). \qquad (17)$$

The solution of problem (9) – (12) is the following.

$$h(x,t) = \sum_{n=1}^{\infty}\left(A_n e^{-\left(\frac{\pi n a}{l}\right)^2 t} + \frac{2}{\pi n}(V_0 - V_l(-1)^n)\left(\frac{l}{\pi n a}\right)^2\left(1 - e^{-\left(\frac{\pi n a}{l}\right)^2 t}\right)\right)\sin\frac{\pi n x}{l} +$$
$$+ H_0(0) - V_0 t + \frac{H_0(l) - V_l t - (H_0(0) - V_0 t)}{l}x, \qquad (18)$$

where

$$A_n = \frac{2}{l}\int_0^l H_0(x)\sin\left(\frac{\pi n}{l}x\right)dx + \frac{2}{\pi n}(-H_0(0) + H_0(l)(-1)^n). \qquad (19)$$

Let's (13)–(17) fulfill the numerical conformal reflection of the area $\Omega = \{(x,z) \mid x \in (0,l), z \in (0, H_0(x))\}$ on the parametric rectangle for the solution of the problem. The built net of conformal reflection is the hydrodynamic net, that is the solution of problem. The meanings of heads on the lines of equal heads are equal with the meanings in the upper points of these lines, which in their turn, are equal to the vertical coordinate of soil water surface.

For the solution of problems (1) – (8) it is transfered to variables ξ, η [11], under this

$$\xi = \xi(x,z), \ \eta = \eta(x,z). \qquad (20)$$

Under the conditions of Koshi-Riman it is:

$$\frac{\partial \xi}{\partial x} = \frac{\partial \eta}{\partial z}, \ \frac{\partial \xi}{\partial z} = -\frac{\partial \eta}{\partial x}. \qquad (21)$$

Substituting (20) in (8), we have:

$$\frac{\partial H}{\partial t} = a^2\left(\frac{\partial^2 H}{\partial \xi^2}\left(\frac{\partial \xi}{\partial x}\right)^2 + \frac{\partial H}{\partial \xi}\frac{\partial^2 \xi}{\partial x^2} + \frac{\partial^2 H}{\partial \eta^2}\left(\frac{\partial \eta}{\partial x}\right)^2 + \frac{\partial H}{\partial \eta}\frac{\partial^2 \eta}{\partial x^2} + \frac{\partial^2 H}{\partial \xi^2}\left(\frac{\partial \xi}{\partial z}\right)^2 + \frac{\partial H}{\partial \xi}\frac{\partial^2 \xi}{\partial z^2} + \frac{\partial^2 H}{\partial \eta^2}\left(\frac{\partial \eta}{\partial z}\right)^2 + \frac{\partial H}{\partial \eta}\frac{\partial^2 \eta}{\partial z^2}\right). \qquad (22)$$

Taking into consideration, that $\xi(x,z)$ and $\eta(x,z)$ – harmonic functions, we have:

$$\frac{\partial H}{\partial t} = a^2\left(\frac{\partial^2 H}{\partial \xi^2}\left(\left(\frac{\partial \xi}{\partial x}\right)^2 + \left(\frac{\partial \xi}{\partial z}\right)^2\right) + \frac{\partial^2 H}{\partial \eta^2}\left(\left(\frac{\partial \eta}{\partial x}\right)^2 + \left(\frac{\partial \eta}{\partial z}\right)^2\right)\right) \qquad (23)$$

Taking consideration (21), (23) will be in the following way

$$\frac{\partial H}{\partial t} = a^2\left(\left(\frac{\partial \xi}{\partial x}\right)^2 + \left(\frac{\partial \xi}{\partial z}\right)^2\right)\left(\frac{\partial^2 H}{\partial \xi^2} + \frac{\partial^2 H}{\partial \eta^2}\right). \qquad (24)$$

In this case, a grid constructed with a numerical conformally represented net will be a hydrodynamic grid, that is, the solution of the problem (1) – (8). The solution is the same as solving the problem (13) – (17). The obtained exact formulae are used for ($h(x, t)$ (for calculating the replacement of a series sum by a finite sum, the integration with a parabola method is used. The advantage of this approach is that pressure and movement at any given time can be found without finding them at previous moments of time (only at the initial moment of time).

For finding transferences in the areas below soil water level Ω_1 and above Ω_2, we have the system of differential equations:

$$k_1 \frac{d^2 u_1}{dz^2} = \gamma_{sb} + \gamma_w \left(\frac{dh}{dz} + 1 \right), x \in (0, l_1), \quad (25)$$

$$k_2 \frac{d^2 u_2}{dz^2} = \gamma_n, x \in (l_1, l), \quad (26)$$

$$u_1(0) = 0, \quad (27)$$

$$\frac{du_2(l)}{dz} = 0, \quad (28)$$

$$u_1(l_1) = u_2(l_1), \quad (29)$$

$$k_1 \frac{du_1(l_1)}{dz} = k_2 \frac{du_2(l_1)}{dz}, \quad (30)$$

where γ_{sb}, γ_w, γ_n – specific soil weigh, that is located in weighted state, water and soil, which is in natural state relatively;

$h(z)$ – head (x_1, z) at the time moment t_1;

$k_1 = \lambda_1 + 2\mu_1$, $k_2 = \lambda_2 + 2\mu_2$ – elastic steels;

$u_1(z)$, $u_2(z)$ – transference of point, which is in the moment t_1 located in the point (x_1, z);

l_1 – soil water level, and l – determined vertical coordinate of the upper point of soil's massif in the point $x = x_1$ at the time moment $t = t_1$;

indexes 1, 2 near k, λ, μ, u mean the location of points (x_1, z) below or above soil water level relatively.

Integrating equations, it is got:

$$\frac{du_1}{dz} = \frac{\gamma_{sb} z}{k_1} + \frac{\gamma_w}{k_1}\left(h(z) + z\right) + c_1,$$

$$u_1 = \frac{\gamma_{sb} z^2}{2k_1} + \frac{\gamma_w}{k_1}\left(\int_0^z h(\xi) d\xi + \frac{z^2}{2} \right) + c_1 z + c_2,$$

$$\frac{du_2}{dz} = \frac{\gamma_n z}{k_2} + c_3,$$

$$u_2 = \frac{\gamma_n z^2}{2k_2} + c_3 z + c_4,$$

$$u_1(0) = 0 \Rightarrow c_2 = 0,$$

$$\frac{du_2(l)}{dz} = 0 \Rightarrow \frac{\gamma_n l}{k_2} + c_3 = 0 \Rightarrow c3 = -\frac{\gamma_n l}{k_2},$$

$$k_1 \frac{du_1(l_1)}{dz} = k_2 \frac{du_2(l_1)}{dz} \Rightarrow \gamma_{sb} l_1 + \gamma_w \left(h(l_1) + l_1\right) + c_1 k_1 = \\ = \gamma_n l_1 - \gamma_n l.$$

Considering $h(l_1) = l_1$ it is got:

$$c_1 = \frac{1}{k_1}\left(\gamma_n (l_1 - l) - 2\gamma_w l_1 - \gamma_{sb} l_1\right),$$

$$u_1(l_1) = u_2(l_1) \Rightarrow \frac{\gamma_{sb} l_1^2}{2k_1} + \frac{\gamma_w}{k_1}\left(\int_0^{l_1} h(\xi) d\xi + \frac{l_1^2}{2} \right) + c_1 l_1 = \frac{\gamma_n l_1^2}{2k_2} - \frac{\gamma_n l_1 l}{k_2} + c_4 \Rightarrow$$

$$c_4 = \frac{\gamma_{sb} l_1^2}{2k_1} + \frac{\gamma_w}{k_1}\left(\int_0^{l_1} h(\xi) d\xi + \frac{l_1^2}{2} \right) + \frac{l_1}{k_1}\left(\gamma_n (l_1 - l) - 2\gamma_w l_1 - \gamma_{sb} l_1\right) - \frac{\gamma_n l_1^2}{2k_2} + \frac{\gamma_n l_1 l}{k_2} =$$
$$= \frac{1}{k_1}\left(\gamma_w \int_0^{l_1} h(\xi) d\xi - \frac{3\gamma_w l_1^2}{2} - \frac{\gamma_{sb} l_1^2}{2} + \gamma_n \left(l_1^2 - l_1 l\right) \right) + \frac{\gamma_n l_1}{k_2}\left(l - \frac{l_1}{2} \right),$$

So far as we take into consideration all risk factors on soil, transference is located relatively some primary level. This starting point, which we mark l_0, is necessary to be found, using known at the primary moment of meaning l.

$$u_2(l) = l_0 - l, \quad (31)$$

$$u_2(l) = \frac{\gamma_n l^2}{2k_2} - \frac{\gamma_n l^2}{k_2} + c_4 = -\frac{\gamma_n l^2}{2k_2} + c_4.$$

Substituting $u_2(l) = l_0 - l$, found meaning c_4 it is got:

$$l_0 = l - \frac{\gamma_n l^2}{2k_2} + \frac{1}{k_1}\left(\gamma_w \int_0^{l_1} h(\xi) d\xi - \frac{3\gamma_w l_1^2}{2} - \frac{\gamma_{sb} l_1^2}{2} + \gamma_n \left(l_1^2 - l_1 l\right) \right) + \frac{\gamma_n l_1 l}{k_2} - \frac{\gamma_n l_1^2}{2k_2}. \quad (32)$$

Substituting known values in the obtained formula at the primary time moment, we will find l_0.

For finding l at any time moment from (32) we will get quadratic equation:

$$\left(\frac{\gamma_n}{2k_2}\right)l^2+\left(\frac{\gamma_n l_1}{k_1}-1-\frac{\gamma_n l_1}{k_2}\right)l+\left(l_0+\frac{\gamma_n l_1^2}{2k_2}+\frac{1}{k_1}\left(-\gamma_w\int_0^{l_1}h(\xi)d\xi+\frac{3\gamma_w l_1^2}{2}+\frac{\gamma_{sb}l_1^2}{2}-\gamma_n l_1^2\right)\right)=0. \quad (33)$$

Having solved the equation (33), we will find the coordinate of the soil upper boundary for arbitrary x and t:

$$l=\frac{\left(1+\gamma_n l_1\left(\frac{1}{k_2}-\frac{1}{k_1}\right)+\sqrt{\left(1+\gamma_n l_1\left(\frac{1}{k_2}-\frac{1}{k_1}\right)\right)^2-\left(\frac{2\gamma_n}{k_2}\right)\left(l_0+\frac{\gamma_n l_1^2}{2k_2}+\frac{1}{k_1}\left(-\gamma_w\int_0^{l_1}h(\xi)d\xi+\frac{3\gamma_w l_1^2}{2}+\frac{\gamma_{sb}l_1^2}{2}-\gamma_n l_1^2\right)\right)}\right)k_2}{\gamma_n}. \quad (34)$$

It has been chosen the sign «+» before square root taking into consideration, that $l > 0$

For finding integral from the function of head

$$\int_0^{l_1}h(\xi)d\xi$$

built hydrodynamic net is used. Geometric algorithm of integral finding in the point $x = x_1$ at time moment $t = t_1$: on the hydrodynamic net for time next. It is determine the straight $x = x_1$. At points of this line intersection with lines of hydrodynamic grid currents, head meaning is found with the help of linear interpolation between adjacent nodes. Integral is found using the formulae of the central rectangles for unevenly located nodes. This algorithm is simply implemented in the programming language using analytical geometry.

In accordance with described algorithm, numerical experiments have been carried out with the determination of vertical displacements under next output data:

$x = 100m$;

$H_0(z)\big|_{t=0}=35m$; $\quad H_r(z)\big|_{t=t_i}=23m$;

$\lambda_1=18493kPa$; $\quad \mu_1=15748kPa$;

$\lambda_2=18000kPa$; $\quad \mu_2=18000kPa$;

$\gamma_w=10kN/m^3$; $\quad \gamma_n=16{,}48kN/m^3$;

$\gamma_{sb}=9{,}63kN/m^3$; $\quad \gamma_{dr}=16{,}48\,kN/m^3$.

Given net step $\approx 3m$.

Here λ_1, μ_1 – Lame coefficients for soil in water saturated state coefficients, but λ_2, μ_2 – for drained soil.

Numerical modeling is performed under partial drainage of the soil massif. The results of the vertical displacements at given points of the soil massif for the case during drainage process studies are partly presented in table 1.

Table 1 – Meanings of soil massif vertical displacements

z, m \ x, m	12,5	35	53,5	66,0	81,5	935	100,0
39,9	-58,56	-58,66	-58,67	-58,64	-58,62	-58,63	-58,74
29,925	-55,53	-55,60	-55,59	-55,57	-55,54	-55,54	-55,61
19,95	-41,44	-41,44	-41,41	-41,36	-41,31	-41,28	-41,28
9,975	-26,28	-26,25	-26,21	-26,17	-26,13	-26,09	-26,06

Numerical meanings of displacements in given nodes, which are presented in the given tables, are measured in millimeters.

For visualization, the results of research in graphic form are presented (fig. 2). The upper part of the figure shows soil surface at the initial and at the final moment of time, in the lower part – a hydrodynamic net.

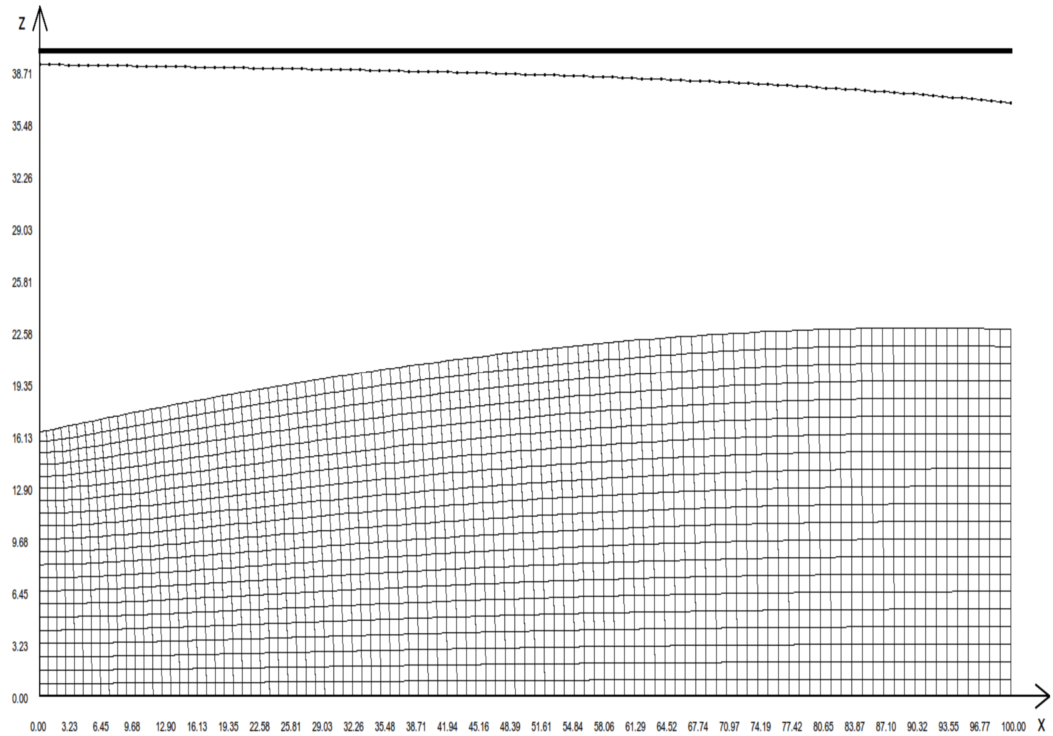

Figure 2 – Graphic representation of research results

Conclusions. In scientific articles only one of the many engineering problems in predicting the subsidence of the earth surface during the groundwater level lowering by pumping water from horizontal drains has been considered. One-dimensional mathematical model has been studied, which may only correspond to some practical problems. The simple solution obtained in this paper has the advantage of its speed and the possibility of finding the soil massif state at any given time, without finding at previous time moments.

The obtained mathematical solutions enables to fulfill the forecast of water-saturated ground masses subsidence during their drainage. Further areas of researches can be mathematical solutions obtaining of the set tasks for a two-dimensional case.

References

1. Сергеев, Е.М. (1978). *Инженерная геология*. Москва: Изд-во Моск. ун-та.
2. Остапчук, С.М. (1997). Кількісна оцінка зміни рельєфу на меліоративних землях. *Меліорація і водне господарство: міжвідомчий темат. наук. збірник*, 84, 125-131.
3. Кузло, М.Т. (2013). Моделювання деформацій природного схилу при його осушенні. *Збірник наукових праць. Серія: Галузеве машинобудування, будівництво,* 3(38)-1, 199-2007.
4. Martyniuk, P., Michuta, O., Ulianchuk-Martyniuk, V. & Kuzlo M. (2018). Numerical investigation of pressure head jump values on a thin inclusion in one-dimensional non-linear soil mousture transport problem. *International Journal of Applied Mathematics*, 31(4), 648-661. Retrieved from doi:10.12732/ijam.v31i4.10
5. Martyniuk, P., Kuzlo, M., Matus, S. & Tsvietkova, T. (2017). Mathematical model of nonisothermal moisture transference in the form of water and vapor in soils in the case of chemical internal erosion. *Far East Journal of Mathematical Sciences,* 102, 3211-3221. Retrieved from doi:10.17654/MS102123211.
6. Бойко, И.П., Лебеда, А.Ф. & Давыдюк, В.В. (1991). Влияние гидрогеологических условий на деформацию оснований существующих фундаментов. *Основания и фундаменты,* 24, 23-30.
7. Кушнер, С.Г. (2005). Повысить внимание к учету факторов, влияющих на осадку оснований зданий и сооружений. *Світ геотехніки*, 2, 16-21.
8. Моргун, А.С. (2003). Числове моделювання процесу взаємодії штампа з пружно-пластичним середовищем грунту за МГЕ. *Збірник наукових праць. Серія: Галузеве машинобудування, будівництво,* 12, 147-152.
9. Моргун, А.С., Андрухов, В.М. & Меть І.М. (2009). Вплив техногенного фактора замокання ґрунтової основи на НДС висотної будівлі. *Дороги і мости: Зб. наук. праць,* 3, 233-238.
10. Николаев, А.П. (2009). Деформации грунтового массива на участках строительного понижения напоров ратмировского горизонта в г. Москве. *Инженерная геология*, 2, 26-31.
11. Власюк, А.П. & Михальчук, В.Г. (1989). *Автоматическое построение конформных и квазиконформных отображений четырехугольных областей с помощью разностных сеток с «плавающими» узлами*. Киев: Препр. АН УССР. Ин-т математики.

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Overall stabilization of underground workings in limestone-shells

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The characteristic properties of limestone-shell used as the base of foundation are described. Using the example of multistorey residential building design, comprehensive solution has been proposed for plugging workings in an array of limestone-shell for the subsequent construction of slab-pile foundation, which includes engineering and geological surveys, geotechnical design, the choice of mortars and their compositions, as well as continuous monitoring plugging conditions. It is stated that two types of cement-sand mortar are used for plugging workings: the first is with the addition of superplasticizers, which provide high solution mobility, low water loss and non-shrinkage after hardening; the second is without adding superplasticizers with fluidity high degree. The results of laboratory studies to select the composition of soil-cement are given.

Keywords: limestone-shell, underground working (catacomb), plugging, injection, soil-cement, bearing capacity

Комплексне закріплення виробок у вапняку-черепашнику

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Описано характерні властивості вапняку-черепашнику, який використовують як основу фундаментів. На прикладі проектування багатоповерхового житлового будинку в м. Одесі запропоновано комплексне розв’язання задачі тампонування виробок (катакомб) у масиві вапняку-черепашнику для подальшого влаштування плитно-пальового фундаменту; воно включає в себе проведення робіт з інженерно-геологічних вишукувань, геотехнічного проектування, вибору розчинів і їх складів, а також виконання робіт з постійним моніторингом умов тампонування. Виробки розташовано на глибині близько 30 м від денної поверхні, вони складають понад 26% площі забудови будинку. Викладено, що для надійного тампонування виробок на першому етапі використовувалися два типи цементно-піщаних розчинів: перший – з додаванням суперпластифікаторів, що забезпечують високу рухливість розчину, низьку його водовіддачу та безусадочність після твердіння; другий – з високим ступенем розтікання. На першому етапі розчин подавався у свердловину як наливом, так і насосом, що дозволяв створювати тиск до 2 МПа; на другому етапі виконувалося ін’єктування розчину в зону виробок з використанням манжетних колон. Для ін’єкції використовувався цементний розчин з добавками-пластифікаторами, а на окремих ділянках – ґрунтоцемент. Наведено результати лабораторних досліджень для вибору складу ґрунтоцементу. При цьому встановлено, що раціонально приймати витрату суглинку як відходу виробництва і найдешевшого матеріалу в кількості 1 м^3 за насипною густиною на 1 м^3 ґрунтоцементу. Витрати цементу призначено у діапазоні 100 – 200 кг/м^3 розчину з урахуванням необхідності отримання марок розчину М25.

Ключові слова: вапняк-черепашник, підземна виробка (катакомба), тампонування, ін’єктування, ґрунтоцемент, несуча здатність

Introduction. Increase in the pace of construction and reduction in the number of construction sites with a relatively simple geological structure makes geotechnical engineers to develop areas with complex geological conditions, including those complicated by the presence of underground workings (the so-called «catacombs»).

Therefore, in the design and construction of high-rise buildings in terms of a dense urban development in the area of underground workings, the issue of accumulation and analysis of experience in building effective and reliable bases and foundations, as well as geotechnical monitoring of the construction of such objects, is relevant [1, 2].

Analysis of resent research and publications. The experience of foundations in the area of underground workings is given in [1 – 6].

From the analysis of these studies aimed at solving the problem of bored piles installation in areas of underground workings, the most popular is the technology of rib control by plugging the cavity with its subsequent piles construction [5, 6].

Identification of general problem parts unsolved before. The problem of rocks stratum possible forcing by the weight of a building, and methods for solving these problems, in particular the use of plugging workings in an array of limestone-shell for the subsequent construction of multi-storey buildings on slab-pile foundations, has not been adequately investigated in geotechnical design practice.

T**he aim of the work** is to develop an effective complex of works on overall stabilization of underground workings in an array of limestone-shell for the subsequent construction of multi-storey buildings on slab-pile foundations.

Basic material and results. At the construction site of a 24-storey residential building in Odessa, Gagarin Avenue, 19, workings are located at a depth of about 30 m from the surface and account for more than 26% of the built-up area of the building (Fig. 1, a).

At the site in question, the workings were previously tamped with sand filling by injecting water-sand pulp. The search work performed in the process of engineering-geological prospecting enabled clarifying the planned-altitudinal position of the workings and the quality of the plugging performed.

As a result, it was defined that the workings are not filled with sand to the full height and measures for their additional stabilization are required. The complexity of the plugging is that, due to insignificant height of the identified voids (0.4 ... 0.8 m), access to them through the vertical holes that have to be arranged beforehand, is difficult to implement, and the mine survey of workings has not been performed.

The project, considering geotechnical conditions complexity, provides for a plate-pile foundation of prismatic piles with a section of 35x35 cm, length 14.0 m, with their bottoms stop in the Strata-8 – light red-brown clay. The grating is designed as a solid monolithic slab, 1.8 m thick. Below the sole of the clay layer there limestone-shell array lies, in the sawn difference of which the underground workings are revealed (Fig. 1, b). Thus, to ensure the base operation reliability, it was necessary to fill all the voids in terms of the lack of their exact location and the quality of the filling that was previously performed

Work reliable tamping was provided by the gradual injection of mortars into the zone of their distribution. At the first stage, cavities connected both with each other and with previously drilled exploration wells were filled, which is shown in Fig. 1

To solve the problems, two types of cement-sand mortars were used, namely: the first type with the addition of superplasticizers, which provide high solution mobility, low water loss and non-shrinkage after hardening; the second one is without adding superplasticizers with a high degree of fluidity.

Depending on the voids height in each well and the identified production slopes in the interval between the wells, a sequence of filling them with a solution, alternation and injection volume of each type of mortars were developed. The mortar was fed both in bulk into the well, and with a pump that enabled to create pressure up to 2.0 MPa. The control of the cavities filling degree with the mortar was carried out with the help of a flap, which was lowered both into the filled and control wells.

At the second stage, the solution was injected into the area of workings using cuff columns (Fig. 2). In the plan the cuff columns were arranged in the interval between the wells for the injection of cement-sand mortar (first stage), the height of production in 80 cm starting from its bottom. The injection was carried out as follows: a cuff column of pipes with a diameter of 86 mm was installed into a pre-drilled injection well with a diameter of 168 mm, and the gap between the cuff tube and well walls was filled with casing mortar to prevent the injection mortar from escaping to the surface.

After the casing mortar developing its strength from 1 to 2 MPa (one day), cementation began. The packer was sequentially installed over each cuff and cement mortar penetrated the soil under pressure, breaking the created casing.

Injection was performed using a double packer, which ensured mortal penetration into the soil exclusively at a given site. For the injection, cement mortar with plasticizing additives was used, and in some areas it was soil-cement. The injection of the mortal through the cuff was made sequentially from the bottom to the top within the height of the working.

Laboratory studies were performed to select the soil-cement mix. Content of loam, as a waste of production and the cheapest material, was taken in the amount of 1 m^3 in bulk density per 1 m^3 of soil-cement.

The intergranular pores in the loam are filled with mixing water and grains of Portland cement. Cement content was allocated in the range of 100...200 kg/m^3 of the mortar according to the reference sources recommendations [7, 8] and taking into account the need in obtaining M25 grades of mortar.

The mixtures were put into 4×4×16 cm forms, after 3 days the formwork was removed and stored in baths with water until the required age. The day before the strength test, the samples were removed from the bath and stored in air-dry conditions. The age of the samples was 7, 14 and 28 days. Some part of the samples was left to study the durability for a longer period of hardening. Samples were weighed to determine the average density after molding and before each strength test.

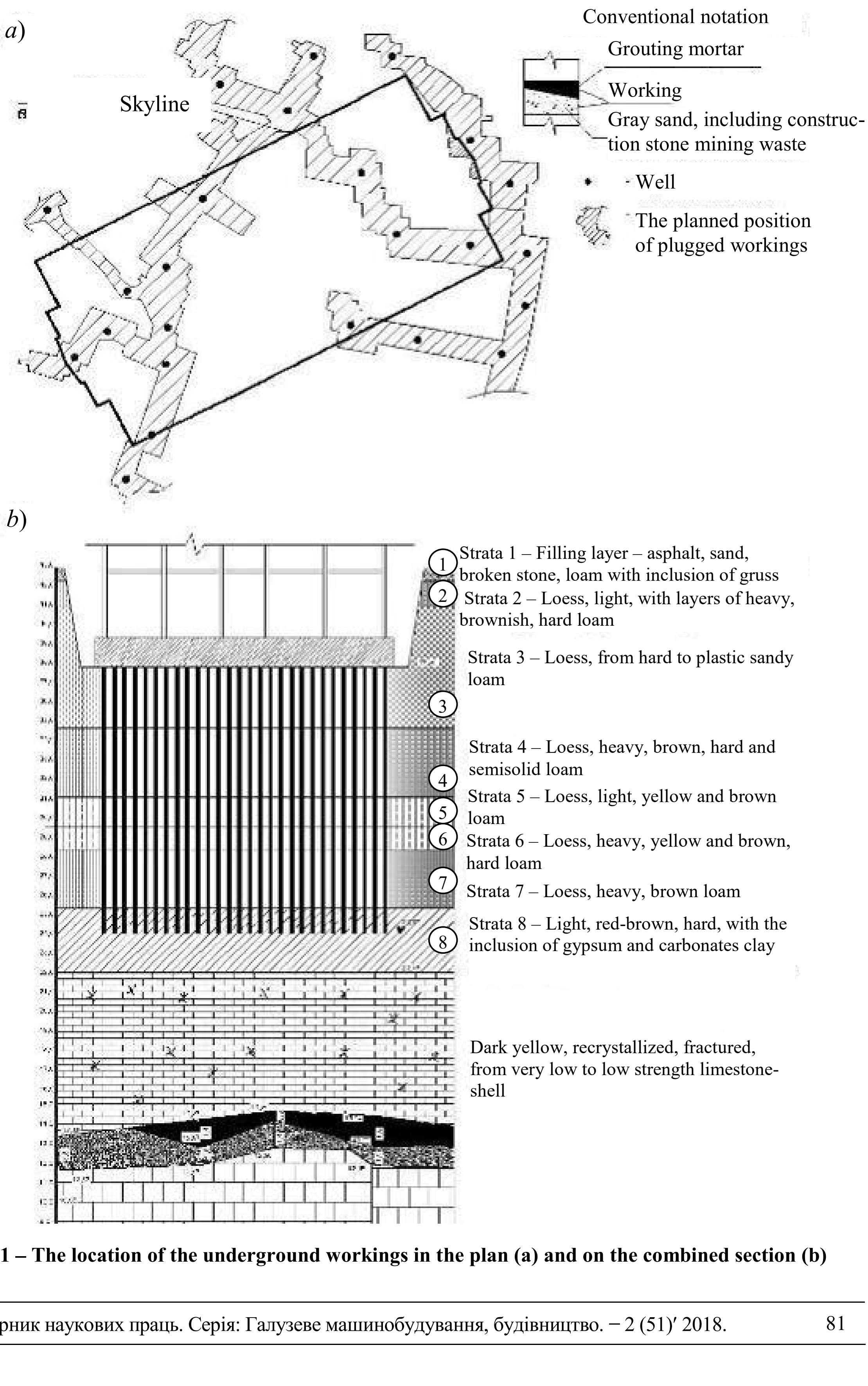

Figure 1 – The location of the underground workings in the plan (a) and on the combined section (b)

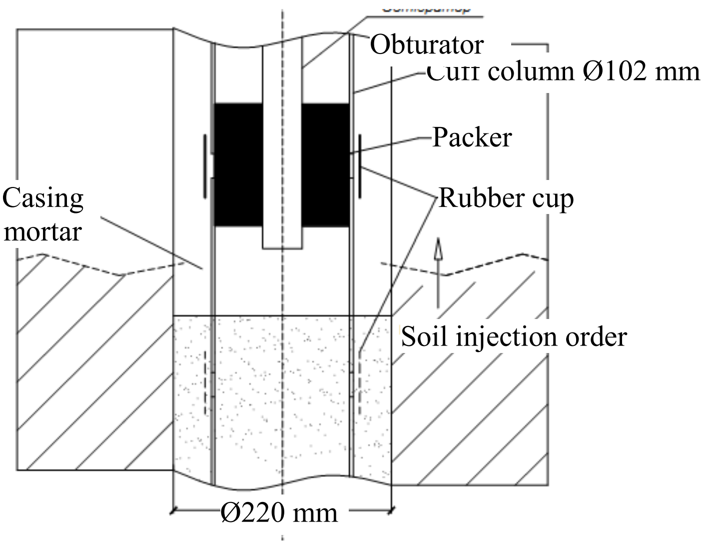

Figure 2 – The cuff column bottom

Compressive strength was determined on a PG-10 press. The value of strength was taken as the average result for the three sample beams. Sample marking was performed in series, nine sample beams in each series. The effect of cement content on the strength of soil-cement at the age of 28 days is presented in Fig. 3.

The effect of cement content on strength is considered at its age of 28 days. The intermediate 7 and 14 days have the same regularity and are lower than the grade strength by 50 and 30%. Increased cement content leads to increase in mortar strength.

Therefore, on the curves (Fig. 3), by means of interpolation, economically profitable cement content for each grade of soil-cement have been found.

The results of the experiments enabled to find a relation between soil-cement grade (M) and cement content:

Soil-cement grade	5	10	15	20	25	35
Cement (C), kg/m^3	50	85	115	150	165	200

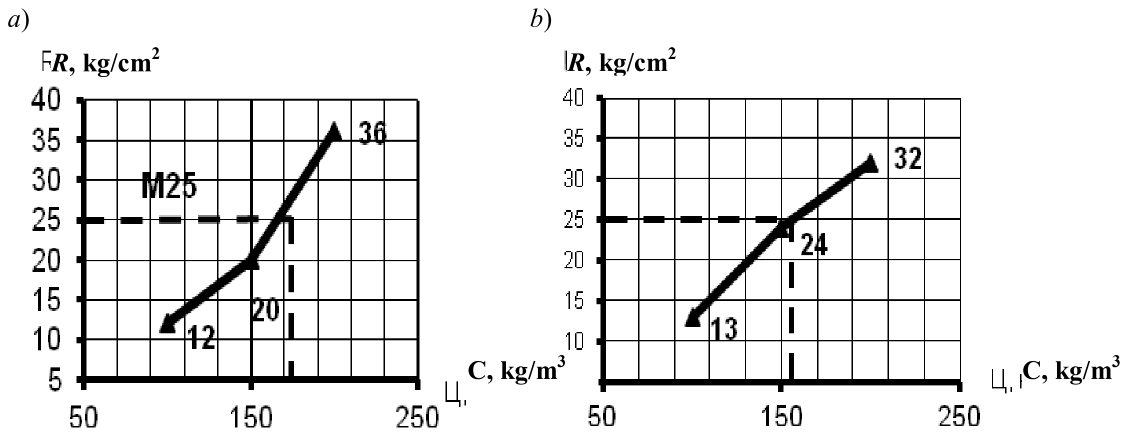

Figure 3 – The effect of cement content on the strength of soil-cement from mushy consistency (a) and low-slump (b) mixtures

The ratio between cement and loam, that is C / C : Loam / C = 1 : X (Fig. 4), is an important characteristic. It shows the uniformity and homogeneity of the distribution of cement grains in the intergranular space of loam and in the volume of its filling, determines the strength of soil-cement, its depending on the strength of the actual clay grains.

The curve nature at the age of 28 days indicates two general patterns in changing soil-cement strength from the C: Loam taken. There is slow influence of C: Loam in early terms – up to 14 days and accelerated – in the range of 14-28 days. The greater the ratio between cement and loam, the higher the soil-cement strength in the range of content C = 100 – 200 kg/m^3. This curve enables to choose the necessary ratio C: Loam for the range of soil-cement obtained grades by us using the example of M25 (there are dotted lines in the figures. Each soil-cement grade meets the only C ratio: Loam = 1:X: M10→1:10; M20→1:7; M25→1:6; M35→1:5.

The average density of soil-cement (Fig. 5) at the age of 28 days varies between 1560 – 1820 kg/m^3. The difference is 260 kg/m^3, which is 17%. Increase in the density of soil-cement by 1% leads to increase in its strength by 19%. This means that density is primary and strength is secondary.

Than density is higher, than cement content (not even water) is higher, since during samples testing (destruction) at the age of 28 days, it is noticed increasing amount of non-evaporated water to the sample center. And only a layer of 2 – 4 mm from the surface of the samples is dried. The inner zone of the samples is still wet and as they dry and hydrate, their strength should increase.

The strength of soil-cement over time (Fig. 5, b) increases with increasing cement content (Table 1).

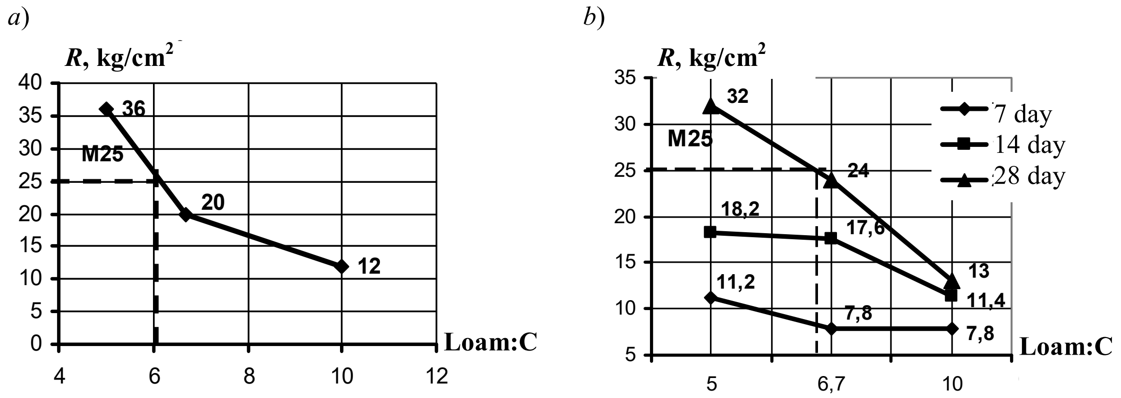

Figure 4 – The effect of soil and cement ratio on the strength of soil-cement from mushy consistency (a) and low-slump (b) mixtures

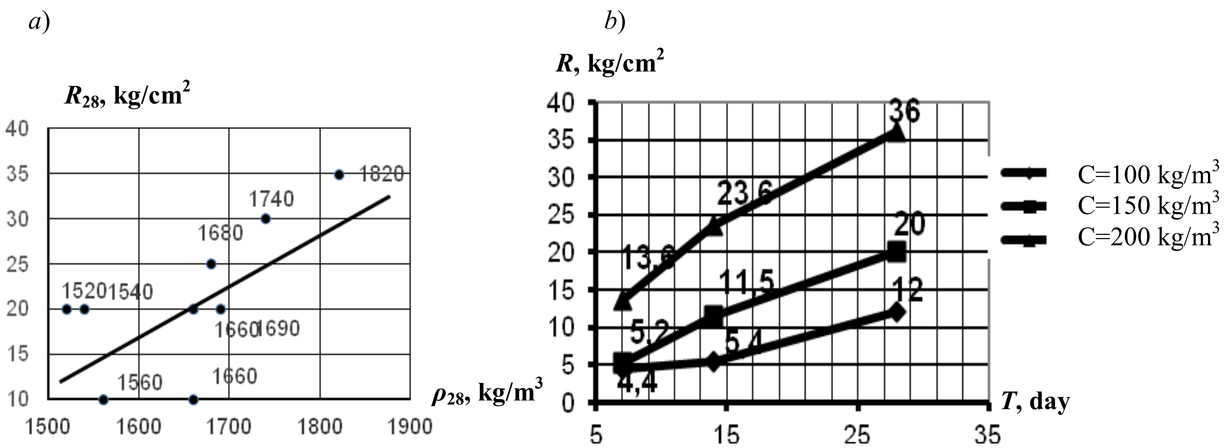

Figure 5 – The strength variation of soil-cement depending on its average density (a) and age (b)

Table 1 – Soil-cement mix for grades 10 – 35 from mushy consistency mixtures

Grade of soil-cement	Content of components for soil-cement, kg/m^3			
	Portland cement M400	Water	W/C	Loam
M10	85	460	5,4	1000
M15	110	470	4,3	1000
M20	150	495	3,3	1000
M25	165	495	3,0	1000
M35	200	500	2,5	1000

Conclusion. Thus, using the example of a 24-storey residential building design, comprehensive solution has been suggested for plugging workings (catacombs) in an array of limestone-shell rock for the subsequent construction of high-rise buildings on slab-pile foundations, which includes engineering and geological surveys, geotechnical design, selection of soil-cement grouts and their compositions, as well as works implementation with the constant monitoring of plugging conditions.

References

1. Fleming, K., Weltman, A., Randolph, M. & Elson, K. (2008). *Piling Engineering*. London and New York: Taylor and Francis.

2. Katzenbach, R., Bachmann, M. & Gutberlet, C. (2007). *Soil-structure interaction of deep foundations and the ULS design philosophy*. Geotechnical Engineering in Urban Environments: proc of the 14th European Conf. on Soil Mechanics and Geotechnical Engineering (Madrid, 2007). Millpress Science Publishers Rotterdam.

3. Готман, Н.З. & Каюмов, М.З. (2011). *К вопросу о расчете плитных фундаментов подземных сооружений на закарстованных территориях*. Фундаменты глубокого заложения и проблемы освоения подземного пространства: труды междунар. конф. Пермь, ПНИПУ.

4. Bauduin, C., Chitas, P., Raucroix, X. & Goffinet M. (2017). *Design of a piled raft foundation for a building at an alpine valley in Switzerland: Soil-structure interaction analysis and comparison with pile test results.* Proc. of the 19th Intern Conf. on Soil Mechanics and Geotechnical Engineering. Seoul, Korea.

5. Zotsenko, M.L., Vynnykov, Yu.L., Kharchenko, M.O., Nalyvaiko, L.G., Mitinskiy, V.M. & Aniskin. A. (2017). Foundations of the high rise building in the area of underground mining. *Academic Journal. Series: Industrial Machine Building, Civil Engineering,* 249). 252-260.

6. Митинский, В.М. (2016). Цементация зон разуплотнения грунтов с использованием манжетных колонн. *Будівельні конструкції. Механіка ґрунтів, геотехніка та фундаментобудування,* 83-2, 657-663.

7. ТУ УВ.2.7-26.6-02071033-001. (2007). Технологическая инструкция по производству закладочных работ при креплении (ликвидации) подземных горных выработок (катакомб) в г. Одессе. Одесса.

8. ТУ УВ.2.7-26.6-02071033-001. (2007). Бетон легкий (твердеющая закладка) на щебне известняка-ракушечника для крепления подземных горных выработок. Одесса.

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Experience of using Odessa region limestones as foundation base

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The characteristic properties of limestone-shell rock used as foundation base, including shear resistance transformation into friction resistance, are described. The main provisions of the method for determining piles bearing capacity using limestone mechanical characteristics dependence on the tensile strength under uniaxial compression in Odessa region. The results of limestone-shell rock full-scale tests by bored piles are presented. Experimental values of piles bearing capacity are compared with the values obtained by calculation using different methods.

Keywords: limestone-shell, bearing capacity, bored pile

Досвід використання вапняків одеського регіону як основи фундаментів

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Установлено, що при будівництві висотних будівель в Одесі часто використовують фундаменти з буронабивних паль, які занурюють у вапняк-черепашник та не прорізають усієї його товщі. Заглиблення паль у менш міцні меотичні глини не доцільно, оскільки навантаження від будівлі палями передається на весь масив вапняку. Визначено, що при розрахунку таких фундаментів виникають труднощі, оскільки вапняк-черепашник Одеського регіону не є скельною породою, а в нормативних документах відсутні дані для визначення несучої здатності буронабивних паль у вапняку як паль тертя. Екс**периментально доведено, що за результатами статичних випробувань буронабивних паль у вапняку-черепашнику їх несуча здатність значно вище значень аналітичних розрахунків за існуючими нормативними документами у вигляді паль-стійок, оскільки середнє значення межі міцності при одновісному стисненні вапняку малої міцності зазвичай становить R_c = 0,5... 1,0 МПа. Виявлено, що завданнями цих досліджень є визначення несучої здатності буронабивних паль у вапняку-черепашнику шляхом проведення статичних випробувань натурних зразків, а також порівняння отриманих результатів з аналітичними рішеннями різними способами для обґрунтування проектних рішень. Для розв'язання поставлених завдань застосовано експериментальні та розрахункові методи, як стандартні, так розроблені за участю авторів. З'ясовано, що комплексні дослідження дозволили визначити несучу здатність і допустиме навантаження на буронабивні палі шляхом проведення статичних випробувань та аналітичного розрахунку за нормативними документами й запропонованою методикою. Установлено, що результати розрахунку запропонованим методом дають хорошу збіжність з результатами статичних випробувань.

Ключові слова: вапняк-черепашик, несуча здатність, буронабивна паля

Introduction. While constructing high-rise buildings in Odessa, foundations from bored piles are often used, which are dipped in limestone-shell or cut it and deepen piles ends into the meiotic clay. Difficulties occur while calculating such foundations as far as limestone-shell rock is nor hard rock in Odessa region, and according to the indices of its physical and mechanical properties, cannot refer to rock or semi-rock, and consequently piles stopped in it cannot be regarded as bearers. Normative documents lack the data for determining lime stones resistance occurring under considering bored piles work as hanging piles. Peculiarities of these rocks are noted by other authors as well [1, 5, 7]. According to results of bored piles static testing in limestone-shell rock, their bearing capacity is considerably higher than those of analytical calculations under their work condition as bearing pile, since average value of strength limit under low strength limestone single-walled compressions usually equal to R_c = 0,5... 1,0 MPa.

Aim and objective. The main tasks of the present studies are to determine limestone-shell rock resistance on lateral surface and under bored piles heel according to the results of data obtained from static testing of models and full-scale samples; comparison of the results obtained with analytical solutions, and development of recommendations for their correction.

Study object and methods. The object of research is limestone-shell rock of Odessa region. To solve the tasks set for determining bearing capacity of bored piles in limestone-shell rock, experimental and calculating methods were used, including both standard ones and those developed by the authors.

Study results. Geological structure of the Black Sea plateau earth cover is represented by sediments of Quaternary and Tertiary age. Under the complex loess rocks with 6 to 23 m thickness there is red-brown clay underlain by Pontian limestone of Neogene.

Figure 1 shows the columns of the engineering-geological structure of some sites within Odessa city, where studies of pile foundations partially or completely buried in limestone-shell rock were carried out.

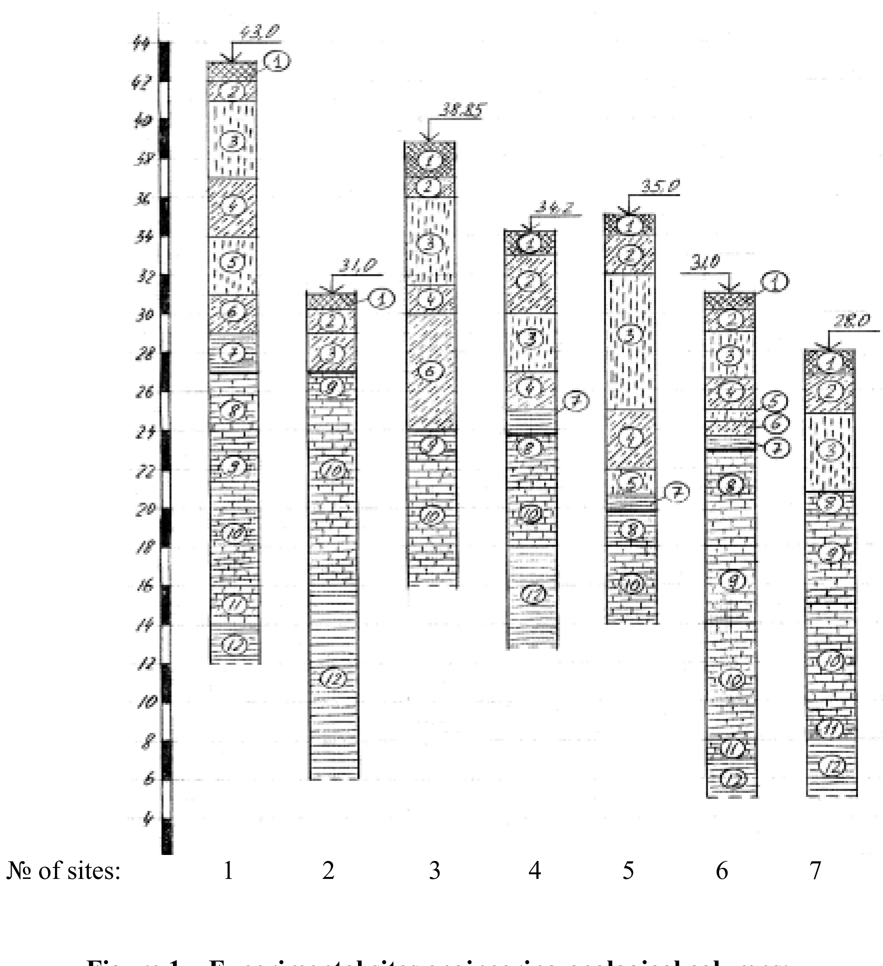

Figure 1 – Experimental sites engineering-geological columns:
1 – Lastochkina street (Odessa National Theater of Opera and Ballet);
2 – Primorsky Boulevard (section of the retaining wall); 3 – Zhukovsky street, 18 (regional archive);
4 – Pushkinskaya street, 9 (Odessa Museum of Western and Oriental Art);
5 – 15 Bolshoi Fountain station (cottage town); 6 – Genuezskaia street, 1 (group of high-rise buildings);
7 – Polskaia street, 10 (multi-storey office building)

The limestone layer of Pontic Stage has a rather complex and volatile structure, both in vertical and horizontal directions. The thickness of Pontic limestone can be divided into four layers. The first – the lowest layer (EGE-11) – is represented by slab limestone with 0.2 ... 1.0 m thickness. The second layer (EGE-10) is represented by a uniformly cemented limestone-shell rock ("sawn" limestone) with 4.5 ... 7.9 m thickness. Within this layer there are workings, so-called "catacombs". The third layer (EGE-9) is made up of strongly recrystallized limestone-shell rock. The fourth – the uppermost – layer (EGE-8) is predominantly represented by plate-clastic limestone. The total thickness of the third and fourth layers is equal to 5.0 ... 5.6 m. Placed above, there are loess and loess-like loams of different consistency, which are underlain by red-brown clay. Their thickness reaches 12 ... 16 m. Quaternary aquifer is widespread in loess rocks. It was formed mainly due to leaks from water-bearing communications.

Physical and mechanical properties of Pontic limestone depend upon many factors, including their fracturing. Limestone resistance to uniaxial compression varies widely. Characteristic feature is increase in their strength in the direction of watersheds and reduction in strength near large erosion cuts. In some parts of Odessa territory, strength of limestone in dry condition varies from 0.5 to 2.0 MPa. In condition of water saturation, limestone strength is reduced to 1.5-2 times.

Soil layering analysis and experience of using piles in Odessa region show that the most effective foundations while using limestone shells are drill piles, which do not completely cut its thickness. Sloping all the limestone thickness with piles and dipping them into less durable meiotic clay is not feasible, since load from the building is transferred over to the whole massif of limestone. Limestone layers availability as a cemented hard layer in the deformable zone of the base determines formation of a complex stressed state in it, in particular, concentration of tangential stresses. The problems occur due to possible pushing the thickness of limestone along the surface, forming a prism over the pile outer contour of slab-pile foundation [6]. Due to the limestone existence in the weakened zones, the inhomogenuity of its properties, the presence of natural cracks, pushing on one side of the foundation may take place, which lead to occurrence of an excessive roll and significantly impede the further building operation.

Experimental studies have established that when immersing drill piles in limestone-shell, along the lateral surface of the trunk, two types of resistance arise: resistance to structural bonds destruction, which transforms into frictional resistance after the «breakdown». Fracture resistance occurs under stresses exceeding structural strength of f_{str} shear, which is the ultimate shear strength. Friction resistance f occurs on the surface formed after the «breakdown». This phenomenon should be considered when determining drilling piles bearing capacity.

According to laboratory studies conducted earlier [3, 4], «breakdown» occurs when the barrel of the pile is moved to in average of 0.3 mm.

The average weighted value of shear resistance reduction coefficient along the trunk according to results of the studies conducted is 0.69, with the placement of piles across the layer.

Table 1 – Results of limestone-shell research by models of piles, located across the layers

№ Serial	Tests number	Resistance on the side surface		Pile side surface resistance decrease coefficient γ_{cf}
		before the breakdown f_c, MPa	after the breakdown f, MPa	
1	8	0.67	0.36	0.54
2	8	0.45	0.36	0.80
3	8	0.38	0.23	0.61
4	8	0.42	0.26	0.62
5	8	0.44	0.37	0.83
6	8	0.52	0.41	0.74
Average	**48**	**0.48**	**0.33**	**0.69**

At one of the objects in Odessa, limestone testing was carried out with six drill piles.

Pile basis is made of dark yellow, recrystallized, «plated», fractured with clay filler from very low to low strength limestone-shell (Fig. 2).

According to hydrogeology, the territory is characterized by lack of Quaternary and Pontic aquifers. The results of limestone testing with drilling Ø600 mm piles at construction site of the 1st stage are shown in Fig. 3, and the second line in Fig. 4. The exterior of the test installation is shown in Fig. 5.

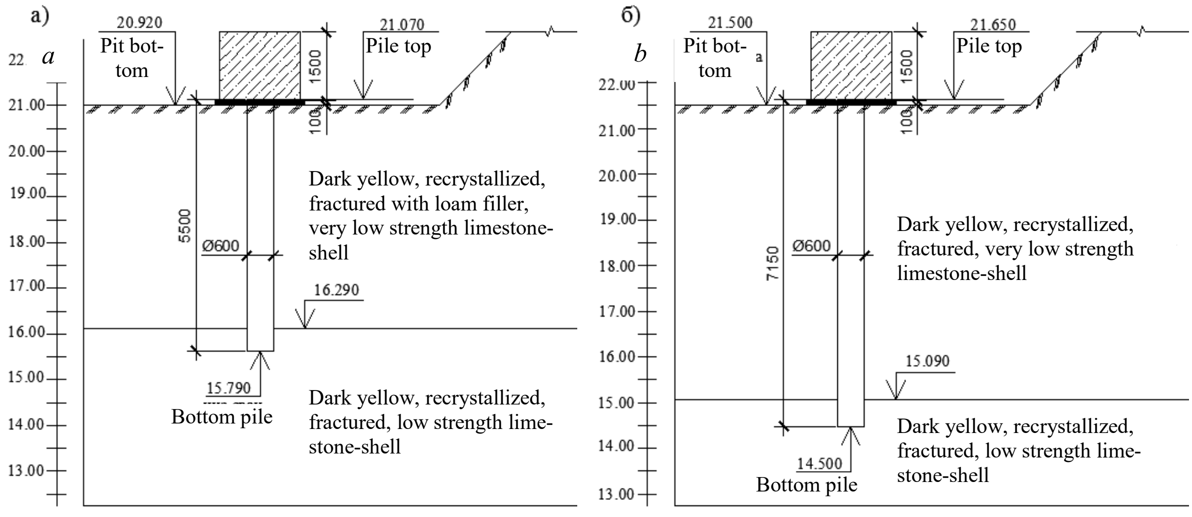

Figure 2 – Binding of piles to engineering-geological section:
a – 1st stage of construction (C-1, C-2 and C-3); *b* – 2nd stage of construction (P-4, P-5 and P-6)

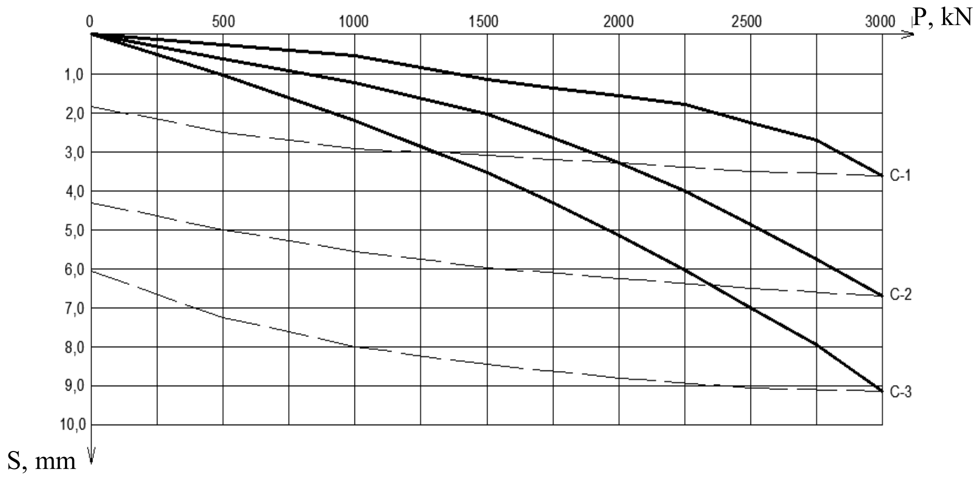

Figure 3 – Graphs of C-1, C-2 and C-3 piles moving from vertical load

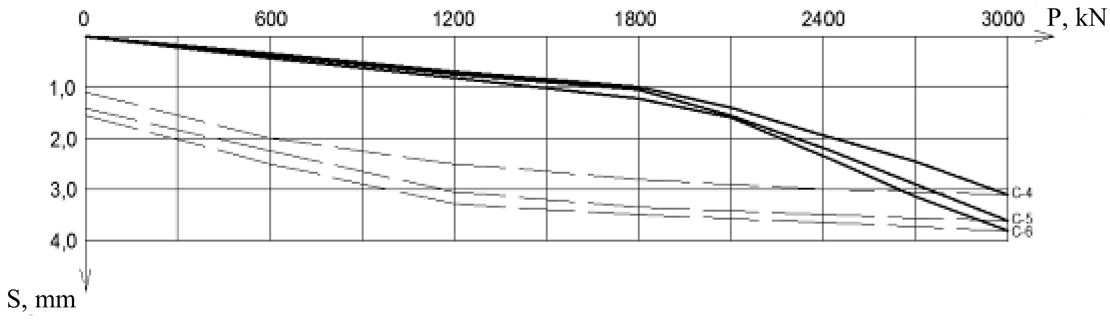

Figure 4 – Graphs of C-4, C-5 and C-6 piles moving from vertical load

Figure 5 – Fragment of soils testing with bored piles

The recommendations for calculation of bored piles partially or completely buried in a layer of limestone-shell rock are given. The H.3.1 formula [2] is used as a basis for calculation based on the vertical indentation load. This formula included coefficients and characteristics that determine bored piles work features in limestone-shell.

$$F_d = \gamma_c\, (\gamma_{cR}\, RA + u \sum \gamma_{cf} f_{c,i}\, h_i), \qquad (1)$$

where γ_c is pile work conditions coefficient; in case of piles being supported by dusty clay soils with moisture content S_r <0.9 and on loess soils γ_c = 0.8, in other cases γ_c = 1.0;

γ_{cR} – soils work conditions coefficient under pile bottom; γ_{cR} = 1;

R – calculated soil resistance under pile lower end, supported on limestone-shell rock, equal to its structural strength p_{str} and taken according to the graph in Fig. 6, in other case – according to Tab. H.2.1 [2], kPa;

A – pile support area, m^2;

u – perimeter of pile shaft cross section shaft, m;

γ_{cf} – coefficient of soil working conditions along pile lateral surface within limestone-shell rock, taken as γ_{cf} = 0.65, in other cases – according to Tab. H.3.1 [2];

$f_{c,i}$ –ultimate resistance to i-th soil layer shift along pile within limestone-shell rock lateral surface, $f_{c,i} = f_{str}$ is taken from the graph of Fig. 6, in other cases $f_{c.i} = f_i$ according to design resistance of the i-th layer of soil on the pile lateral surface, kPa, taken from Tab. H.2.2 [2].

Figure 6 shows correlation of limestone-shell rock strength parameters from the ultimate strength to uniaxial compression in the air-dry and water-saturated state, which were used while determining piles bearing capacity according to the proposed procedure.

Table 2 represents the results of tested piles bearing capacity determining as a result of full-scale tests, and also according to the current normative document [2] as pile-pillars and by the method proposed by the article authors and described in detail in [3].

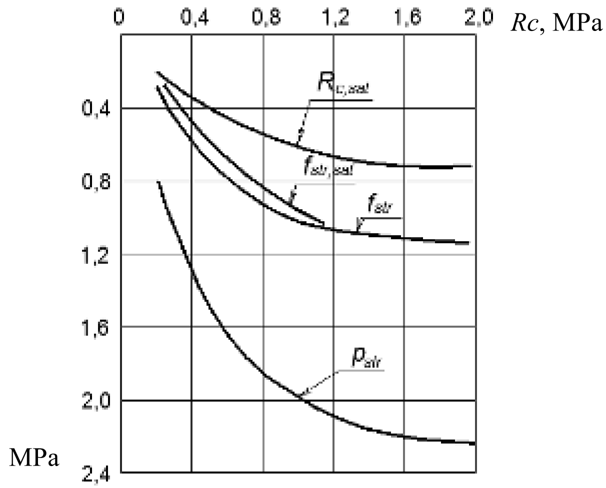

Figure 6 – Correlation dependencies of sawn limestone-shell rock limit values resistance parameters from the ultimate strength to uniaxial compression

Table 2 – The results of bored piles bearing capacity determining with different methods

Pile number	Piling depth Ø600 mm in limestone, m	Bearing/calculated bearing, kN, defined according to:		
		DBN V.2.1-10-2009	Methodology suggested	Testing results
C-1,C-2, C-3	5.50	1050/750	4030/2880	3000/2500
C-4,C-5, C-6	7.15	1340/947	5570/3980	3000/2500

Conclusion. The completed complex studies enable to establish bearing capacity and permissible load on the bored pile in limestone-shell rock by carrying out static tests and analytical calculations of the normative documents and the proposed methodology. The calculation results by the method suggested are convergence with static tests results. Convergence increases with the approach of displacements during testing to limit values.

References

1. Бойко, І.П., Карпенко, Ю.В., Новофастовський, С.М. & Подпрятов, В.С. (2004). Визначення несучої здатності буроін'єкційної палі великого діаметра за допомогою різних методів. *Основи і фундаменти,* 28, 79-94.
2. ДБН В.2.1-10-2009. (2011). *Основи та фундаменти споруд. Зміна №1.* Київ: Мінрегіонбуд України.
3. Новский, В.А. (2016). К расчету буронабивных свай в известняке-ракушечнике. *Вісник Одеської державної академії будівництва та архітектури,* 63, 298-303.
4. Kornienko, N.V., Novskiy, A.V., Novskiy, V.A., Tkalich, A.P., & Tugaenko, Y.F. (2011). *Mechanical Properties of Semi-Rocks Soils and Methods of Their Determination.* Proceedings of the 15th European Conf. on Soil Mechanics and Geotechnical Engineering. Part 1. (Athens). 43-47. Retrieved from http://ebooks.iospress.com/publication/31172
5. Зерцалов, М.Г. & Конюхов, Д.С. (2006). *Проблемы определения несущей способности свай в скальных грунтах. Достижения, проблемы и перспективные направления развития теории и практики механики грунтов и фундаментостроения*. Академические чтения по геотехнике. Казань, 28-30.
6. Митинский, В.М. & Бараник, С.В. (2017). Геотехническое обоснование строительства зданий повышенной этажности в г. Одессе. *Світ геотехніки*, 2(54), 10-13.
7. Aysen, A. (2005). *Soil Mechanics: Basic Concepts and Engineering Applications*. CRC Press.

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Peculiarities of residential buildings deformation with connected block-sections collision as a result of uneven settlement on collapsible soils

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Instrumental inspections and studies experience and results of nine-storeyed residential buildings block-sections actual strained-deformed state under the conditions of their collision in territories of 25 to 35 m thick collapsible soils in Zaporizhzhia city and the subsidence value of 100 cm and more are summarized. It is shown that depending on the process of uneven settlement in the time value of deviation vectors is stabilized over time along with the stabilization of subsidence and deformation of block-section structures on the collision area. It has been established that uneven settlement growth processes stabilization, block-sections deviation from vertical and collision forces is achieved at equilibrium between subsidence values, collision forces of block-sections and resistance of wall sections.

Keywords: collapsible soils, block-sections collision, structures damage, protection, restoration

Особливості деформування житлових будинків при зіткненні зблокованих блок-секцій від нерівномірних осідань на просадочних ґрунтах

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З’ясовано, що досвід експлуатації житлових будинків на деформованих основах свідчить про актуальність таких результатів для їх урахування при експлуатації житлових будинків у складних інженерно-геологічних умовах. Наведено результати інструментальних обстежень фактичного напружено-деформованого стану конструкцій блок-секцій житлових будинків за умов їх зіткнення на просідаючих ґрунтах зі значними величинами просідань. При цьому відзначено, що на стадіях розроблення типових серій будинків і проектування конкретних будинків не враховувалося можливе зіткнення блок-секцій. Зрозуміло, що зусилля зіткнення блок-секцій є неврахованим впливом на конструкції будинку має ряд особливостей роботи конструкцій будинку в зоні зіткнення, одна з яких – навантаження ділянок стін зусиллями зіткнення із площини стіни. Узагальнено досвід експлуатації та результати інструментальних обстежень і досліджень фактичного напружено-деформованого стану блок-секцій дев’ятиповерхових житлових будинків за умов їх зіткнення на територіях з просадочними ґрунтами товщиною 25 – 35 м у м. Запоріжжя й величинами осідань і просідань 100 см та більше. Показано, що залежно від процесу розвитку нерівномірних осідань у часі величини векторів відхилення стабілізуються у часі паралельно зі стабілізацією осідань і деформуванням конструкцій блок-секції на ділянці їх зіткнення. Установлено, що стабілізація процесів зростання нерівномірних осідань відхилення блок-секцій від вертикалі й зусиль зіткнення досягається при рівновазі між величинами осідань, зусиллями зіткнення блок-секцій та опором ділянок стін.

Ключові слова: просадочний ґрунт, зіткнення блок-секцій, пошкодження конструкцій, захист, відновлення.

Introduction. More than 80% of Ukraine territory is occupied by complex geotechnical conditions, including collapsing soils, undermined areas, karst phenomena, weak soils, etc.

A characteristic feature of these conditions is that during building and structure useful life they can get deformation of surface base and ground, negatively affect building operation [1].

Recent sources of research and publications analysis. In order to ensure the development of such territories, leading research and design institutions of Ukraine [5-9, 11] and abroad [10], integrated experimental research and design of various buildings and structures have been performed. Considerable volume is occupied by houses of various constructive decisions, which are used for mass development of the territory with such conditions.

Study of connected block-sections under the conditions of uneven settlement and block-sections collision in the last 20–30 years were not conducted. The results of instrumental observations of block-sections collision processes and deformations development in wall and ceiling structures are also not available. Experience in the operation of resitential houses on deformed bases indicates the relevance of such results for their consideration in residential buildings operation in complex engineering and geological conditions.

Selection of previously unsettled parts of the general problem.. When designing residential houses, the principles of block-sections blocking are often used. Figure 1 shows the schemes of blocking connected block-sections of residential buildings of mass development.

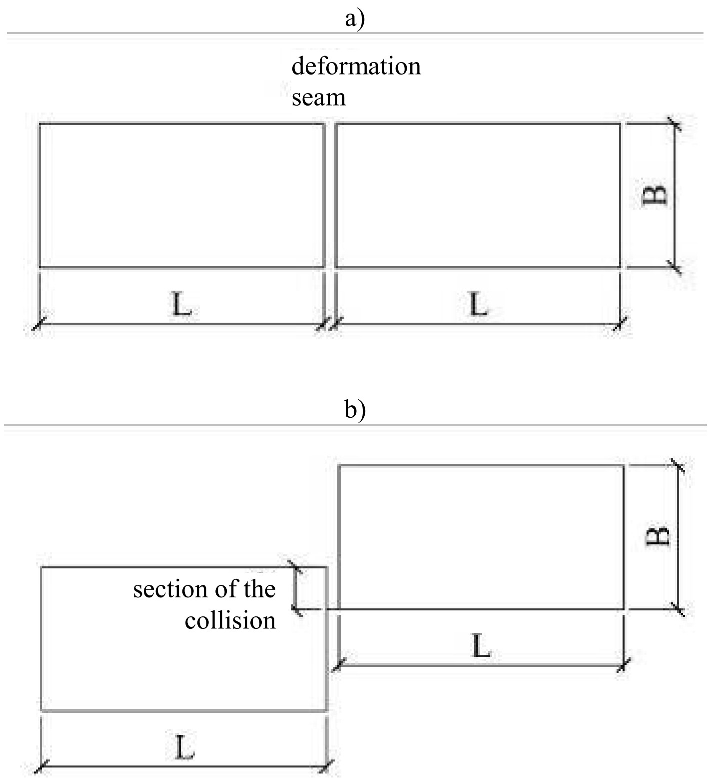

Figure 1 – Blocking circuit in terms of residential buildings block-sections:
a – without displacement in transverse direction;
b – with mutual displacement in transverse direction

From Figure 1 it can be seen that variants provide blocking of block-sections without mutual displacement in the transverse direction of block-sections and so displacements. Buildings operation experience shows that when uneven subsiding and tilting in block-sections collisions of the upper parts are formed. An important factor is the block section collision area magnitude. Figure 1 shows the blocking options for block-sections that are used on mass building of Ukrainian cities. The practice of designing residential buildings shows that the areas of block-sections collision vary from 100% of block-section width (B) to 30% of the width. It is obvious that at areas of block-section collision 50 – 30% of the block section width with deformation and damage of block-sections structures in the zone of their collisions is more intensive than when collision of block-sections is across the width.

It should be noted that at the stages of buildings typical series and specific houses designing development, the possible collision of block-sections has not been considered. The collision force of block-sections is not considered by the influence on the building design, it has a number of building construction features in the collision zone, one of which is the loading of wall sections by collision forces from wall plane.

Setting objectives. The article summarizes the experience of exploitation and the results of instrumental sinspection and studies of actual strained-deformed state of block-sections of nine-story residential houses in the conditions of their collision in areas with collapsible soils of Zaporizhzhia with a depth of 25 – 35 meters and the values of sedimentation and sinking of 100 cm and more [2].

The main material and the results. Figures 2, 3 give a general view of a residential building and block-sections collision zone in a deformation joint with a 30% area of collision from the width of block section.

At the same time, the gap between the block-sections along the top of a deformation joint on this house was 250 mm under the project.

Figure 4 shows the indicative vectors of block-sections deviation from the vertical at the level of their top when they collide.

Figure 2 – General view of a residential building with collision of block-sections

Figure 3 – The areas of house block-sections collision in a deformation joint

Figure 4 – Indicative vectors of deviation from vertical at the level of block-sections top when they collide

The unevenness of settlement on each block-section at the moment of measuring the block-sections deviation values was 225 – 250 mm. Depending on the values of uneven settlement and their distribution, in terms of each block-section, block-sections deviation magnitude is different. Depending on uneven settlement process course increasing in time, the values of the deviations vector stabilize in time in parallel with deposition stabilization and block-sections structures deformation in the area of their collision. Processes stabilization of block-sections deviation uneven settlement increasing from the vertical and the collision forces is achieved with the equilibrium between the values of uneven settlement, the efforts of block-sections collision and walls resistance. The values of structural elements deformation and damage are conditioned by the magnitude of block-sections collision force. With the values of block-sections deviations vectors (tilt) shown in Figure 4, deformations and damage of the panels and their joints in the area of block-sections collision are recorded. Along a deformation joint, complete «closure» of the gap between block-sections outer walls planes at the level of the ninth and eighth floors is formed. Panels joints opening is up to 6 mm. The cracking in the wall panels reaches 6 – 8 mm. The zones of panels damage by grid of cracks are also fixed with the fragmentation of panel concrete by the efforts of plane pressure with the areas up to 1 – 3 m^2. Slabs of the overlap formracks with opening up to 0,3-0,5 mm. According to the criteria [2], structures technical state in the zone of block-sections collision in general has reached the «3» category – unsuitable for normal exploitation. According to the criteria [2], structures technical state in block-sections collision zone in general has reached the «3d» category - unsuitable for normal exploitation.

The study of a residential building connected block-sections with three block-sections structures deformations with values of the collision areas on the deformation joint of 70% of the block-sections width has been carried out.

Maximum uneven settlement for each block-section ranged from 294 – 350 mm to 488 – 507 mm. In this case, the maximum vectors of the top block-sections deviation from the vertical were 330–350 mm. It has been established that in zones with block-sections collision of panels deformation and floor slabs, their joints are more intense on the upper floors (the ninth and the eighth). The joints in the panels reached 1.5 – 2.5 mm. In separate panels, cracks opening was up to 4 mm. Cracks detection in the panels was 0.4 – 1.5 mm. In overlappings slabs, the main crack opening did not exceed 0.3 – 0.5 mm.

In staircase cover, slab joints opening reaches 3-4 mm with areas of damage to the joints and zones of long soaking by atmospheric precipitation.

The values of deformation at the level of the ninth floor overlay were:

– joints of sites with wall panels, elevator shaft opening - mainly up to 1 mm;

– joints disclosure of intermediate platforms, staircase marches in refinement knots – up to 1mm.

At the levels of the eighth, seventh floors, the values of similar deformations are 1 – 3 mm. At the levels of the sixth, fifth floors they are in the range of 0.5 – 1 mm. From the fourth to the first floor, the opening of slabs, stairs and marches joints does not generally exceed 0.5 mm. All structures and joints of deformation magnitude exceeded the limit, regulated by normative documents, have the «3» category of technical condition, and require the performance of repair and restoration work.

The received values of block-sections structures deformations comparison shows that with the magnitude of the collision area up to 70 – 100% increase in the unevenness of settlement and deviation from the vertical almost twice - structures deformation of block-sections in the collision zones have smaller values than with the magnitude of the collision zone 30 %

It should be noted that the given results are obtained with the size of the deformation joint on the project between block-sections of 250 mm.

It is important that the inhabitants of apartments located in the zones of block-sections collision in the process of forming collision and development of deformations and damage in the collision zone repeatedly performed «decorative» repairs of damage parts, but with further uneven settlement of block-section – areas of deformation and damage appeared and increased.

According to the requirements [2], in the technical condition of structures «3» and «4» category, it is necessary to restore, reinforce or replace damaged structures in order to ensure inhabitants safety in premises with such structures.

In such situations, the necessary step is to repair block-sections using the alignment technologies and bringing block-sections with a till to avertical position.

At State Research Institute of Building Constructions the technologies of leveling houses have been designed the most:

–method of using the hydraulic jacks system;

–method of exterminating the soil under block-sections foundations.

Each method has its own peculiarities and conditions of rational use. After eliminating the tills, repair and renovation works on deformed and damaged house structures are carried out.

Considering the possibility of block-sections collisions repeated formation on subsurface soils with significant amounts of settlement, the most radical approach is, after blocks removing from block-section (alignment), to fix foundation base to eliminate the possible formation of uneven settlement in future according to the State Standard-NSC.1.1-44: 2016 [3].

Conclusions. Deformation and damage of residential buildings under the conditions of block-sections collision are specific influences on the design of building block section upper part. Such influences have been not considered at the stage of block-section designs and residential buildings design under specific building conditions development. On territories formed by collapsible soils with a thickness of 20–30 m and the values of subsidence and shrinkage of 100 cm or more when block-sections collision is formed in deformated and damaged structure technical condition category«3» and «4», which excludes the normal further operation of such block-sections and significantly increases operating costs. The number of block-sections multiple collisions where tilts were previously eliminated, increases in time. Necessary measures are speedy repair and restoration work with the main stages: block-sections tilts (leveling) elimination; repair and restoration works performance.

On soils with significant values of subsidence it is rational, after removing the blocks of the block-section (alignment), to fix foundation base to eliminate the possible formation of uneven settlement in time. Necessary factor is technologies practical implementation and development for restoring bearing capacity, bearing structural elements with significant damage strength in the areas of block-sections collision under the conditions of access from internal premises. The obtained results enable to carry out built-up areas analyzes of cities with connected sections of residential buildings and to develop the necessary preventive measures for collisions reduction or their complete exclusion. Biased analysis of built-up areas in Ukrainian cities with the identification of potentially dangerous block-sections in the conditions of block-sections collisions and tilts manifestation is important and expedient. Such biased analyzes and measures provide minimization of risks and consequences for residential buildings operation, as well as significant reduction in operating costs.

References

1. ДБН В.1.1-45:2017. (2017). *Будівлі і споруди в складних інженерно-геологічних умовах. Загальні положення.* Київ: Мінрегіонбуд України, Укрархбудінформ.
2. ДСТУ-НБВ.1.2-18:2016. (2017). *Настанова щодо обстеження будівель і споруд для визначення та оцінки їх технічного стану*. Київ: ДП УкрНДНЦ.
3. ДСТУ-НБВ.1.1-44:2016. (2017). *Настанова щодо проектування будівель і споруд на просідаючих ґрунтах*. Київ: ДП УкрНДНЦ.
4. ДСТУ БВ.3.1-2:2016. (2017). *Ремонт і підсилення несучих і огороджувальних будівельних конструкцій та основ будівель і споруд*. Київ: ДП УкрНДНЦ.
5. Клепиков, С.Н. (1996). *Расчет сооружений на деформируемом основании*. Київ: НИИСК.
6. Петраков, А.А. (1997). Исследование предельных состояний сооружений на деформируемом основании. *Современные проблемы строительства: науч.-техн. сб*орник, 3, 22-28.
7. Москалина, И.Н., Трегуб, А.С. & Григорьев, Г.М. (1997). Экспериментальные исследования крупнопанельных зданий новых конструктивных решений для их строительства на подрабатываемых территориях Донбасса *Современные проблемы строительства: науч.-техн. сб*орник, 3, 28-32.
8. Григорьев, Г.М., Трегуб, А.С. & Москалина, И.Н. (1997). Экспериментальные исследования крупнопанельных зданий и закономерности трещинообразования в их несущих элементах под влиянием просадок. *Будівельні конструкції: зб. наук. праць,* 24, 142-148.

9. Клепиков, С.Н., Трегуб, А.С. & Москалина, И.Н. (1980). Испытание зданий на прочность и устойчивость. *Основания, фундаменты и механика грунтов,* 4, 7-8.

10. Smirakova, M., Mateckova, P. & Buchta, V. (2016). Deformation of Foundation Structure and their Experimental Testing. *International Journal of Theoretical and Applied Mechanics*, 1, 303-308.

11. Kryvosheiev, P., Farenyuk, G., Tytarenko, V., Boyko, I., Kornienko, M., Zotsenko, M., Vynnykov, Yu., Siedin, V., Shokarev, V., Krysan, V. (2017). *Innovative projects in difficult soil conditions using artificial foundation and base, arranged without soil excavation.* Proc. of the 19th International Conf. on Soil Mechanics and Geotechnical Engineering (Sep. 17 – 22, 2017 / COEX, Seoul, Korea), Seoul.

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Economic efficiency of vibroreinforced soil-cement piles implemented in construction

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The results of the executed economic comparison of foil-cement piles reinforcement use by vibroreinforced - vibroreinforced soil-cement piles are presented. The comparison is based on the results of piles bearing capacity by field tests. Considering the results of field tests two variants of piles foundation have been designed for residential building. Economical comparison is made for implementation effectiveness of reinforced and non-reinforced soil-cement piles at the piles calculated bearing capacity on the soil that is greater than the calculated bearing capacity by the material. .Economical comparison is made for implementation effectiveness of vibrated vibroreinforced soil-cement piles and bored piles at their calculated bearing capacity by the material at times greater than that on the soil. The obtained results and the determined economic effect are analyzed. The results of comparison are used for design and implemented in construction.

Keywords: the vibroreinforced soil-cement piles, economic efficiency, economic efficiency, deep soil mixing method, the bored piles.

Економічна ефективність упровадження віброармованих ґрунтоцементних паль

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Доведено, що підвищення несучої здатності ґрунтоцементних паль завдяки збільшенню міцності матеріалу паль та підсиленню їх арматурними каркасами дозволяє значно збільшити несучу здатність ґрунтоцементних паль та зменшити їх кількість, що також позитивно впливає на зменшення ресурсомісткості зведення фундаментів будівель. Викладено результати проведеного економічного порівняння використання підсилення ґрунтоцементних паль віброармуванням – віброармованих ґрунтоцементних паль. В основу порівняння прийнято дані результатів натурних випробувань несучої здатності паль, що були проведені на будівельних майданчиках. Виконано економічне порівняння ефективності впровадження непідсилених та віброармованих ґрунтоцементних паль при розрахунковій несучій здатності паль по ґрунту більшій, ніж розрахункова несуча здатність по матеріалу палі, а також економічне порівняння ефективності впровадження віброармованих ґрунтоцементних паль та буронабивних паль. При цьому розрахункова несуча здатності по матеріалу для віброармованих ґрунтоцементних паль була незначно більша, ніж по ґрунту, а для буронабивних паль розрахункова несуча здатність по матеріалу в рази більша, ніж по ґрунту. Проаналізовано отримані результати та визначено економічний ефект використання віброармованих ґрунтоцементних паль. Виділена оптимальна сфера застосування, в якій ґрунтоцементні палі є ефективним та економічно вигіднішим видом пальового фундаменту. Результати порівняння впроваджені в будівництво.

Ключові слова: віброармовані ґрунтоцементні палі, економічна ефективність, бурозмішувальний метод, буронабивні палі

Introduction. Increasing the bearing capacity of soil-cement piles by increasing the piles material strength and reinforcing them with frames enables to increase the bearing capacity of the soil-cement piles and reduce their number significantly that also positively affects the resource volume reduction of the foundation and building or structure underground part. [1 – 6, 9 – 14]

Review of the latest research sources and publications. Works of domestic and foreign researchers are devoted to the study of soil-cement foundations: V. Askalanov, V. Bezruk, I. Boyko, Yu. Vynnykov, M. Zotsenko, V. Chrisan, I. Lartseva, V. Marchenko, T. Nesterenko, R. Petrash, O. Petrash, P. Rebinder, A. Tokin, V. Shapoval, N. Denies, A. Le Kouby, S. Lambert, F. Rocher-Lacoste, Osamu Taki, R. Bell, and others.

The conducted studies [1, 6, 7] have proved the economic efficiency of using the soil-cement foundations as an improvement ground. In particular, it is necessary to highlight a perspective using area of one of the soil-cement foundations types, namely, soil-cement piles as an alternative to reinforced concrete piles with a lower resource volume for the foundation.

Definition of unsolved aspects of the problem. The performed increase of piles bearing capacity on the material enables to expand the scope of vibroreinforced soil-cement piles using. [1 – 4, 11 – 14] Most of the pile foundations on the territory of Ukraine are made with friction piles that lean on the sandy soils on the heel, so the bearing capacity of the reinforced concrete piles on the material far exceeds the bearing capacity of the soil. To reduce the cost of foundation building rationally use piles bearing capacity on the material should in a small degree exceed the bearing capacity on the ground, which reduces the resource volume of the foundation.

Problem statement. The developed vibroreinforced soil-cement piles [8] need to compare their implementation with already known types of piles and a sufficiently widespread method of reinforcing the base with soil-cement elements, which uses similar drilling equipment.

Basic material and results. For the more effective use of soil-cement piles, the author solves the problem of low bearing capacity on the material of piles trunk. Experimental way has determined the basic physical and mechanical properties of soil-cement that affect its strength, methods of increasing piles bearing capacity and testing their effectiveness in laboratory and field conditions.

The conducted studies enable to distinguish a new type of improved soil-cement foundations - vibroreinforced soil-cement piles (VRSCP). [11]

Comparison of pile foundation variants and determination of efficiency and resource volume of methods application for increasing the bearing capacity of soil-cement piles was carried out at the design and construction of a 10-storey residential building bulk 8 in the Esplanada district of Sumy.

By comparison, two variants of soil-cement piles foundations were adopted: soil-cement piles without reinforcement (non reinforced) and soil-cement piles with reinforcement and vibration sealing of the material (reinforced) – vibroreinforced soil-cement piles. The piles can be arranged using a single set of equipment, and an additional deep-hole, pin-shaped vibrator is used for reinforcement.

The presence of sand inhomogeneous emulsions enables to construct the bored piles without fixing the walls of the well. The hammering of concrete piles was not used due to the close location of existing buildings.

For both variants of soil-cement piles the following is typical: length 9 m diameter 750 mm (area of the section 0,44 m^2), the bottom end of the piles based on firm middle sand.

The results of the variants comparison for foundations are shown in Table 1.

Table 1 – Comparison variants of soil-cement piles foundations

Index	Variants of soil-cement piles	
	non reinforced	vibro-reinforced
Bearing capacity, tons	65	105
Design load on pile, kN	510	824
Piles number	256	166
Full volume, m^3	1013,76	657,36
Cement cost, UAH	387 070	250 990
Armature cost, UAH	0	124 080
Production cost, UAH	1 317 888	920 304
The economic effect of piles, UAH		410 784
Grillage construction, UAH	717 000	591 000
The overall economic effect, UAH		536 784

Before the start of construction, static tests of pilot piles with and without reinforcement were performed. As a result, the bearing capacity of non-reinforced piles amounted to 65 tons, reinforced, in turn, 105 tons. These data were used to design two variants for pile foundations and grillage.

The analysis of the performed calculation shows that due to the increase of the bearing capacity of the soil-cement piles, their total quantity decreased, and hence the volume of resources for them. Reducing the piles number also enables to reduce the grillage width.

The overall economic effect is 536 784 UAH, which is 26,4%.

Vibroreinforced soil-cement piles were also introduced at the design of pile foundations for the Student Cultural Center for 500 places at Sumy National Agrarian University.

For the construction two variants of pile foundations were designed: from bored piles and vibroreinforced soil-cement piles (VRSCP). The two variants as a result of the calculation were designed with a depth of 6.5 m and a diameter of 500 mm, while the bearing capacity of the piles in the soil is the same.

Calculated bearing capacity of bored piles on a material from concrete C8/10 considering reinforcement, was 1470 kN. The lack of groundwater and soil types allow drilling wells for drill piles without forcing walls of the well.

The estimated bearing capacity of the soil-cement piles without reinforcement was 196.0 kN, which is less than the permissible estimated ground load, therefore it was decided to design vibroreinforced piles. The estimated bearing capacity for the material of the vibroreinforced soil-cement pile was 294.0 kN.

The results of options comparison for foundations are shown in Table 2.

The analysis of the performed calculation shows that in the case of soil conditions where the bearing capacity on the soil is low, there is a problem of material for piles made of concrete bearing capacity underutilization. At the same time, at the expense of the same work volumes, vibroreinforced soil-cement piles have a lower resource volume and production costs.

The introduction of vibroreinforced soil-cement piles was carried out in the construction of residential buildings in Sumy with the participation of the organization PrJSC «Sumbud».

The comparison of variants of pile foundations has shown the efficiency and resource intensiveness of application of methods of increasing the bearing capacity of soil-cement piles, reducing the resource volume of using reinforced soil-cement piles.

Table 2 – Comparison variants of piles foundations

Index	Variants of pile foundation	
	BP	VRSCP
Design bearing capacity bearing on soil, kN	334,0	334,0
Design bearing capacity bearing on material, kN	1470,0	294,0
Design load on pile, kN	238,57	238,57
Piles number	145	145
Full volume, m^3	178,36	178,36
Material cost, UAH	227 442	156 859
Production cost, UAH	259 844	249 704
The overall economic effect, UAH		80 724
Design bearing capacity bearing on soil, kN	334,0	334,0
Design bearing capacity bearing on material, kN	1470,0	294,0

The overall economic effect 80 724 UAH, what is 16,57 %.

Reducing resource volume has a positive effect on reducing the construction estimated cost, so it is a cost-effective solution.

Construction of a 10-storey 110 apartment residential building 8 in the district of Esplanade, in Sumy, finished in the 1st quarter of 2016. The building is built on 166 vibroreinforced soil-cement piles with a diameter of 750 mm, 9 m length, 6 m depth reinforced. The plan of the pile field is shown in Fig. 1.

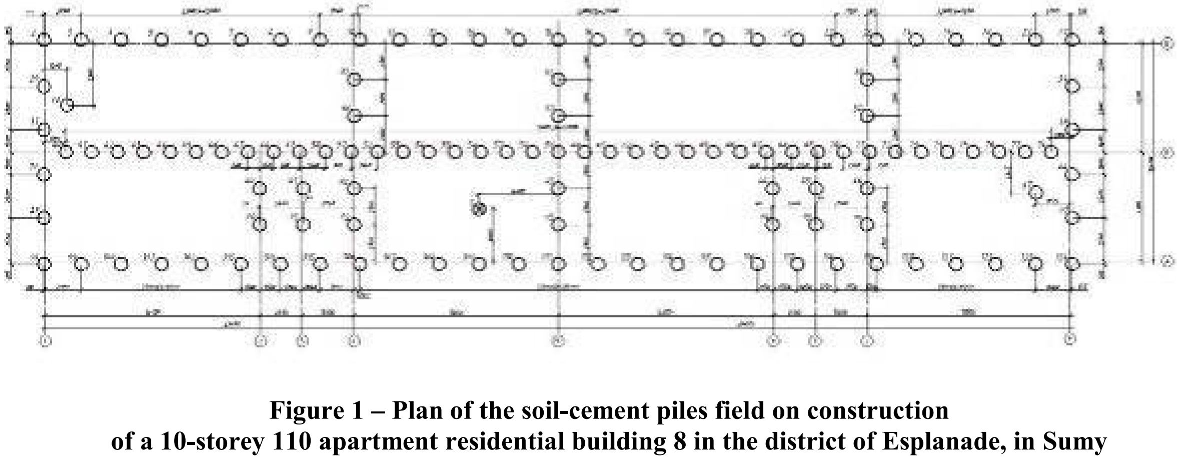

Figure 1 – Plan of the soil-cement piles field on construction of a 10-storey 110 apartment residential building 8 in the district of Esplanade, in Sumy

Figure 2 – Vibrireinforsing of soil-cement pile

The soils on the site are divided into 17 engineering geological elements (EGE), an array of nonuniform fine sand to a depth of 4.6 m; clay soils to a depth of 4.6 to 6.8 m; fine sands 6,8 – 11,0 m and from a depth of 11 m – medium sand.

After the economic comparison of different types of foundations, a pile foundation was designed. The analysis of soils showed the possibility of using the mixing technology for the placement of soil-cement piles. The detailed calculation of the dependence of the bearing capacity of the pile from the depth indicated the need for immersion of piles in the EGE VIII medium sands, with the calculated bearing capacity on the soil was 1491 kN.

It was decided to carry out the construction of vibroreinforced soil-cement piles with a depression in the bearing layer of soil not less than 1 m.

The VRSCP arrangement is carried out using a set of equipment which includes:

– drilling rig on the basis of RDC-250 with attachments. The working body for the soil destruction has openings for distributing the cement slurry throughout the well section;

– mortar mixer for making a cement slurry;

– mortar pump for injection of a slurry to a well;

– deep vibrator.

Composition of water-cement suspension – «cement + water». For 1 running meter of vibroreinforced soil-cement pile with a diameter of 750 mm (volume 0.44 m^3) is spent:

– cement M500 – 133 kg, 300 kg / m^3,
that is 1200 kg for 9.0 m SCP 3.96 m^3
(assumed by the project);
or
– cement M400 – 150 kg, 340 kg / m^3,
that is 1350 kg for 9.0 m SCP 3.96 m3
(assumed by the project);

– water – 60% of the weight of cement.

Water-cement ratio of the suspension W / C = 0,6.

Vibroreinforced soil-cement piles were arranged with a deep soil mixing technology without the removal of soil. The essence of the applied technology is that during the well drilling, the natural soil is loosened without taking it out of the well. In the well bore the solution pump a water-cement suspension, which is carefully mixed with the loose soil by the working organ. Loosening the soil, supplying the cement mortar and mixing it with the soil is carried out throughout the length of the soil-cement pile. In the newly made soil-cement piles, a metal frame is pressed, in the middle of which a high-frequency vibrator is placed, which simplifies the deepening of the frame. Constructive material is subjected to high-frequency deep vibrating when removing the vibrator up. After the mixture has dried, a solid soil-cement pile, which does not soak in the aqueous medium, is formed. Using the deep soil mixing technology without removing soil enables to made soil-cement piles below ground water level.

Conclusions The experience of introducing vibroreinforced soil-cement piles shows their use possibility in difficult engineering and geological conditions. The performed economic comparisons of work and materials cost indicate that vibroreinforced soil-cement piles are more economical and less resource volume variant of foundations compared to non-reinforced cement piles; if the load bearing capacity on the soil is greater than the bearing capacity of the material as well as in comparison with bored piles if bearing capacity of the soil is much smaller than the bearing capacity of piles material.

The vibroreinforced soil-cement piles have been used in the construction and design of residential and public buildings as a more economical version of pile foundations.

References

1. Denies, N. & Lysebetten, G.V. (2012). *Summary of the short courses of the IS-GI 2012 latest advances in deep mixing*. Proc. of the Intern. Symposium on Ground Improvement IS-GI. Brussels.

2. Zotsenko, M., Vynnykov, Yu., Doubrovsky, M., Oganesyan, V., Shokarev, V., Syedin, V., Shapoval, A., Poizner, M., Krysan, V. & Meshcheryakov, G. (2013). *Innovative solutions in the field of geotechnical construction and coastal geotechnical engineering under difficult engineering-geological conditions of Ukraine.* Proc. of the 18th Intern. Conf. on Soil Mechanics and Geotechnical Engineering. Paris.

3. Zotsenko, N., Vynnykov, Yu. & Zotsenko V. (2015). Soil-cement piles by boring-mixing technology. *Energy, energy saving and rational nature use*. Oradea University Press, 2015.

4. Зоценко, М.Л., Винников, Ю.Л. & Зоценко, В.М. (2016). *Бурові ґрунтоцементні палі, які виготовляються за бурозмішувальним методом.* Харків: «Друкарня Мадрид».

5. Зоценко, М.Л., Сухоросов, І.М. & Зоценко, Л.М. (2007). Порівняльна характеристика фундаментів будівель і споруд із паль та на армованій основі. *Міжвідомчий наук.-техн. зб. наук. пр. (будівництво),* 66, 405-409.

6. Зоценко, Н.Л., Коршунов, М.О. & Передерий, Н.Ф. (1987) Сокращение энергозатрат при устройстве фундаментов. *Промышленное строительство и инженерные сооружения,* 1, 14-17.

7. Крисан, В.І. (2010). *Дослідження напружено-деформованого стану ґрунтового масиву, армованого ґрунтоцементними елементами, що виготовлені по струминно-змішувальній методиці*. (Автореф. дис. канд. техн. наук). Полтавський національний технічний університет імені Юрія Кондратюка. Полтава.

8. Ларцева, І.І., Петраш, Р.В. & Петраш, С.С. (2006). Економічна ефективність використання ґрунтоцементних паль як фундаментів будівель і споруд. *Економіка і регіон*, 1(8). 118-121.

9. Нестеренко, Т.М. (2013). *Ґрунтоцементні основи і фундаменти, які виготовлені з використанням вібрування.* (Автореф. дис. канд. техн. наук). Полтавський національний технічний університет імені Юрія Кондратюка. Полтава.

10. Петраш, О.В. (2013). *Ґрунтоцементні палі, виготовлені за бурозмішувальною технологією.* (Автореф. дис. канд. техн. наук). Полтавський національний технічний університет імені Юрія Кондратюка. Полтава.

11. Новицький, О.П. (2015). *Віброармовані ґрунтоцементні палі, виготовлені за бурозмішувальним методом.* (Автореф. дис. канд. техн. наук). Полтавський національний технічний університет імені Юрія Кондратюка. Полтава.

12. Bruce, D.A. (2000). Introduction to the Deep Soil Mixing Methods as Used in Geotechnical Applications. *FHWA-RD-99-138*. Springfield, Virginia. https://books.google.com.ua

13.Lambert S., Rocher-Lacoste, F. & Le Kouby, A. (2012). *Soil-cement columns, an alternative soil improvement method*. International symposium of ISSMGE-TC211. Recent research, advances & execution aspects of ground improvement works (31 May – 1 June 2012). Brussels, Belgium.

14. Denies, N. & Van Lysebetten, G. (2012). *General Report. Session 4– soil mixing 2 – deep mixing*. International symposium of ISSMGE-TC211. Recent research, advances & execution aspects of ground improvement works (31 May – 1 June 2012). Brussels, Belgium.

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New design of a tapered bored pile For installation in structurally unstable soils

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New designs of bored piles with a tapered shaft shape are proposed. To confirm reduction or absence of the potential impact of additional load (negative) friction forces effectiveness on piles lateral surfaces in structurally unstable soils (fill-up grounds etc.) due to a change in the edge slope angle, laboratory experimental research on models of tapered piles have been conducted; the confirming results have been provided and described in detail in the laboratory experimental research. The impact of the changed slope angle of the edge on the effect of additional load friction forces on the lateral surfaces of experimental piles was demonstrated; the correlation between the change in taper of piles and the decrease in their surface area has been proven.

Keywords: tapered pile, soil subsidence, lateral surface, additional load friction forces

Нова конструкція бурової конусоподібної палі для влаштування в структурно-нестійких ґрунтах

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Будівництво в структурно-нестійких (насипних і т.п.) ґрунтах з потужною просідаючою товщею в умовах сучасного світу стає дедалі актуальним, тому в більшості випадків найбільш економічно ефективним та проектно обґрунтованим рішенням є будівництво на пальових фундаментах. Сучасне будівництво з деяких часів усе частіше при виробництві пальових робіт звертає свою увагу на безударні технології їх виконання. Це підвищує популярність бурових паль. Як відомо, при визначенні несучої здатності бурових паль у структурно-нестійких (насипних і т.п.) ґрунтах у випадках, коли умовна деформація ґрунту навколопальового масиву може перевищувати швидкість осідання пальового фундаменту, слід ураховувати довантажувальні (негативні) сили тертя. Установлено, що зміна кута нахилу грані стовбура палі (конусоподібна форма стовбура) дозволяє забезпечити зниження або відсутність можливого впливу довантажувальних (негативних) сил тертя по бічній поверхі паль у структурно-нестійких (насипних і т.п.) ґрунтах, що, як наслідок, підвищує несучу здатність паль на вдавлюючі навантаження. Запропоновано нові варіанти конструкцій бурових паль з конусоподібною формою стовбура. Для підтвердження ефективності зниження або відсутності можливого впливу довантажувальних (негативних) сил тертя по бічній поверхні палі в структурно-нестійких (насипних і т.п.) ґрунтах за рахунок зміни кута нахилу грані було проведено лабораторні експериментальні дослідження на моделях конусоподібних паль, наведено підтверджуючі результати та детальний опис лабораторних експериментальних досліджень. З'ясовано залежність зміни конусоподібності паль і зменшення площі їх поверхонь, показано вплив зміни кута нахилу грані на дію довантажувальних сил тертя по бічний поверхні експериментальних паль.

Ключові слова: конусоподібна паля, просідання ґрунту, бічна поверхня, довантажувальні сили тертя

Introduction. Collapsible soils cover more than 35% of the territory of Ukraine. By little humidity, collapsible loess soils in their natural state have rather good physical and mechanical properties for construction, but in the presence of moisture, their structural stability decreases, and vertical deformations may occur. Uneven deformation is especially dangerous as it can result in partial or full loss of stability and operational capacity of buildings. It should also be noted that, in the early study phases, only loess soils were deemed collapsible, but the construction practice of recent decades has shown that many non-loess loam soils, fill-up grounds, and loose dusty sands appear to be collapsible soils when in contact with moisture.

Construction in the conditions of structurally unstable soils with thick subsiding mass has become possible primarily due to properly designed foundations, most of which are pile foundations.

When designing buildings and structures on pile foundations in the conditions of structurally unstable soils, additional load (negative) friction forces should be taken into consideration. These emerge if the nominal rate of soil deformation in the near-pile mass exceeds the settlement rate of the pile foundation, which can also occur:

– in the presence of structurally unstable soils in the foundation;

– when constructing pile foundations on very thick mounds by natural consolidation of the soil mass;

– when using pile and raft foundations or applying significant loads to the surface near a pile foundation, which occur due to buildings being constructed nearby on shallow foundations, by land planning when placing fills, when storing materials, installing equipment etc.;

– by soil compaction due to dynamic impacts and increased effective stress by lowering of groundwater level;

– by thawing of frozen soils.

Additional load (negative) friction forces of the soil caused primarily either by additional loading of the surface or due to the presence of soils with specific properties in the foundation have a significant impact on design solutions with regard to pile foundations.

Therefore, it is necessary to conduct complex experimental and theoretical research aimed at determining methods of reduction of additional load friction forces acting on the lateral surfaces of piles in structurally unstable soils, thus facilitating an increase in the bearing capacity of bored piles.

The issues of the development of negative friction forces in pile foundations have been addressed in works of national and foreign researchers: Dalmatov B.I., Lapshin F.K., Rossikhin Yu.V., Grigorian A.A., Zaretskii Yu.K., Morozov V.N., Broma Beng B., Fellenius B.H., Crawford C.B., Endo M., Bjerrum L., Johannessen I.J., Kerisel J., Lee C.J., Bolton M.D. [14, 15] et al.

Except theoretical researches, there are practical ones investigating the forces of negative friction by means of the field methods, that are most reliable. It is possible to mark the use of tenzopile, and also normative and patented methods and methods of friction acting forces determination on the lateral surface of foundations and piles. However, offered field tests with the use of the considered methods or are labour intensive at application of tenzopile or the forces of negative friction determined on the basis of pile tests on the action of the pressing and pulling out loads, and the equality of the soil resistance forces along the lateral surface of the pile is assumed.

Review of the latest research sources and publications. In rough engineering and geological conditions, especially in case of multi-storey and high-rise structures, reinforced concrete auger cast or injection piles (bored piles) [1] are used, including pedestal ones. These piles are installed directly into the soil, which envisages forming a pile shaft by drilling a hole with the target depth using an auger with the required diameter and applying the rotary drilling principle depending on used equipment [2].

There are well-known methods for installing auger cast piles in structurally unstable soils using steel, polyethylene, and other casings, which ensure reduction of additional load friction forces due to lower values of friction against soil in case of the casing material compared to those of a concrete pile surface [2], but such methods are characterized by higher costs and lower antifriction properties [3]. Other known methods applying a so-called «antifriction jacket» [4] make the process of installing bored piles very complicated.

It is known that reinforced concrete driven piles are used that possess a pyramid, trapezoidal, or tapered shaft shape [1, 2, 5], which is also used as a mold for special rammers when installing concrete or reinforced concrete cast-in-place piles (foundations) in a stamped out bed (pit) [1, 4, 6, 7]. Also, for driven piles with constant cross-section, antifriction coating is used along the length of the lateral surface [1]. However, such driven and cast-in-place piles have a limited length (up to ≈ 10 m) and application field, and the effect of reduction of additional load forces along the lateral surface of these piles due to additional squeezing of the soil mass around the piles during their installation is doubtful.

In his thesis [8], Vertynskii O.S. has proposed a design solution of a tapered cast-in-place pile for structurally unstable soils. This pile represents a structure consisting of a metal pipe and outer tapered casing. The structure is intended to be submerged into a hole until the final position is achieved, then concrete is pumped under pressure into the space between the metal pipe and the casing. During this procedure, the casing thrust occurs, and a tapered pile emerges. A drawback of this method is high labor intensity of works aimed at achieving the desired effect.

The research conducted by scientists O. Yeshchenko and D. Cherniavskyi [9, 10] should be mentioned who address issues of the bearing capacity of tapered injection piles in various types of soils; however, the authors do not consider the impact of additional load friction forces on the lateral surface of piles.

Definition of unsolved aspects of the problem. At present, no rational and reliable method for installing tapered bored piles in structurally unstable soils has been proposed, which would reduce the impact of additional load friction forces of soil on their lateral surfaces; research in this direction doesn't exist either.

Research objective. This article offers designs of bored piles created using the patented method [13] in order to ensure reduction or absence of the impact of additional load friction forces of soil on the lateral surface of bored piles in structurally unstable soils (fill-up grounds etc.).

Main material and results. Fig. 1 shows fundamental designs of bored piles installed using the proposed method; the tapered shape of the shaft of the pile *1* is shown, which is created by drilling out a hole using a tapered auger with required geometric parameters ($d_в$ and $d_н$) and depth H and broadening *2* at the end of the pile shaft *1*.

The laboratory experiments were based on the method for determining specifically maximum additional load friction forces of soil on models of piles, which was proposed and implemented by Prof. Samorodov O.V. and Naydenova V.E. and published in works [11, 12], (Fig. 3).

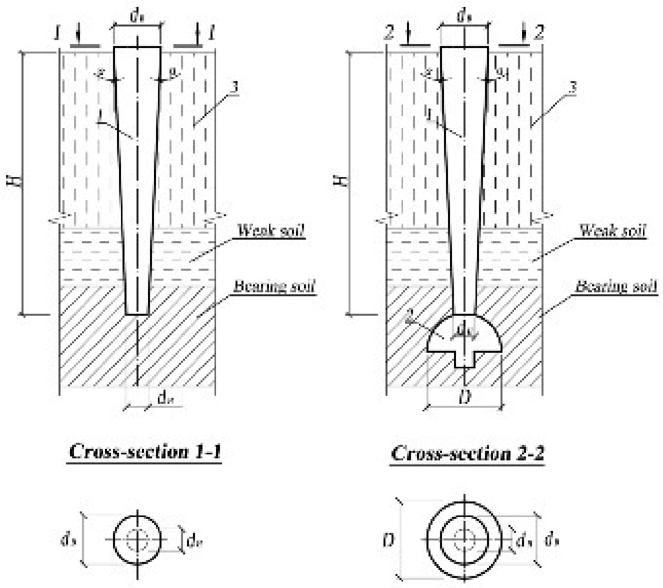

Figure 1 – Fundamental designs of bored piles installed using the proposed method

a) b)

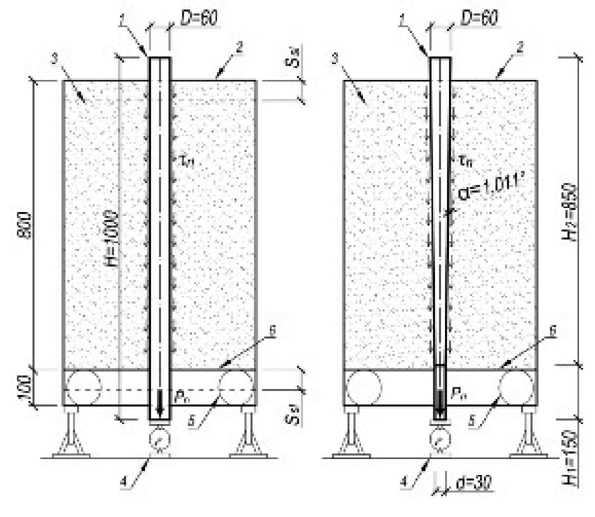

Figure 2 – Installation diagram during the experiment to determine additional load friction forces of soil P_n on the lateral surface of the pile by different angles of slope of the shaft surfaces α:

a) standard cylindrical pile; b) proposed tapered pile;

1 – wooden pile model; 2 – tray; 3 – sand (fine, dry, homogenous $\gamma \approx 15$ kN/m^3, $\varphi = 30^{\circ}$);

4 – dynamometer; 5 – rubber air «cushion»; 6 – partition (particle board)

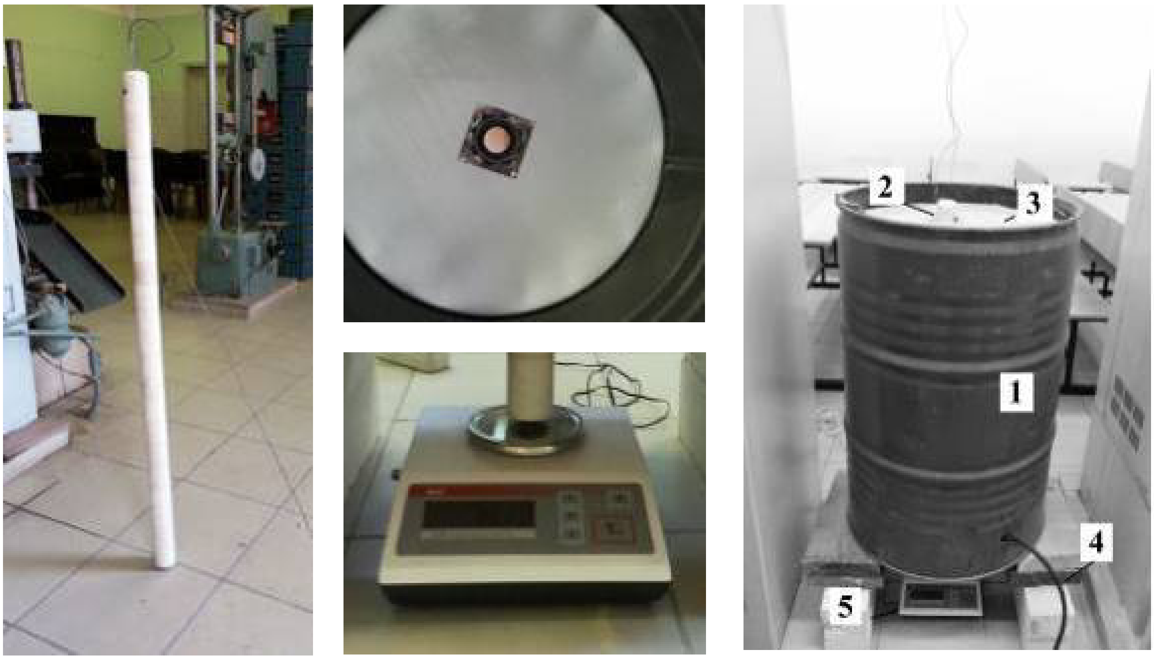

Figure 3 – Laboratory equipment

A specially equipped tray in the form of a metal barrel was used in the capacity of experimental unit (Fig. 3). Its dimensions were as follows: H = 900 mm, Ø = 560 mm. In the lower part of the tray, a double bottom was made with a space between (H_1 = 100 mm), which was filled with a rubber air «cushion».

The laboratory experiment for testing the new pile designs was conducted in a similar manner.

Two models of wooden piles were used, which were wrapped up in sandpaper for better adhesion with sand. One pile had a cylindrical shape, was 1,000 mm in length and 60 mm in diameter; the other had a tapered shape, was 1,000 mm in length and 60 to 30 mm in diameter. The correlation between the numeric values of model piles, such as their length and diameter, was similar to the correlation for actual piles.

Fine, dry, homogenous sand was used as a fill (γ = 15 kN/m^3, φ = 30°). An average dimension of a sand grain was 0.20 mm to 0.25 mm. After the filling up was completed, prior to the start of the first series of experiments, the unit was maintained in the design position for at least 30 minutes. The distance between the pile and the tray walls equaled approximately 250 mm.

Preparation and implementation of the experiment included several stages (see Fig. 2):
– the model pile was installed in the final vertical position by free suspension whereby the lower end of the pile was passed through the entire tray construction through special holes in bottoms, with the pile resting on a dynamometer;
– the tray was filled to the entire height with sandy soil in a «raining» manner;
– settlement of the entire soil mass by the value of s_{sl}=100 mm was imitated by letting the air out of the rubber chamber («cushion»);
– additional weight of the pile due to additional load friction forces of soil P_n on the lateral surface of the pile was registered using a dynamometer.

Table 1 shows the results of laboratory experimental research aimed at determining the maximum additional load friction force of loose soil P_n on the lateral surface of the pile due to a slope of the shaft surfaces.

Table 1 – Results of laboratory research

No.	H, m	H_1, m	H_2, m	D, m	d, m	α, degrees	S (h=0.85 m), m^2	P_n, (10^{-2} kN)
a)	1.0	-	-	0.06	0.06	0	0.16	18.3
b)	1.0	0.85	0.15	0.06	0.03	1.011	0.12	7.5

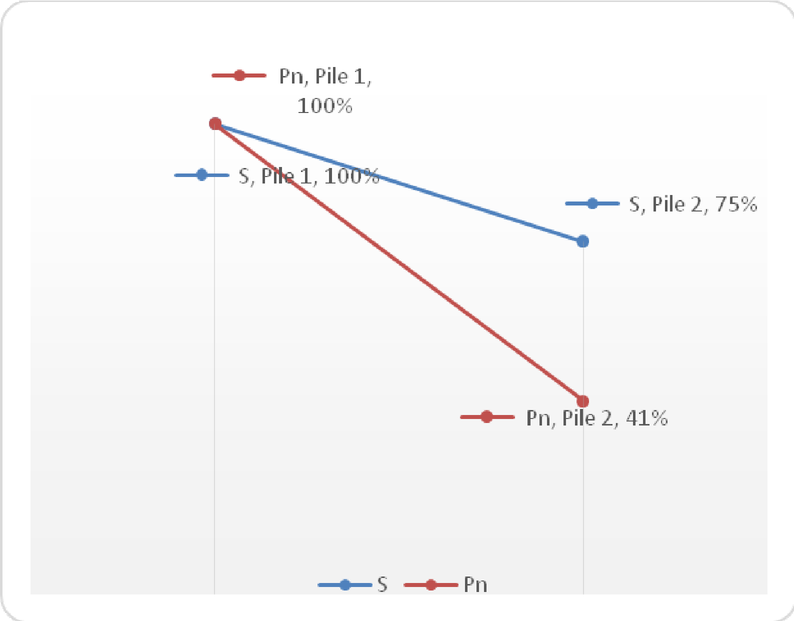

Figure 4 – Comparison of the decrease in values of the surface area of piles (S) and the maximum loading force (Pn) in percentage terms
(pile No. 1 – cylindrical, pile No. 2 – tapered)

Conclusions:

The rational and reliable method for installing tapered bored piles in structurally unstable soils has been proposed.

The results of intermediate laboratory research indicate significant reduction of additional load friction forces of soil on tapered piles compared to the cylindrical shape of their lateral surface, which allows increasing the bearing capacity of piles by pressing loads when designing pile foundations in order to ensure a certain economic effect.

References

1. ДБН В.2.1-10-2009. (2011). *Основи та фундаменти споруд. Основні положення проектування. Зміна № 1.* Київ: Мінрегіонбуд України.

2. Смородинов, М.И. (Ред.), Егоров, А.И., Губанова, Е.М. и др. (1988). *Свайные работы. Справочник строителя.* Москва: Стройиздат.

3. Ермошин, П.М. (1982). *Устройство буронабивных свай*. Москва: Стройиздат.

4. Снісаренко, В.І., Гембарський, Л.В. & Щерба, М.О. (2011). *Спосіб улаштування паль з «антифрикційною сорочкою».* Патент України на корисну модель 57669. Київ: Державне пантентне відомство України.

5. Горбунов-Посадов, М.И., Ильичев, В.А., Крутов, В.И., Сорочан, Е.А. (Ред.), Трофименков, Ю.Г. (Ред.) и др. (1985). *Основания, фундаменты и подземные сооружения*. Москва: Стройиздат.

6. Зоценко, М.Л., Винников, Ю.Л., Павліков, А.М. та ін. (2014). *Посібник з проектування та влаштування паль у пробитих свердловинах*. ПолтНТУ, ДП НДІБК.

7. НИИОСП им. Н. М. Герсеванова. (1986). *Пособие по производству работ при устройстве оснований и фундаментов (к СНиП 3.02.01-83)*. Москва: Стройиздат.

8. Вертынский, О.С. (2007). *Разработка и экспериментально-теоретическое обоснование новых конструкций набивных свай.* (Автореф. дис. канд. техн. наук.). ВГАСА, Волгоград.

9. Ещенко, О.Ю., Чернявский, Д.А. (2015). Исследование работы буроинъекционных конических свай в глинистых грунтах при действии вертикальных статических нагрузок. *Инженерный вестник Дона,* 31, Взято з http://ivdon.ru.

10. Ещенко, О.Ю., Чернявский, Д.А., Каженцев, Н.Г. (2016). Влияние модуля деформации глинистого грунта на несущую способность одиночных конических и цилиндрических буроинъекционных свай. *Материалы VIII Всероссийской конференции «Современные технологии в строительстве: наука и практика»,* 6, Взято з : http://sbornikstf.pstu.ru.

11. Самородов, А.В. (2017). *Проектирование эффективных комбинированных свайных и плитных фундаментов многоэтажных зданий: монография*. Харків: Типография Мадрид.

12. Naydenova, V.E.(2018). Laboratory experimental research of loading forces development acting on the side surface of the piles. *Academic Journal. Series: Industrial Machine Building, Civil Engineering,* 1(50), 174-180. doi:10.26906/znp.2018.50.1073.

13. Самородов. О.В., Убийвовк, А.В., Найдьонова, В.Є., Купрейчик, А.Ю. (2018). *Спосіб улаштування бурових паль у структурно-нестійких ґрунтах.* Заявка на винахід України 2018 00812. Київ: Державне пантентне відомство України.

14.Robinsky, E.I., Sagar, W.L & Morrison, C.F. (1964). Effect of Shape and Volume on the Capacity of Model Piles in Sand. *Canadian Geotechnical Journal*, 1(4), 189-204. https://doi.org/10.1139/t64-015

15. Hesahm El Naggar, M. (1998)/ Experimental study of axial behavior of tapered piles. *Canadian Geotechnical Journal*, 35(1), 641-654. https://doi.org/10.1139/t98-033.

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Review of design solutions for nominally strip (continuous) foundations

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The paper reviews the existing designs of nominally strip (continuous) foundations that are offered in foundation engineering, that due to their shape (configuration) of contact with the base enable more rationally to design foundations for continuous structures of buildings and facilities. The advantages and disadvantages of continuous foundations different types are shown, and some calculation methods and methodologies when calculating these foundations in interaction with soil bases are given.)

Keywords: soil base, foundation, conditionally banded, shape of bottom, longitudinal cutout, efficiency

Аналіз конструктивних рішень умовно стрічкових (протяжних) фундаментів

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Проаналізовано існуючі конструкції умовно стрічкових (протяжних) фундаментів, які за рахунок своєї форми (конфігурації) контакту з основою дозволяють більш раціонально проектувати фундаменти протяжних конструкцій будівель і споруд. Показано переваги та недоліки різних типів протяжних фундаментів, а також наведені деякі методи й методики розрахунку таких фундаментів при взаємодії з грунтовими основами. Встановлено, що розробка нових конструктивних рішень умовно стрічкових фундаментів та удосконалення методик їх розрахунку враховує тільки вертикальні навантаження на фундаменти. Однак, існує клас споруд, типа масивних підпірних стін, що сприймають значні моментні навантаження по підошві, які потребують розробки ефективної конструкції фундаментної частини із забезпеченням розрахункового опору грунту основи. Запропоновано запатентовану комбіновану конструкцію стрічкового фундамент з поздовжнім вирізом по підошві, що складається з фундаментної частини шириною (2b+a) та вирізу шириною a і висотою Δ, що заповнюється низькомодульним матеріалом, наприклад, пінопластом. З одного боку, така конструкція фундаменту ефективно сприймає ексцентричні навантаження у порівнянні з суцільною формою підошвою з дотриманням нормативних вимог за крайовими тисками на основу. З другого боку, конструкція дозволяє збільшити розрахунковий опір грунту основи фундаменту з вирізом за рахунок заповнення порожнини вирізу низькомодульним матеріалом, за рахунок чого відбувається «сприятливий» перерозподіл напружень в основі фундаменту з вирізом у порівнянні із суцільним фундаментом при прийнятті будь-якого критерію розвитку зон граничної рівноваги під фундаментом. Тому, розрахунковий опір грунту запропонованого стрічкового фундаменту може бути до 2-х разів більше у порівнянні із суцільним фундаментом шириною 2b.

Ключові слова: ґрунтова основа, фундамент, умовно стрічковий, форма підошви, поздовжній виріз, ефективність

Introduction. Increasing the economic efficiency of foundation construction, which cost can in some cases amount to 40% of buildings construction total cost, is currently a priority area. Cast in-situ and precast strip foundations for walls are the most common type of shallow foundations, which is used in residential construction, although types of nominally strip (continuous) foundations with various shapes of bottoms in contact with the base have been provided recently. Using other shapes of continuous foundations bottoms alters favourably the stress-strain state of the soil base, that is, from the plane to the space ("plane-space") state; hence, they are more rational than strip foundation classical shape.

Review of recent research and publications. A number of researchers such as V. Alekseev, L. Anshin, D. Arkhipov, B. Barykin, V. Ermashov, M. Fidarov, R. Furunzhiev, E. Livshits, F. Lyalin, E. Sorochan, P. Poyta, R. Mangushev, A. Pilyagin, E. Neiburg, A. Razoryonov, V. Solomin, V. Tarshish, E. Vinokurov, G. Skibin, S.Yevtushenko, V. Pankov, T. Krakhmalny, Yu. Tugayenko, N. Kiselev and many others have devoted their work to developing methods of calculation and designing non-conventional continuous foundations (discontinuous strip foundations, adjacent foundations, etc.), and optimizing foundation designs on the whole. Designs of nominally strip (continuous) foundations have been intensively improved and investigated up to the present time, as evidenced by recent theses defended abroad [1, 2, 26-30]. Currently, using discontinuous strip foundations enables, under otherwise equal conditions, to increase the allowable pressure on the soil base up to 1.3 times due to the change of the stress-strain state from the plane to the space state, which was introduced back in SNiP 2.02.01-83* «Buildings and structures» foundation has been implented in force according to date in the national standards DBN [3].

Uninvestigated parts of general matters defining. Development of new design solutions for nominally strip foundations, experimental and theoretical research of their combined behaviour with soil bases, and improvement of their calculation and designing methods are an area of current interest in soil mechanics and foundation engineering.

Research objective is to analize nominally strip (continuous) foundations design solutions .

Main content and findings. In the Research Institute of Bases and Underground Structures named after N.M. Gersevanov, the first attempt has been made to change the traditional way of transferring load in a way that substantially improves the service behaviour of the foundation and the base itself. It has been experimentally confirmed that the ultimate load on base soils increases 1.5 times. Therefore, it is possible to transfer more load on a discontinuous foundation, under otherwise equal conditions, or, using the principle of calculation from the second limit state, reduce the area of the foundation bottom and hence its materials intensity. This principle was implemented in 1954 under the guidance of E. Sorochan [4] when developing precast discontinuous strip foundations (Fig. 1) which were first used in Moscow.

Experimentally, the behaviour of discontinuous strip foundations started to be studied for the first time by E. Sorochan [4], however, studies dealt with the linear phase of base behaviour . The limit stress state of discontinuous strip foundations base soil was particularly covered by studies of M. Fidarov [5]. The proposed theory of discontinuous foundations and bases combined behaviour studied by M. Fidarov is based on the solutions to soil limit stress-strain state and soils pressure over the roofs of underground mining have been studied by M. Protodiakonov. This theory considers the emergence of an arching effect in the soil between the pad blocks, due to them foundation bottom can be considered as continuous. In his doctoral thesis, P.Poyta [6] provides considerable theoretical and experimental studies on the combined behaviour of base soils and discontinuous foundations, involving the theory of arching effect occurrence, thus, the simple formula can be derived to determine the space between the slabs in a discontinuous foundation (see Fig. 1):

$$a = b\ sin\varphi. \quad (1)$$

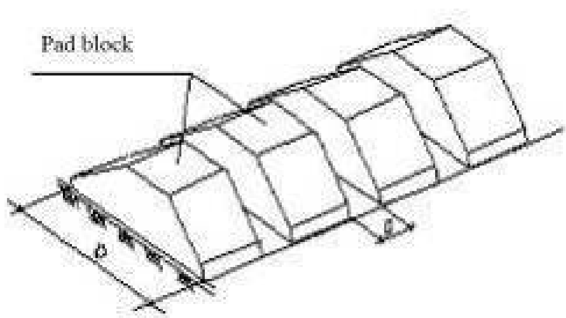


Figure 1 – Precast discontinuous foundation

To date, the current DBN [3] standardize the use of structures of discontinuous strip foundations, including those with angular cut-outs, which enables, under otherwise equal conditions, to increase the allowable pressure (design resistance R) on the soil base up to 1.3 times, which was already introduced in the Soviet regulatory documents.

The design solution close to discontinuous foundations in terms of the behaviour peculiarities with the base is sleeper foundations (Fig. 2), which were considered in depth in the works by Yu. Tugayenko and S. Kushchak [7], P. Poyta [6] and V. Pankov [8].

Later on, strip foundations structures improvement was aimed at developing pad blocks with rectangular cut-outs in the corners, thus, pad blocks area is 12% smaller than typical blocks area (see Fig. 3).

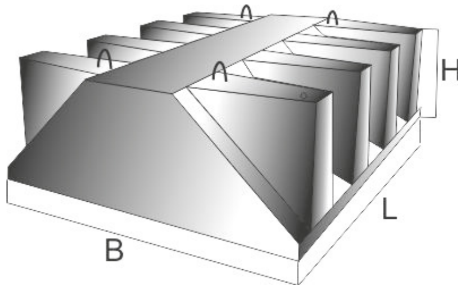


Figure 2 – Sleeper foundation

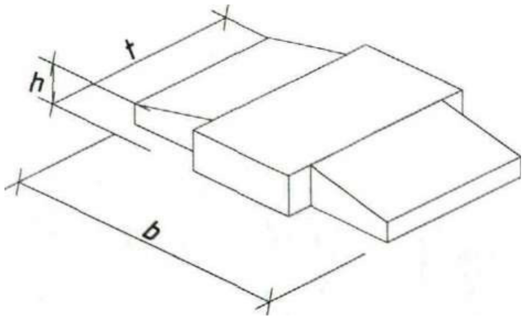


Figure 3 – Foundation pad block with angular cut-outs

The work of V. Ermashov [9] is devoted to studying foundation slabs with angular cut-outs shape influence ontheir bases stress-strain state. The research was experimental in a sand tray measuring 6.0 by 5.0 m in size with a height of 5.5 m. The variables included foundation stiffness, contact surface relative size, and reinforcement percentage. Consequently, it was found that the arrangement of angular cut-outs in slabs results in the formation of limit state local zones in the soil through concentrating the contact stresses in the central part of foundation bottom. It enables to reduce the bending moment in slab critical section and foundation materials intensity. Based on the conducted research, reinforced concrete slabs for strip foundations with a rational contour of the contact surface were developed, which use reduces the concrete consumption to 18 to 20% and steel consumption to 15 to 18% compared with solid slabs.

A number of researchers from South Russian State Technical University (Novocherkassk Polytechnic Institute) (Novocherkassk Polytechnic Institute) including G. Skibin, S.Yevtushenko, D. Arkhipov, T. Krakhmalny [1, 10-13] and others are deeply involved in developing continuous foundations for walls and retaining structures with various options of moving apart and turning pad blocks, and the broken contour of the base slab boundary zone (see Fig. 4).

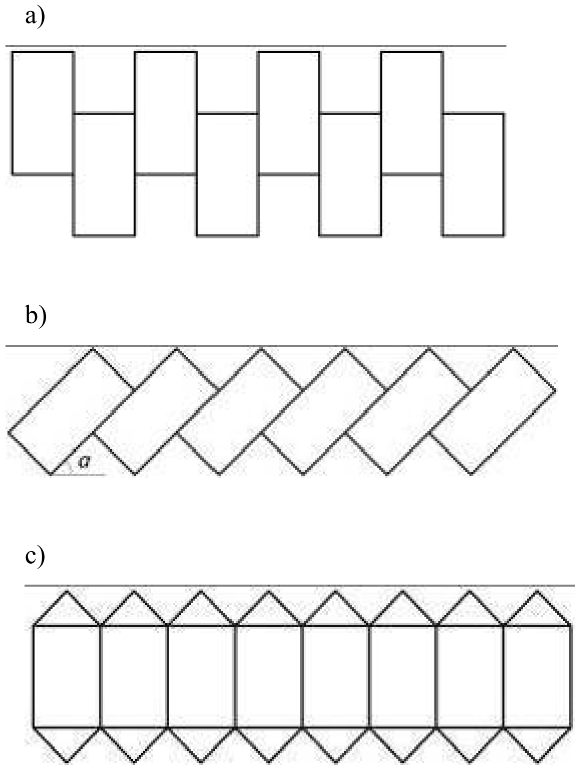


Figure 4 – Layouts of continuous foundations:
a) with blocks moved apart in the transverse direction;
b) with blocks turned at the angle α;
c) with the broken contour of the boundary zone.

The work of T.A. Krakhmalny [1], based on numerous laboratory experimental studies, provides foundations various layouts comparative analysis (Fig. 4) and dependence evaluation of base bearing capacity on bottom shape change. It is furthermore concluded that with increase of foundation model perimeter ratio to its area (ξ), the critical load ΔP to the base and its bearing capacity increase. Figure 5 shows the graph of change in model bearing capacity (ΔP) against foundation perimeter ratio to its area (ξ).

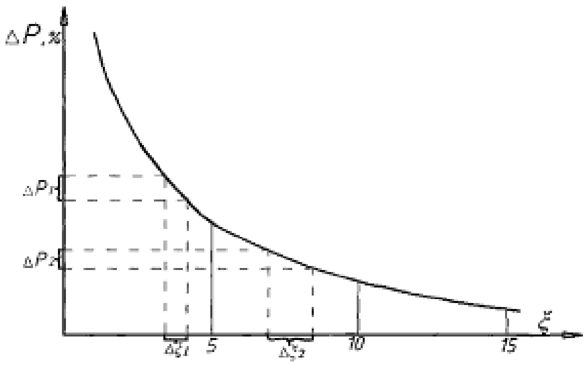


Figure 5 – The graph of change in bearing capacity of the model against foundation perimeter ratio to its contact area

According to the graph, change in the perimeter at ξ < 5 results in a greater increase in the bearing capacity ΔP_1 than the increase in the perimeter at 5 < ξ < 10, which gives an increase in the bearing capacity by the value ΔP_2.

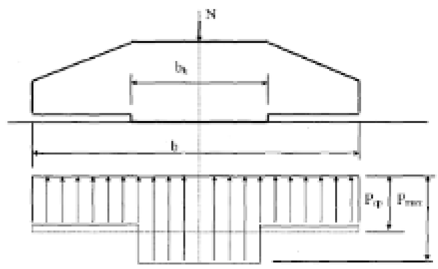
Other conclusions of the authors have been considered, related to calculation methods peculiarities for bases of such foundations (see Fig. 4):

- performed calculations show the possibility of determining the design resistance of strip foundations base with the rotation of square and rectangular supporting blocks and base slabs broken contour according to SNiP (DBN) [3; 14];
- the design resistance of sandy soil *R* is 4.0 to 5.0 times lower than the actual range of the linear dependence of the foundation settlement on the load. Hence, the regulations [3, 14] greatly reduce the allowable load. To determine foundation size on a sandy base it is more appropriate to perform calculation from the first limit state using base ultimate resistance N_u, which more closely corresponds to the limit of sand base linear deformability than the value *R* does. The same conclusion was made in the works by V.A. Ilichev , A.B. Fadeev, and V.A. Lukin [15, 16], where calculation methods resilts are compared with Eurocode 7 [17].

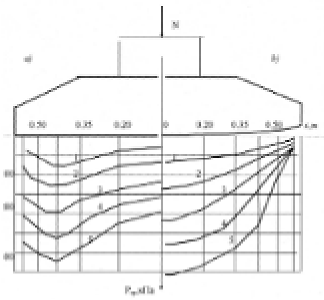
Concepts for adjustable distribution of reaction pressure were developed in the works by E.A. Sorochan [18, 19], who suggested accumulating the contact stresses under the centre of strip foundations by arranging nonuniform stiffness in the centre by means of concrete, and along the perimeter by means of sand. Thus, foundation behaviour relative to base changes considerably: on the one hand, the bending moments in the overhanging lengths of slab decrease, on the other hand, the bearing capacity of base soil increases due to “closure” of plastic shear zones under the centre. It is possible to adjust the distribution of contact stresses in other ways: by laying plastic foam inserts under strip foundation edge or by producing precast blocks with concrete protrusions under centre and unsupported console overhangs (Fig. 6, a). Such approaches to adjusting reaction pressure under the foundations are intensively developed by Ya.O. Pronozin and N.Yu. Kiselev [2] when developing and studying the behaviour of slab foundations with a compensatory layer.

M.S. Gritsuk [20] has theoretically and experimentally proved the efficiency of strip foundations from precast slabs with convex curvilinear or trapezium-shaped bottoms (Fig. 6, b), which provides parabolic distribution of reaction pressures with zero values near the edges. V.F. Bai and A.N. Kraev [21] have investigated strip foundations of compressed curvilinear sand masses reinforced along the contour (Fig. 6, c).

a)



b)



c)

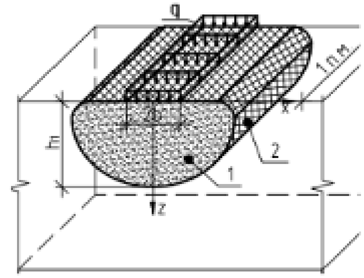


Figure 6 – The influence of the shape of a foundation on reaction pressures and settlements:
a) aoundation with an intermediate bed;
b) aoundation with a convex bottom;
c) aoundation with a curvilinear sand bed

A new development in the field of foundation engineering is new designs of strip foundations with longitudinal cut-outs on the bottom, which are highlighted in the works by I.Ya. Luchkovsky and O.V. Samorodov [22 – 24], where the authors offer the optimum shapes of eccentrically loaded strip foundations with a longitudinal cut-out bottoms, which can also be used for massive retaining walls (Fig. 7).

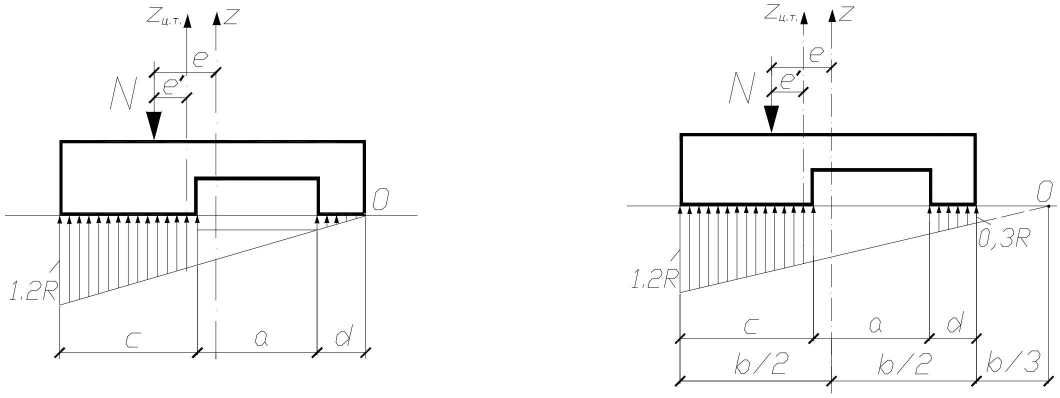


Figure 7 -- Eccentrically loaded strip foundations of retaining walls with asymmetrical cut-outs on the bottom

Foundations with longitudinal asymmetrical or symmetrical cut-outs on the bottom enable to make full use of soil base strength properties, while observing the regulatory edge constraints (Fig. 7), and the optimum geometric parameters of foundation bottom at given efforts N and $M = Ne$, and at base soil design resistance R are equal to:

- for the design with limiting edge pressures on the base $p_{max} \leq 1.2R$ and $p_{min} \geq 0$ (Fig. 7, a):

$$b = \left(e + 0{,}495m\right)\left[1 + \sqrt{1 + \frac{1}{5}\left(\frac{m}{e + 0{,}495m}\right)^2}\right], \qquad (2)$$

where

$$\bar{a} = 0{,}804\left[\sqrt{1 + 1{,}355\left(3 - \bar{N}\right)} - 1\right].;$$

$$m = N / R\ ;$$

$$\bar{N} = 5N / bR\ ,$$

and for the case of a centrally located cut-out at $c=d$:

$$b = \left(e + 0{,}833m\right)\left[1 + \sqrt{1 - \frac{1}{3}\left(\frac{1{,}666m}{e + 0{,}833m}\right)^2}\right], \qquad (3)$$

where

$$\bar{a} = \frac{a}{b} = 1 - \frac{\bar{N}}{3}\ ;$$

- for the design with limiting the edge pressures on the base $p_{max} \leq 1.2R$ and $p_{min}/p_{max} \geq 0.25$ (Fig. 7, b):

$$b = \frac{e + 1{,}33m + \sqrt{\left(e + 1{,}33m\right)^2 - 2m^2}}{1{,}54}, \qquad (4)$$

where

$$\bar{a} = \sqrt{1{,}78 - 0{,}45\bar{N}} - 0{,}29\ ,$$

and for the case of a centrally located cut-out at $c=d$:

$$b = \frac{e + 0{,}4m + \sqrt{\left(e + 0{,}4m\right)^2 - 0{,}213m^2}}{0{,}6}, \qquad (5)$$

where

$$\bar{a} = \frac{a}{b} = 1 - \frac{\bar{N}}{3{,}75}\ .$$

However, preliminary calculations show that in the case of an empty space inside cut-out cavity a soil design resistance decreases sharply compared to the continuous bottom; therefore, there is provided a patented design of strip foundation with longitudinal cut-out (Fig. 8) [22] that consists of foundation part *1* of ($2b + a$) in width and cut-out of a in width and Δ in height, filled with low-modulus material *2* such as foamed plastic. Furthermore, it is possible to significantly simplify the technology of such foundations arrangement due to "low-modulus insert" location in the concrete bed *3* with thickness Δ under the foundation (see Fig. 8).

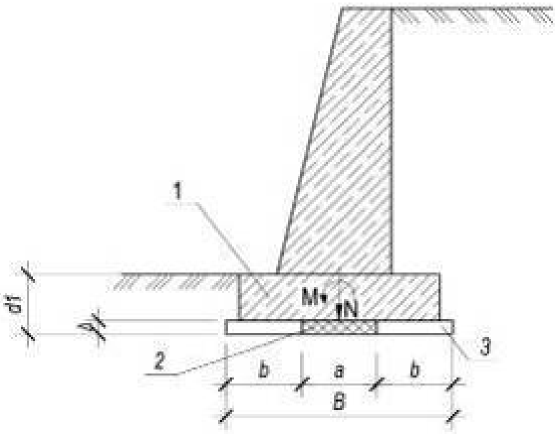


Figure 8 – Mass retaining wall with a longitudinal centrally located cut-out on the bottom

In this case, in order to determine soil design resistance R, it is suggested considering the calculation model of interaction between the foundation and the soil base, which is given in Fig. 9, where pressure equal to natural pressure $\gamma\ d_1$ outside foundation is transferred within the width of cut-out a when loading the foundation.

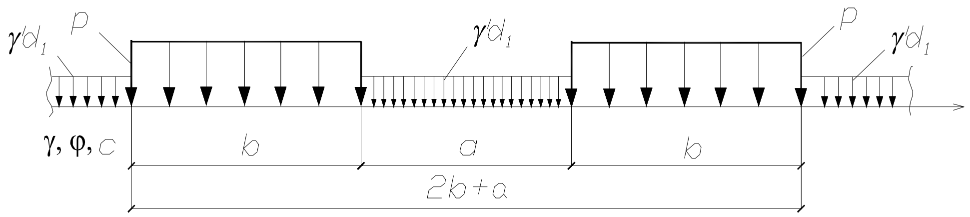


Figure 9 – Calculation model of interaction between foundation and soil base

The pressure inside the longitudinal cut-out of Δ in height is transferred by filling the cavity with a low-modulus material, such as foamed plastic, with a modulus of elasticity E, which is equal to

$$E = \frac{\Delta}{s}\gamma\ d_1\ , \tag{6}$$

where s is forecasted settlement of foundation, m;
Δ is the height of the cut-out, m;
γ is soil specific gravity above foundation bottom, kN/m^3;
d_1 is foundation laying debth, m,
or, alternatively, by performing a cut-out of a in width and Δ in height, which is equal to

$$\Delta = \frac{E}{\gamma\ d_1} s\ . \tag{7}$$

In this case, foundation soil design resistance with the cut-out R_{2b+a} of ($2b+a$) in width is equal to:

$$R_{2b+a} = R_b \cdot k_d\ , \tag{8}$$

where R_b is foundation base soil design resistance of b in width to be determined according to the standard formula E.1 [3], subject to taking any criterion for boundary equilibrium zones development under the foundation;
k_d is the coefficient to be determined according to the graphs in Fig. 10 and obtained based on our analytical studies in the work [25].

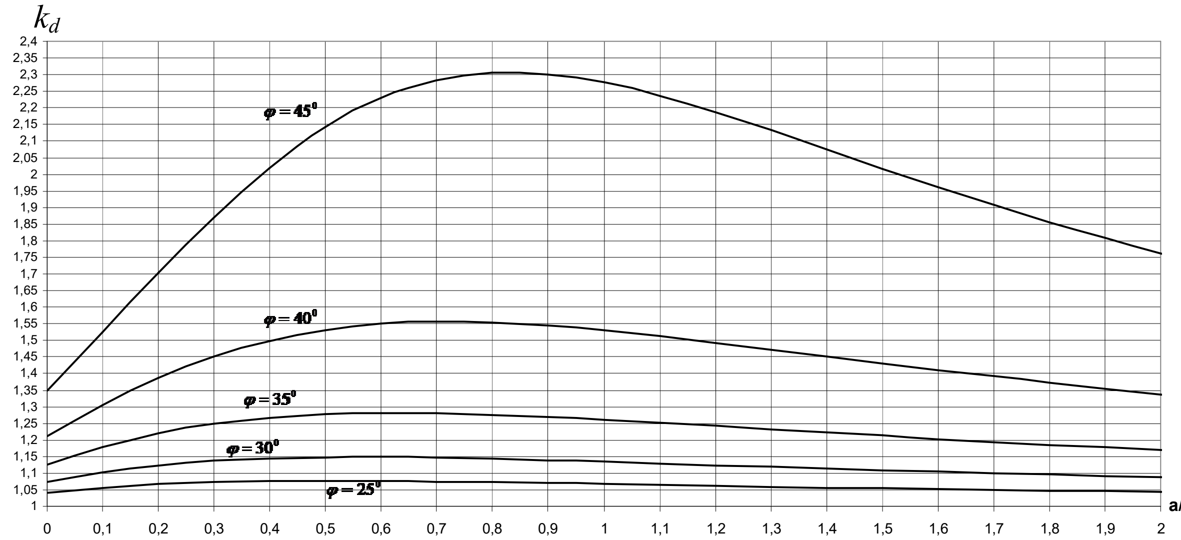


Figure 10 – Graphs of the coefficient kd against the relative width of the cut-out a/b

Conclusions. The analysis has been performed of the existing non-conventional nominally strip foundations such as discontinuous foundations, adjacent foundations, foundations with pad blocks turned at an angle, foundations with cut-outs, foundations with moving apart on the bottom and others that are more rational than the classical shape of a strip foundation, which enables to designefficient foundations for continuous structures of buildings and facilities.

References

1. Крахмальный, Т.А. (2010). *Исследования работы песчаного основания ленточного фундамента с ломаным очертанием опорной плиты.* (Дис. канд. техн. наук). СПбГАСА, Санкт-Петербург.
2. Киселев, Н.Ю. (2017). *Работа плитных фундаментов с компенсирующим слоем на грунтоваом основании.* (Дис. канд. техн. наук). Тюменский индустриальный институт, Тюмень, 2017. – 146 с.
3. ДБН В.2.1-10-2009. (2009). *Основи та фундаменти споруд*. Київ: Мінрегіонбуд України.
4. Сорочан, Е.А. (1959). Исследования вопросов применения прерывистых фундаментов. *Труды НИИОСП*, 40, 28-45.

5. Фидаров, М.И. (1986). *Проектирование и возведение прерывистых фундаментов*. Москва: Стройиздат.
6. Пойта, П.С. (1985). *Совместная работа прерывистых ленточных фундаментов с продольным расположением разрыва и намывных песчаных оснований (для региона Белорусской ССР)*. (Дис.канд. техн. наук). БГТУ, Брест.
7. Тугаенко, Ю.Ф. Кущак, С.И. (1986). Деформации в основаниях фундаментов из шпальных элементов. *Основания, фундаменты и механика грунтов,* 2, 9-11.
8. Панков, В.К. (1971). *Исследование работы составных фундаментов из траверс и нестыкуемых подкладных плит.* (Автореф. дис. канд. техн. наук), СТИ, Свердловск.
9. Ермашов, В.П. (1985). Влияние формы фундамента на распределение нормальных контактных напряжений. *Основания, фундаменты и механика грунтов,* 2, 16-17.
10. Евтушенко, С.И. & Крахмальный, Т.А. (2008). Разработка новых конструкций протяженных фундаментов, эффективно использующих несущую способность основания. *Вестник Волгоградского гос. архитектурно-строительного университета. Серия: Строительство и архитектура,* 10(29), 122-127.
11. Евтушенко, С.И. & Крахмальный, Т.А. (2011). Исследование работы ленточных фундаментов со сложной конфигурацией подошвы. *Основания, фундаменты и механика грунтов,* 3, 14-17.
12. Евтушенко, С.И. & Крахмальный, Т.А. (2011). *Экспериментальные исследования работы новых конструкций ленточных фундаментов с ломаным очертанием краевой зоны на песчаном основании.* Новочеркасск: ЛИК.
13. Крахмальный, Т.А. & Евтушенко, С.И. (2008). Разработка новых конструкций протяженных фундаментов, эффективно использующих несущую способность основания. *Вестник Волгоградского гос. архитектурно-строительного университета. Серия: Строительство и архитектура,* 10(29), 122-127.
14. СНиП 2.02.01-83. (1985). *Основания зданий и сооружений. Нормы проектирования*. Москва: Стройиздат.
15. Ильичев, В.А. & Фадеев, А.Б. (2002). Европейские правила геотехнического проектирования. *Основания, фундаменты и механика грунтов*, 6, 25-29.
16. Фадеев, А.Б. & Лукин, В.А. (2006). Сопоставление методик СНиП и ЕК7 при расчете оснований фундаментов мелкого заложения. *Основания, фундаменты и механика грунтов,* 4. 19-24.
17. EN 1997-1. Eurocode 7. *Geotechnical design. Part 1: General rules*. CEN/TC 250. 2003. ICS: 93.020; 91.080.01.
18. Сорочан, Е.А., Быцутенко, О.В. & Лиховцев, В.М. (1991). Фундаменты на промежуточной подготовке переменной жесткости. *Основания, фундаменты и механика грунтов,* 1, 7-8.
19. Сорочан, Е.А. & Абуханов, А.З. (1988). Экспериментальные исследования напряженного состояния фундамента с промежуточной подготовкой на песчаном основании. *Исследование и расчет оснований и фундаментов при действии статических и динамических нагрузок: Межвузовский сборник*, 79-85.
20. Грицук, М.С. & Игнатюк, В.Ю. (1978). Напряженно-деформированное состояние фундаментных блоков с криволинейной поверхностью опирания. *Известия вузов. Строительство и архитектура,* 10. 31-33.
21. Бай, В.Ф. & Краев, А.Н. (2014). Исследование работы песчаной армированной по контуру подушки с криволинейной подошвой в условиях слабых глинистых грунтов. *Вестник гражданских инженеров*, 3(44). 107-110.
22. Самородов, О.В., Лучковський, І.Я., Конюхов, О.В. & Кротов, О.В. (2013). *Стрічковий фундамент з повздовжнім вирізом по підошві.* Патент України на винахід 100647, Київ: Державне патентне відомствоУраїни.
23. Самородов, А.В. (2005). *Внецентренно нагруженные фундаменты с вырезами по подошве*. (Дис. канд. техн. наук). Приднепровская государственная академия строительства и архитектуры, Днепр.
24. Самородов, А.В., Лучковский, И.Я. & Конюхов, А.В. (2010). Фундаменты с асимметричными вырезами по подошве при действии превалирующих односторонних моментных нагрузок. *Науковий вісник будівництва*, 61, 140-145.
25. Самородов, А.В., Лучковский, И.Я. & Кротов, О.В. (2012). Ленточный фундамент с продольным вырезом по подошве. *Збірник наукових праць. Серія: Галузеве машинобудування, будівництво,* 4(34)-2, 201-204.
26. Patra, C.R., Das, B.M., Bhoi, M. & Shin, E.C. (2005). Eccentrically loaded strip foundation on geogrid-reinforced sand. *Geotextiles and Geomembranes*, 24(4), 254-259. doi:10.1016/j.geotexmem.2005.12.001
27. Das, B.M. & Shin, E.C. (1996). Laboratory model tests for cyclic load-induced settlement of a strip foundation on a clayey soil. *Geotechnical & Geological Engineering*, 14(3), 254-259. Retrieved from https://link.springer.com.
28. Griffiths, D.V., Fenton, G.A. & Manoharan, N. (2002). Bearing Capacity of Rough Rigid Strip Footing on Cohesive Soil: Probabilistic Study. *Journal of Geotechnical and Geoenvironmental Engineering*, 128(9), 145-151. doi:10.1061/(ASCE)1090-0241(2002)128:9(743)
29. Yin, Jian-Hua, Wang, Yu-Jie & Selvadurai, A.P.S. (2001). Influence of Nonassociativity on the Bearing Capacity of a Strip Footing. *Journal of Geotechnical and Geoenvironmental Engineering*, 127(11), 81-89. doi:10.1061/(ASCE)1090-0241(2001)127:11(985).
30. Salih Keskin, M. & Laman, M. (2013). Model studies of bearing capacity of strip footing on sand slope. KSCE Journal of Civil Engineering, 17(4), 699-711. doi:10.1007/s12205-013-0406-x.

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Influence of residential complex construction on the condition of the sewage collector and the underground water supply system in Kyiv on A. Barbyusa st.

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The article presents the design features of a residential complex located in the Barbyus street in Kyiv. The main city sewer and water supply is the basis of the residential designed complex. It is necessary to consider the possible impact of new residential complex construction on the sewer and water supply. To assess this effect, the stress-strain state in the soil around the collector and water supply system has been analyzed. Conclusions are drawn concerning the technical condition of the sewer and water supply system after residential complex construction and its further operation.

Keywords: collector, water supply, stress-strain state, building, designing, construction

Вплив будівництва житлового комплексу по вул. А. Барбюса в Києві на стан каналізаційного колектора та підземного водогону

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Наведено особливості проектування житлового комплексу, розташованого по вул. А. Барбюса в м. Києві. Зазначено, що особливістю проектування житлового комплексу є те, що в ґрунтовій основі комплексу, котрий проектується, на глибині 39,0...42,0 м (від поверхні землі) проходить головний міський каналізаційний колектор, а на глибині 55,5 м – напірний водогін Деснянського водопроводу. При проектуванні житлового комплексу було передбачено конструктивні заходи у вигляді відсічних стінок зі зменшення впливу нового будівництва на каналізаційний колектор та напірний водогін. Необхідно було враховувати можливий вплив нового будівництва житлового комплексу на каналізаційний колектор і водогін з урахуванням ужитих конструктивних заходів та за необхідності надати рекомендації щодо зменшення негативного впливу. Для оцінювання цього впливу було розроблено розрахункову модель системи «будівля – фундамент – ґрунтова основа», в якій оцінювалася зміна напружено-деформованого стану (далі – НДС) у ґрунті навколо колектора і водогону, а також у несучих конструкціях колектора й водогону, та відповідні деформації й зусилля, що створюють НДС і виникають у конструктивних елементах колектора, водогону та в ґрунті. Зроблено висновки щодо зміни технічного стану каналізаційного колектора та водогону після будівництва житлового комплексу та його подальшої експлуатації і наведено рекомендації щодо виключення можливого виникнення аварійної ситуації на колекторі та/або водогоні й необхідності оцінювання впливу будівництва житлового комплексу на існуючу оточуючу забудову.

Ключові слова: колектор, водогін, напружено-деформований стан, будівля, проектування, будівництво

Introduction. Residential complex which is being designed is located in the Barbyus st. in Pecherskyi District of Kyiv and its size is 138×108 m. For the moment the most of the city territories with flat relief, convenient geotechnical conditions, and absence of geohazards and other unfavorable conditions are built up. Therefore, the engineering-geological risk of developing such territories increases as well as the risk of emergencies. It caused a decreasing in free areas with favorable conditions for construction in Kyiv and necessitated new construction objects designing and building in the areas with complex geotechnical conditions, the landslide and landslide-hazard areas, zones affected by geohazards and other hazardous engineering risks.

Beneath the construction site of a residential complex, which is built, at 98.1 m level (at depths of 39 … 42 m) the main city sewer collector is located along with the pressure water pipe (at depths of 55.5 m) that belongs to Desnianskyi water supply system.

Analysis of recent sources and publications. The issue of buildings and structures construction in the convenient and complex geotechnical conditions as well as in the zones of geohazards influence, the bases reinforcement methods, structural solutions of protective structures, and other hazardous engineering risks are addressed by a set of regulations and standards, that is: DBN V.2.1-10 and its Amendments № 1 and № 2, DBN V.1.1-45, DSTU-N B V.1.1-39, DSTU-N B V.1.1-40, DSTU-N B V.1.1-44 [1-7].

Identification of unsolved issues in the problem under consideration. The existing set of standards and regulations on designing in convenient and complex geotechnical conditions, as well as in the zones of geohazards influence, and other hazardous engineering risks does not completely provide the opportunity of buildings and structures design at the modern level.

Introduction of new DBN V.2.1-10:201X «Bases and foundations of buildings and structures. Basic provisions» and standards in its development enable to update the set of regulations on buildings and structures design on landslide and landslide hazardous areas, territories with convenient and complex geotechnical conditions, in zones of geohazards influence including seismic impacts.

Problem statement. Improvement of existing state construction codes [1 – 3] in the field of buildings and structures bases and foundations design, considering the modern principles of their design through development of DBN V.2.1-10:201X enables to takethe reasonable decisions on calculations and to justify technical decisions on areas with dangerous engineering risks. These norms are an integral part of a set of standards and regulations that establish mandatory requirements for buildings and structures design on different constructions sites with different geotechnical conditions and that are intended for use at all stages of the life cycle of construction objects.

Main material and results. In the draft of DBN V.2.1-10:201X the basic provisions are shown as well as requirements for all types of buildings and structures bases and foundations design, construction and reconstruction and classes of consequences (responsibilities), the basic requirements for bases engineering treatment design are contained as well as the composition of engineering surveys, environmental requirements for the buildings and structures bases and foundations design. The article presents the use of the standards and regulations set when designing a specific construction object, complicated by hazardous engineering risks, connected with the location of an underground sewer collector in the development spot (at depths of 39...42 from the surface) and pressure water pipe (at depth of 55 m from the surface).

The sewer collector is located under the buildings A and B, and the water pipe runs directly under the building A (Figure 1).

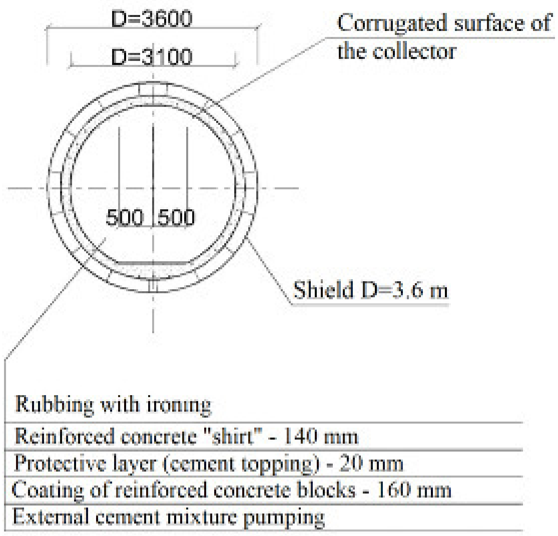

Figure 1 – Cross section along the sewer collector

The designed residential complex includes a stylobate part and three high-rise buildings [8] (Fig.1):

1) building «A» –99,05 m tall, has 22 residential floors, 2 technical and 3 underground floors;

2) building «B» –123,65 m tall, with 29 residential floors, 2 technical and 3 underground floors;

3) Building «C» –123,65 m tall, has 29 residential floors, 2 technical and 3 underground floors.

All the buildings belong to the structures of significant consequences CC3.

The foundations of buildings are plate-grid piles. The piles diameter is Ø 820 mm, the length is 25,0 m (the bottom level is 103,35 m), carrying capacity – 303,5 t, the thickness of a plate grid is 1500 mm. In the zone of the sewer collector location, the thickness of the plate grid is 2000 mm. The grid is supported by piles with the length of 35,0 m (the bottom level is 92,85 m) and carrying capacity of 450 t. Piles are located from both sides of the collector at a distance of 5 m. There are no piles directly above the collector [1]. In the stylobate part of the project, there are three underground floors which height is 3,15...4,20 m.

The design provides for constructive measures to reduce the negative impact of the residential complex construction on a collector, that is, the installation of a collector sheeting from the drilled piles with the length of 30,0 m and diameter of Ø 820 mm at the distance of 5 m from each side of the collector and the limitation of the piles length of building foundation (Fig. 1).

The sewer collector with the diameter of 3,6 m is built by shield tunneling method. The main bearing constructive elements of the collector are tubes with the width of 1.0 m and diameter of 3,6 m. The collector slope is i = 0,0005. The collector consists of 7 tubes and an internal monolithic reinforced concrete «shirt» with a thickness of 140 mm. Concrete is pumped between tubes and the shield tunneling through the openings in the tubing. Cross section of the collector is shown in Figure 2. The class of the collector concrete is C25 / 30. The tubing is reinforced by 5 steel rods with the diameter of Ø 12 mm The reinforcement is of the periodic profile. The cross-section along the collector is shown in Fig. 2.

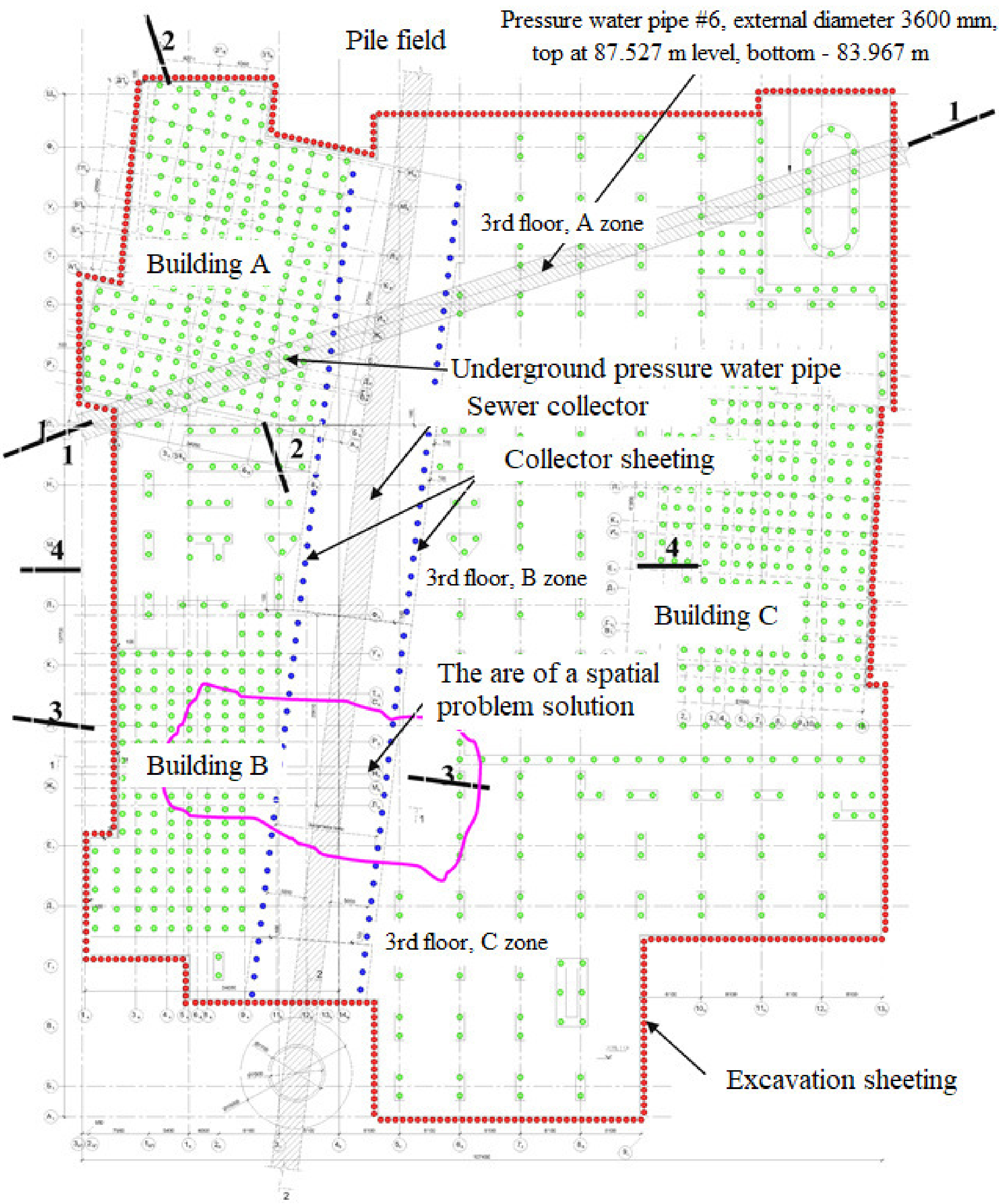

Figure 2 – Plot of the collector, the water supply, and the calculated cross sections 1-1…4-4

The technical condition of the collector on the site is unknown since its visual inspection was not carried out, therefore the possible degree structural elements wear is taken by analogy with the collector section which was surveyed in 1999. In this case, the following damages have been recorded:

1) a reinforced concrete «shirt» of the main city collector, in the upper part corroded to tubing, lost its bearing capacity and purpose;

2) large strata of the «shirt» had separated from tubes and deposited in the collector tray;

3) the groundwater infiltration in the collector was recorded.

According to the conclusion of the Standing Committee on Technological and Environmental Safety and Emergencies executive body of the Kyiv City Council on the collector inspection, it is in an emergency state (minutes № 56 of 10/06/2009).

Pressure water pipe, which passes through the area of the designed residential complex, belongs to the Desnianskyi water supply system in Kyiv. The water supply system with a diameter of 3.56 m was constructed by a shield tunneling method. The design of the water supply system is shown in Fig. 3.

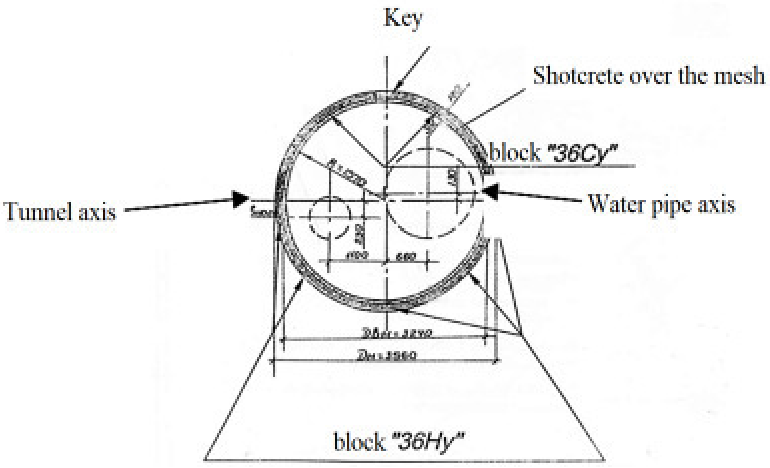

Figure 3 – The structure of underground water pipe

The main structural elements of the water supply are a metal pipe with a diameter of 1.32 m and reinforced concrete tubes with a diameter of 3.56 m. The collector ring consists of 7 tubes that serve as a reinforced lining with the thickness of 160 mm, and an internal monolithic reinforced concrete «shirt». The concrete is poured through the openings in the tubing in the space between the tubes and the shield tunneling. There is no data on the damage to the structures of the water supply, so it is impossible to judge the categories of the structures technical condition.

Concrete was injected into the space between the tubing and tunneling in the soil, the tubing concrete class is C25 / 30. The tubing reinforcement is made by 5 rods with the diameter of 12 mm, the reinforcement is done with periodic profile.

In geomorphologic terms, the site is located on the left slope of the river Lybid valley. The relief of the site has a slight slope in the southwest direction. Absolute marks on the Earth surface vary from 133.90 to 140.20 m.

The geological structure of the site to the explored depth of 50.0 m is composed of diluvia and glacial deposits complex, which is presented by sand, sandy loam, and loam, which are covered over by a fill-up soil. Under the diluvial deposits, there are sands of Kharkiv stage and the marl clay and loam of Kyiv stage. The geological structure of the construction site is composed of sand, sandy loam, loam, and clay covered over by a fill-up soil [9].

Hydrogeological conditions of the construction site are characterized by the presence of an aquifer, timed to diluvia sand, sandy loam [3]. The underground flow was registered by geotechnical surveys at absolute marks of 125.70 ... 128.90 m. The complexity category of geotechnical conditions, according to DBN A.2.1-1:2008 «Engineering surveys for construction», is the third (complex) [9]. Underground waters are registered at a depth of 6.90 ... 11.80 m, at the absolute marks of 125.70 ... 128.90 m and confined to diluvia sand and sandy loam.

Estimation of the SSS alteration of city sewer collector and underground water supply pipe soil base is carried out using finite element method. The calculations were carried out for the building A, which is located above the collector and the water supply pipe, and for the building, which is located above the city collector (Fig. 1).

Deformation module of the marl clay (EGE-10) according to the results of geotechnical surveys is E_1=30 MPa, and according to the results of presiometric surveys is E_2=50 MPa, so calculations for assessing the change in the SSS of soils within the collector and the water supply pipe were performed for these two values of the deformation module. These calculations were carried out in iterative manner using the Mohr-Coulomb model, which was used for the first approximation to soil existing state. The model includes five parameters: Young modulus (E), Poisson ratio (ν), cohesion (c), the friction angle (φ) and the dilatation angle (ψ). The program also considers soil volumetric weight in dry (γ_{unsat}) and water-saturated (γ_{sat}) state, and also the coefficients of filtration k_x and ky.

The soil massif is modeled by 15 nodal elements, where the soil with different physical and mechanical characteristics is distinguished by the 2nd group of limit states. Boundary conditions in the lower part of the model presented in the form of a continuous fixed support, and vertical walls presented by roller supports.

Calculations on the assessment of the SSS alteration for the soil in collector and water supply system base are made considering and without consideration additional loads from the designed buildings foundations A and B and for calculation sections 1-1 ... 4-4.

The calculation scheme for section 1-1 is shown in Fig. 4.

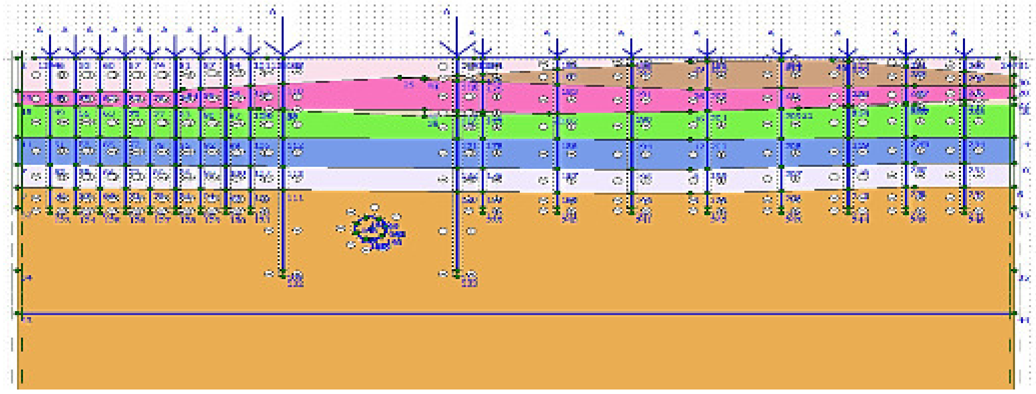

Figure 4 – The calculation scheme for section 1-1

Isolines of normal stresses σ_y Along section 1-1 for phase 6 (considering residential complex weight when E_1 = 30 MPa) are shown in figure 5, and sewer collector and water supply pipe structures deformation scheme – in figure 6, isolines of vertical displacements – in figure 7.

The calculation was performed considering the sequence of complex construction for such phases:

1) soil massif at the excavation bottom level – 128,15 m;

2) installation of the collector and water supply pipe;

3) the installation of piles for the sewer collector sheeting;

4) installation of piles for the foundation of the designed complex;

5) installation of plate foundation for the designed complex;

6) erection of a residential complex with a load of 700 kN/l.m.

Maximum settling of the collector and the water supply pipe, considering residential complex weight for the sections 1-1, 2-2, 3-3, 4-4 (figure 1, when E_1 =30 MPa and E_2 =50 MPa) are shown in table 1.

Normal stress distribution σ_y along the perimeter of the collector when E_1=30 MPa and E_2=50 MPa for sections 1-1, 4-4, 3-3 are shown in table 2.

For the determination of the tension occurring in the constructive elements of the collector and the water supply pipe, calculation was performed using «LIRA» software, whereas as the external loads, there were applied the stresses obtained from calculation of the SSS alteration for the soil around the collector and the water supply pipe. The simulation was performed using the «mounting» system. The modeling was carried out by three stages:

1) the natural state of the soil: enables to estimate the stresses in the soil from its own weight;

2) installation of the collector: enables to evaluate the stress arising in the designed elements of the collector and the water supply pipe;

3) installation of the pipe: enables to estimate the forces that arise in the structural elements of the collector and the water supply pipe from the additional load.

Assessment of the collector and water supply pipe structural elements technical state alteration was performed by increasing the main compressive and tensile stresses in the structural elements of the collector and the water supply pipe. In the calculations for estimating the alteration in the bearing structures SSS, the design concrete class of C25 / 30 was adopted.

Figures 8 and 9 respectively show the diagrams of the main tensile and compressive stresses in the collector (considering residential complex weight).

The main tensile stresses in the tubings' concrete of the collector during the erection of the structures' complex are 86 t/m^2 (fig. 8), which does not exceed the calculated resistance to tension of 105 t/m^2 (for the concrete class of C25/30).

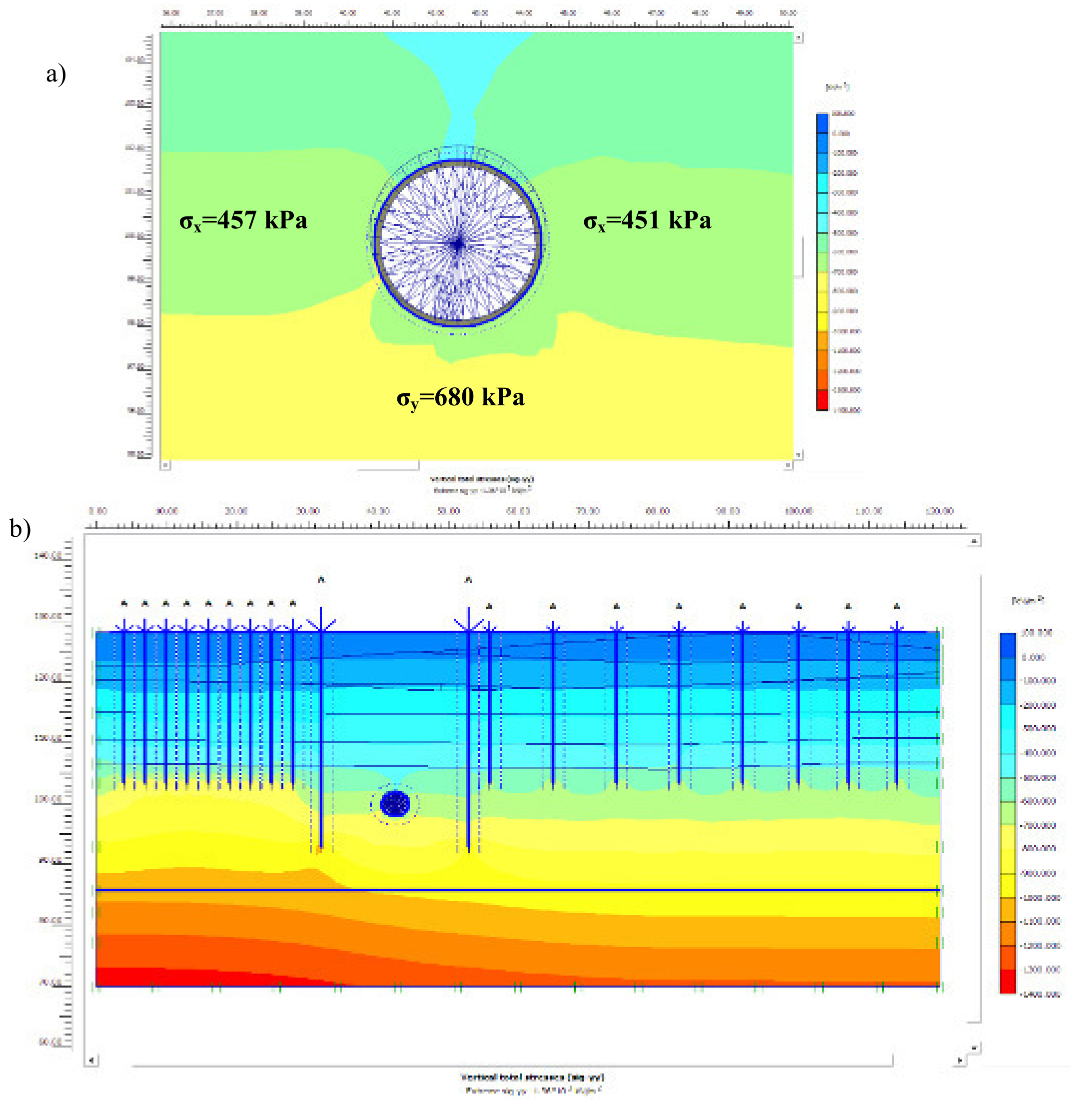

Figure 5 – Isolines of normal stresses σ_y
(taking into account the weight of the residential complex), phase 6 (E_1 = 30 MPa):
a – in the soil around the sewer collector; b – in the soil above the underground water supply pipe

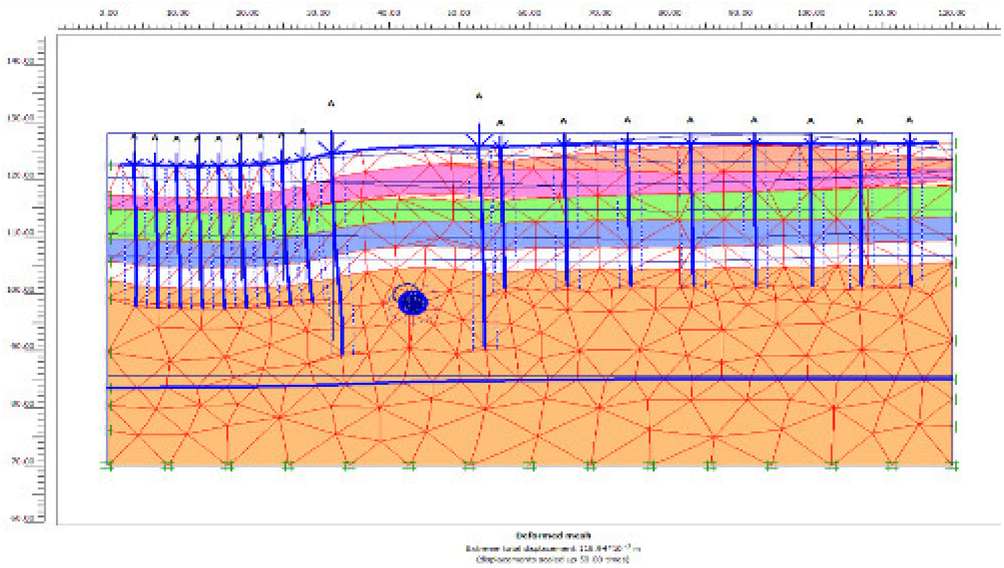

Figure 6 – The deformation scheme of sewage collector and water supply structures
(taking into account the weight of a residential complex), phase 6 (E_1 = 30 MPa)

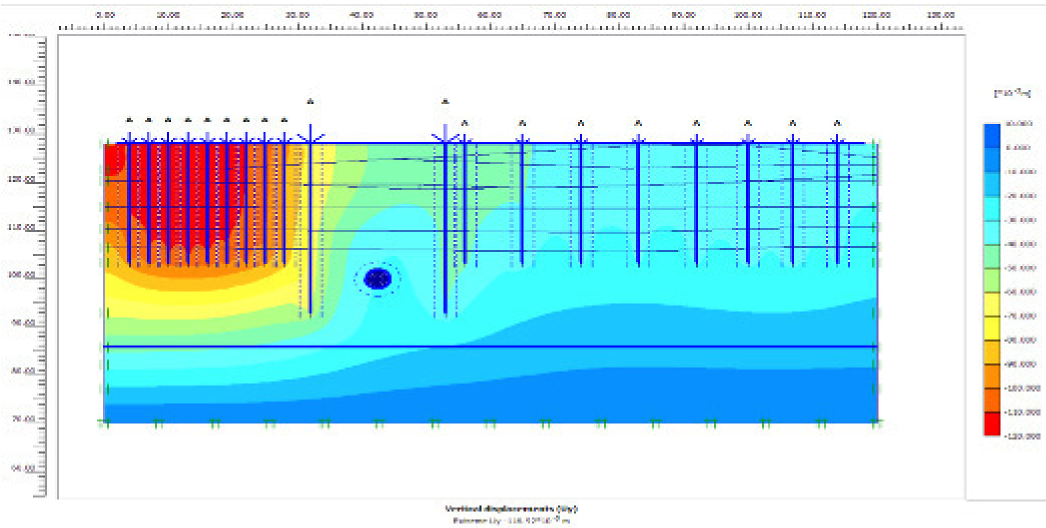

Figure 7 – Isolines of vertical displacements
(taking into account the weight of a residential complex), phase 6 (E_1 = 30 MPa)

Table 1 – Maximum settling of the collector and water supply system,
taking into account the weight of the residential complex

The underground structure’s name	The maximum settling of the collector and water supply pipe taking into account the weight of the residential, mm							
	Deformation module E_1 = 30 MPa				Deformation module E_2 = 50 MPa			
	Cross-sections (figure 1)				Cross-sections (figure 1)			
	1-1	2-2	4-4	3-3	1-1	2-2	4-4	3-3
Collector	32,8	–	10,0	12,0/10,0	18,3	–	6,0	6,0/5,0
Water supply pipe	39,2	34,8	–	–	23,6	21,7	–	–
Note. Denominator shows the settling, calculated for spatial case area shown in fig. 1								

Table 2 – Normal stress distribution σ_y along the perimeter of the collector

Nods number	Stress, kPa when the deformation module is E_1=30 MPa					
	Without accounting for the weight of the residential complex			Taking into account the weight of the residential complex (700 kN/l.m.)		
	Cross-sections (figure 1)			Cross-sections (figure 1)		
	1-1	4-4	3-3	1-1	4-4	3-3
1	467	466	466	487	505	519
2	465	456	456	451	491	491
3	608	603	603	680	667	685
4	466	457	457	457	491	491
	Stress, kPa when the deformation module is E_2=50 MPa					
1	410	461	462	462	491	508
2	383	475	462	468	504	543
3	584	602	598	661	655	679
4	380	477	463	474	503	536

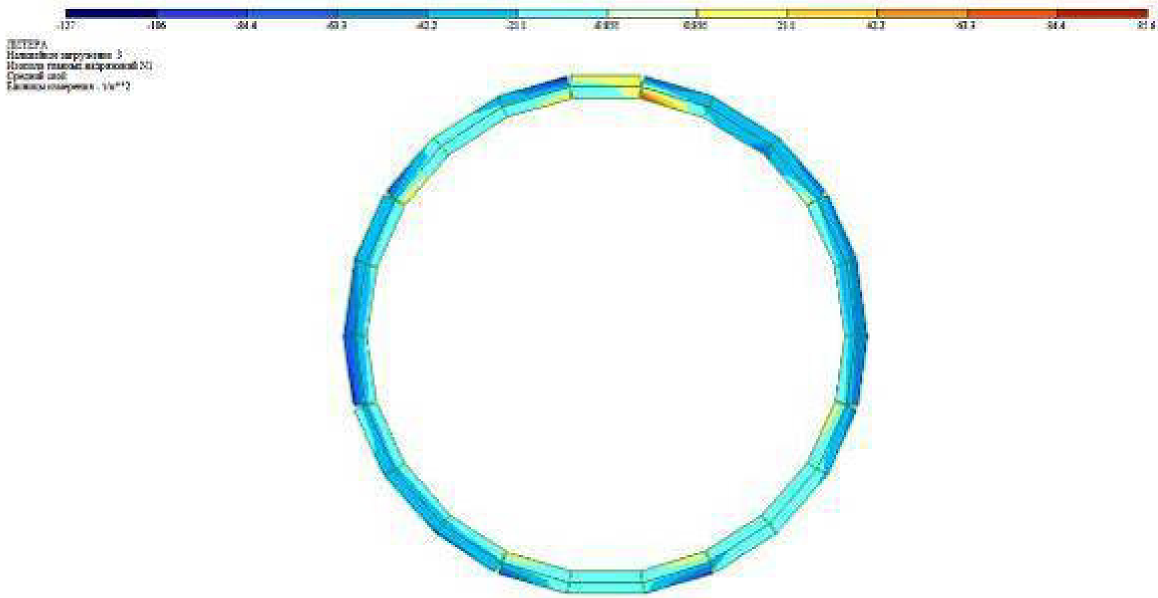

Figure 8 – The diagram of the main tensile stress in the collector (taking into account the weight of the residential complex)

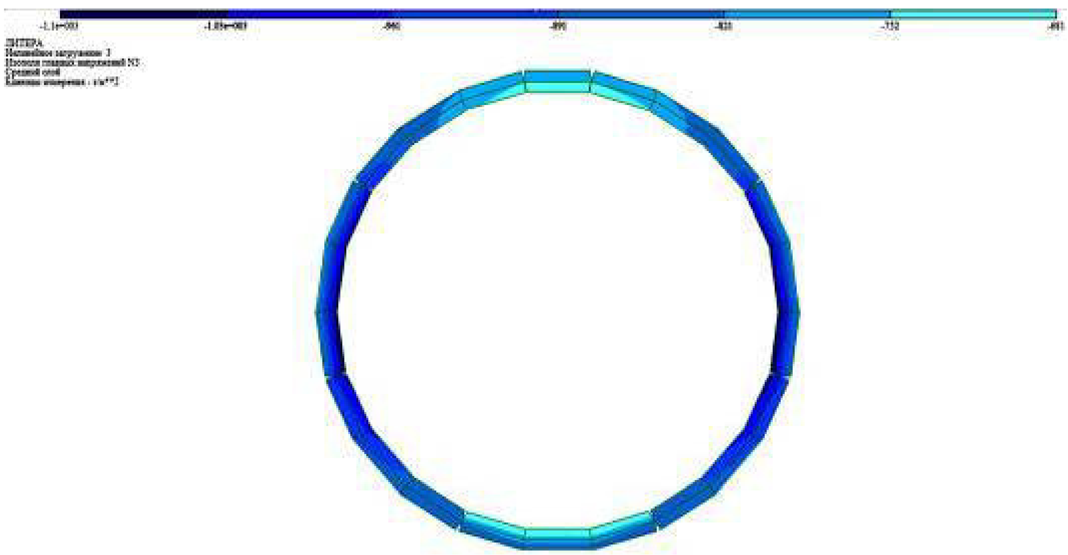

Figure 9 – The diagram of the main compressive stress in the collector (taking into account the weight of the residential complex)

The main compressive stress in the tubing concrete of the water supply pipe during the erection of the structures' complex is 1100 t/m^2, which does not exceed the calculated resistance of the concrete to the tension of 1450 t/m^2 (for the concrete class of C25/30).

Scientific and technical support should be carried out for complex construction or reconstruction objects, special geotechnical, hydro-geological, engineering-ecological conditions and complex relief; structures in the zone of influence (risk) of new construction (reconstruction) or areas where dangerous geological processes are possible [10].

Monitoring is carried out at the stages of designing and construction, as well as reconstruction and preservation operations for significant consequences of CC3 – in all cases, CC2 – in complex geotechnical conditions, in areas of a dense housing, in the new construction or reconstruction influence zone [11]. Monitoring at the construction and operation stage for a functional purpose should include visual-instrumental physical observations and survey (including geodetic control) of structures, bases, territories, hydrogeological and ecological observing system, and results analysis.

Conclusions.

1. The technical decision on reducing the load on the collector (the absence of piles directly over the collector, the use of shut-off pile rows for the collector sheeting has significantly reduced the impact of the residential complex on the collector.

2. The stresses in the bearing elements of the collector tubing and the water supply system, considering residential complex loads, do not exceed the designed values of concrete resistanc(class of concrete C25 / 30).

3. The loading from the projected residential building during the construction and operation period does not lead to deterioration of the existing technical condition of the collector and water supply structures provided that the actual physical properties of the collector and water supply tubing materials to the design value are matched.

Recommendations.

1. In order to exclude the possible emergency situation at the collector and / or water supply, inspection and assessment of the collector and water supply structures technical condition should be undertaken as well as their physical deterioration and the effect of collector and water supply structures subsequent operation on the designed residential building in terms of its reliable and safe exploitation conditions.

2. Evaluate the impact of building the residential complex on the existing surrounding housing.

References

1. ДБН В.2.1-10:2009. (2009). *Основи та фундаменти споруд.Основні положення проектування*. Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

2. ДБН В.2.1-10:2009. (2011). *Основи та фундаменти споруд.Основні положення проектування. Зміна №1*. Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

3. ДБН В.2.1-10:2009. (2012). *Основи та фундаменти споруд.Основні положення проектування. Зміна №2*. Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

4. ДБН В.1.1-45:2017. (2017). *Будівлі і споруди в складних інженерно-геологічних умовах. Загальні положення*. Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

5. ДСТУ-Н Б В.1.1-39:2016. (2017). *Настанова щодо інженерної підготовки ґрунтової основи будівель і споруд*. Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

6. ДСТУ-Н Б В.1.1-40:2016. (2017). *Настанова щодо проектування будівель і споруд на слабких ґрунтах*. Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

7. ДСТУ-Н Б В.1.1-44:2016. (2017). *Настанова щодо проектування будівель і споруд на просідаючих ґрунтах*. Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

8. *Будівництво житлово-адміністративного комплексу з вбудованими та прибудованими приміщеннями громадського і торгівельного призначення та надземним і підземним паркінгами по вул. Анрі Барбюса, 39/2 в Печерському районі м. Києва. Стадія проект*. (2011). Київ: ТОВ «ВІР - РМ».

9. *Звіт про інженерно-геологічні вишукування майданчику під будівництво житлово-адміністративного комплексу по вул. Анрі Барбюса, 39/2 в Печерському районі м. Києва. І етап. Стадія «П». Об'єкт: «Бдівництво житлово-адміністративного комплексу по вул. Анрі Барбюса, 39/2 в Печерському районі м. Києва . Арх. №12212*. (2008). Kyiv: ДП Міністерства оборони України «Центральний проектний інститут».

10. ДБН В.2.1-5:2007. (2009). *Науково-технічний супровід будівництва об'єкту*. Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

11. ДСТУ-Н Б В.1.2-17:2016. (2017). Настанова щодо науково-технічного моніторингу будівель і споруд. Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

12. Kryvosheiev, P., Farenyuk, G., Tytarenko, V., Boyko, I., Kornienko, M., Zotsenko, M., Vynnykov, Yu., Siedin, V., Shokarev, V. & Krysan, V. (2017). *Innovative projects in difficult soil conditions using artificial foundation and base, arranged without soil excavation,* Proc. of the 19th Intern. Conf. on Soil Mechanics and Geotechnical Engineering (Sep. 17 – 22, 2017 / COEX, Seoul, Korea). Retrieved from https://www.issmge.org.

13. Dvornyk, A. (2018). *Numerical modeling of structures fencing deep excavations of complex configuration*. Proc. of the 26th European Young Geotechnical Engineers Conf. (Sep. 11 – 14, 2018 / NAWI Graz Geocenter, Austria).

14. EN 1997-1:2004. Eurocode 7: *Geotechnical design - Part 1: General rules* .

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Designing of buildings and structures at land sliding and slide hazardous segments of slopes

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The main document of the regulatory framework for the design of buildings and structures on landslide and landslide-prone areas is DBN.1.1-46:2017 «Engineering protection of territories, buildings and structures from landslides and landslides. The main provisions» and the state standard DSTU-N B V.1.1-37:2016, «Manual on engineering protection of territories, buildings and structures from landslides and landslips». In development of the provisions of this set of regulatory framework, a number of regulations and standards have been developed to ensure the construction of buildings and structures on landslide and landslide-prone areas, considering the complex geological and hydro geological conditions of the construction site.

Keywords: landslide, building, structure, class of consequences (responsibility), design, construction

Проектування будівель і споруд на зсувних та зсувонебезпечних ділянках схилів

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Основними документами комплексу нормативної бази стосовно проектування будівель та споруд на зсувних і зсувонебезпечних ділянках є ДБН В.1.1-46:2017 «Інженерний захист територій, будівель і споруд від зсувів та обвалів. Основні положення» та ДСТУ-Н Б В.1.1-37:2016 «Настанова щодо інженерного захисту територій, будівель і споруд від зсувів та обвалів». Для розвитку положень цього комплексу нормативної бази розроблено низку нормативних актів і стандартів, які дозволяють забезпечити проектування будівель і споруд на зсувних та зсувонебезпечних ділянках з урахуванням складних інженерно-геологічних і гідрогеологічних умов ділянки будівництва. У комплексі нормативної бази наведено основні принципи та вимоги щодо проектування, будівництва й реконструкції будівель і споруд на зсувних та зсувонебезпечних ділянках усіх видів і класів наслідків (відповідальності), екологічні вимоги при їх проектуванні й оцінювання існуючих інженерно-екологічних умов та прогнозу їх можливої зміни на території (ділянці будівництва). Цей комплекс нормативних документів слід застосовувати при проектуванні будівель та споруд на зсувних і зсувонебезпечних ділянках. Вибір типу фундаментів будівель та споруд і заходів закріплення схилів повинен базуватися на техніко-економічному порівнянні варіантів, на інженерних розрахунках, а також вимоги щодо охорони навколишнього природного середовища, забезпечувати стійкість територій, надійне й безперебійне функціонування впродовж розрахункового терміну служби об'єктів, які проектуються. При подальшому вдосконаленні проектування будівель та споруд за комплексом нормативних документів та стандартів можливе їх розширення із залученням існуючих та нових ДБН і ДСТУ.

Ключові слова: зсув, будівля, споруда, клас наслідків (відповідальності), проектування, будівництво

Introduction. Constant growth of territories with dangerous geological processes happens in Ukraine. The number of landslides increases and previously stable slopes transit to the category of dangerous landslides. As a result, engineering and geological risk of territories development and the occurrence of emergency situations are increasing. The territories majority with smooth-moving relief with ordinary engineering and geological conditions and the dangerous geological processes absence are built up currently in Ukraine. It causes decreasing of free territories for construction with favorable conditions and the necessity of designing and erecting new construction projects in areas with complex engineering and geological conditions, in a landslide and landslide hazardous areas and in zones of dangerous geological processes.

Such areas are characterized by significant marks variations of the earth surface, the possibility of landslide processes activation, high levels of groundwater standing, complex engineering and geological conditions, the presence of lands with special properties, etc.

The National regulatory framework of Ukraine for the design of buildings and structures in a landslide and landslide hazardous areas includes a set of normative documents and standards for their design in the usual and complex engineering and geological conditions and in zones of dangerous geological processes.

The Ministry of Regional Development of Ukraine has instructed the DP NDIBK to develop new construction norms in the specified direction, which should measure up the modern requirements for the construction norms and the development level of the scientific and technical base of the construction industry. DP NDIBK has developed DBN V.1.1-46:2017 and DSTU-N B.1.1-37:2016 that are the main part of the normative documents and standards' set of Ukraine regarding the design of buildings and structures in a landslide and landslide hazardous areas in various engineering and geological conditions.

Analysis of recent sources and publications. The issues of buildings and structures erection in a landslide and landslide hazardous areas in conventional and complicated engineering and geological conditions, as well as in zones of dangerous geological processes, methods of bases strengthening, constructive solutions of counter landslide and retaining structures are set out in a number of normative documents and standards. There are DBN V.2.1-10 and Changes №1 and №2, DBN V.1.1-45, DBN V.1.1-46, DSTU-N B V.1.1-37, DSTU-N B V.1.1-39, DSTU-N B V.1.1-40, DSTU-N B V.1.1-44[3-9].

Identification of unsolved issues in the considered problem. The existing set of normative documents and standards for the design of buildings and structures in a landslide and landslide hazardous areas does not fully ensure the possibility of their design at the modern level.

DBN V.2.1-10-2009 and Changes №1 and №2 relate to designing the foundations of buildings and structures in convenient engineering and geological conditions. These documents were adopted a long time ago, and, therefore, a number of their conditions are out of date, does not measure up modern requirements and it is needed to be modified. The introduction of new DBN V.2.1-10:201X «Bases and foundations of buildings and structures. Basic provisions» and standards in their development enables to update the set of normative documents on the design of buildings and structures in a landslide and landslide hazardous areas, which are projected in the territories in convenient and complex engineering and geological conditions and in zones of dangerous geological processes, including seismic influences.

The purpose of development of the DBN V.2.1-10:201X is to improve the existing state building codes [3 – 5] in the field of buildings and structures design foundations, considering modern principles of its design. These norms are an integral part of normative documents and standards set that establish mandatory requirements for the buildings and structures design in a landslide and dangerous landslide areas and are intended for use at all stages of construction objects life cycle.

The main material and results. In the draft of DBN V.2.1-10:201X general conditions and requirements for the design, construction and reconstruction of buildings and structures foundations of all types and classes of consequences (responsibilities) are given, basic requirements for design of engineering preparation of bases, engineering design content, environmental requirements for buildings and structures foundations design are contained.

The choice of buildings and structures foundations type and measures for fixing the slopes should be based on technical and economic comparison of options, on engineering calculations, considering urban planning requirements, as well as requirements for environmental protection, ensuring the stability of the territories, reliable and uninterrupted functioning during the estimated objects life cycle that are projected.

The contexture of normative documents set and standards for buildings and structures design on the landslide and landslide hazardous areas in convenient and complex engineering and geological conditions in zones of dangerous geological processes is presented in Fig. 1.

For buildings and structures (construction object) design on landslide and landslide hazardous areas, a set of basic normative documents and standards (Figure 1) and comprehensive approach to the assessment of construction projects should be applied and performed according to the scheme (Figure 2).

The sequence of construction object design is given in the form of items in Fig. 2.

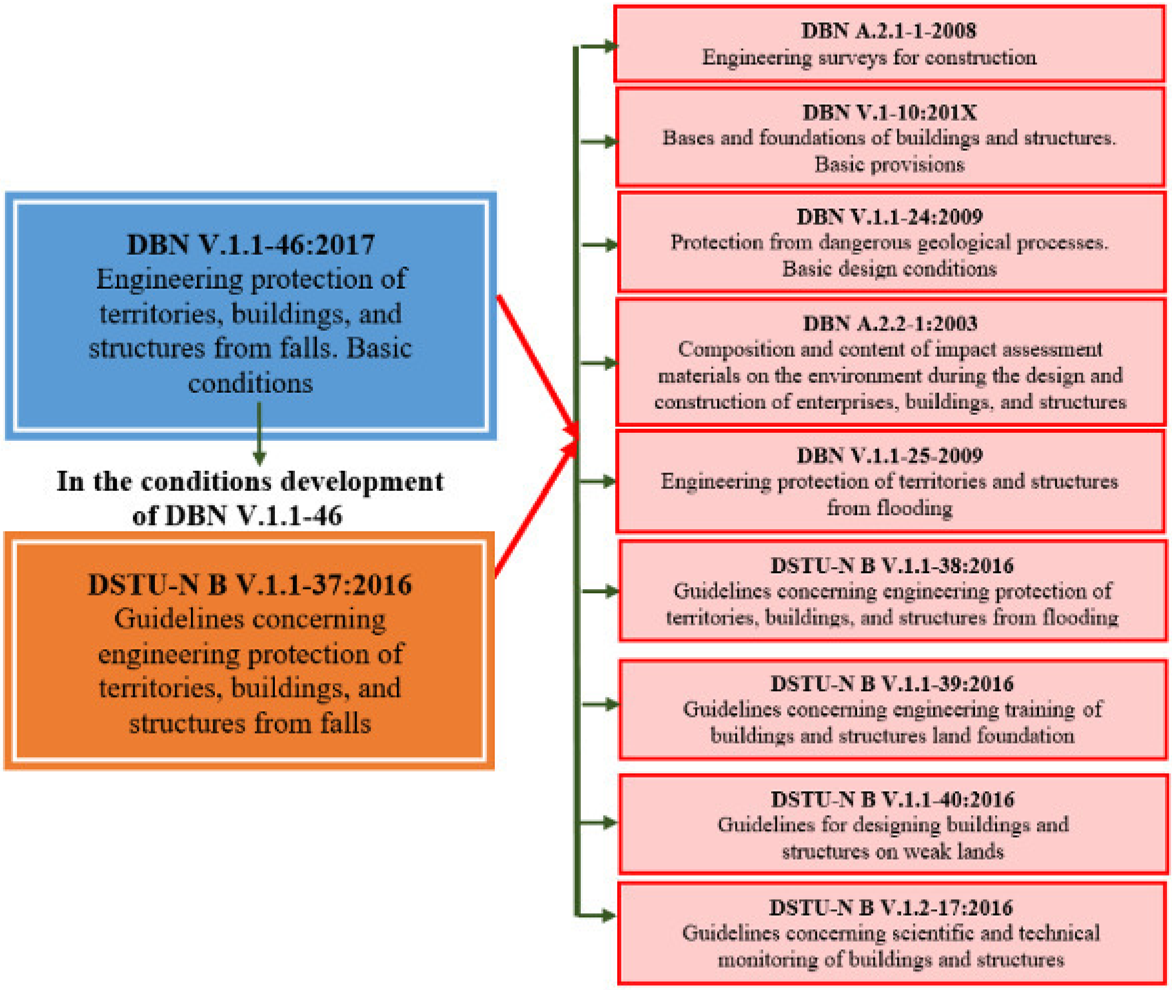

Figure 1 – The contexture of a set of normative documents and standards for the design of buildings and structures on the landslide and landslide hazardous areas

1. Engineering, geological and hydrogeological surveys on the construction site are performed for DBN A.2.1-1-2008 [10]. On the basis of engineering geological surveys there are performed:

– engineering and geological zoning of the territory due to the danger of landslide processes, as well as the peculiarities of its development;

– assessment of slopes stability and its expected changes, indicating the type of possible landslide processes, their location, size, as well as soil masses movement magnitude and velocity;

– evaluation of indirect effects caused by landslide processes (deformation of existing buildings and structures).

The analysis of engineering, geological and hydrogeological conditions of the construction site enables to determine the type of slope (landslide or landslide hazardous), the initial data for calculating the slope stability (types of soil and their physical and mechanical characteristics, groundwater levels, etc.) and presence of lands with special properties.

2. Construction site inspection enables to determine the presence of landslides traces (soil bulgings) on the slope, its type, and to designate the widths for calculating slopes stability.

3. Design of buildings and structures for a site without a slope is made according to DBN V.2.1-10 [3-5].

4. The calculation of the slope stability in the natural state is carried out using two basic methods.

They are based on the theory of limit balance considering the stress state of the soil mass in limit balance conditions [1, 2]:

1) By blocks method (the method of G. M. Shakhunyants).

2) According to the slope soil stress-strain state analysis.

The coefficient of slope stability k_{st} is equal to the sum ratio of all retaining forces (moments) (R_i) to the shear forces (moments) (F_i)

$$k_{st} = \frac{\sum_{i=1}^{i=n} R_i}{\sum_{i=1}^{i=n} F_i}. \qquad (1)$$

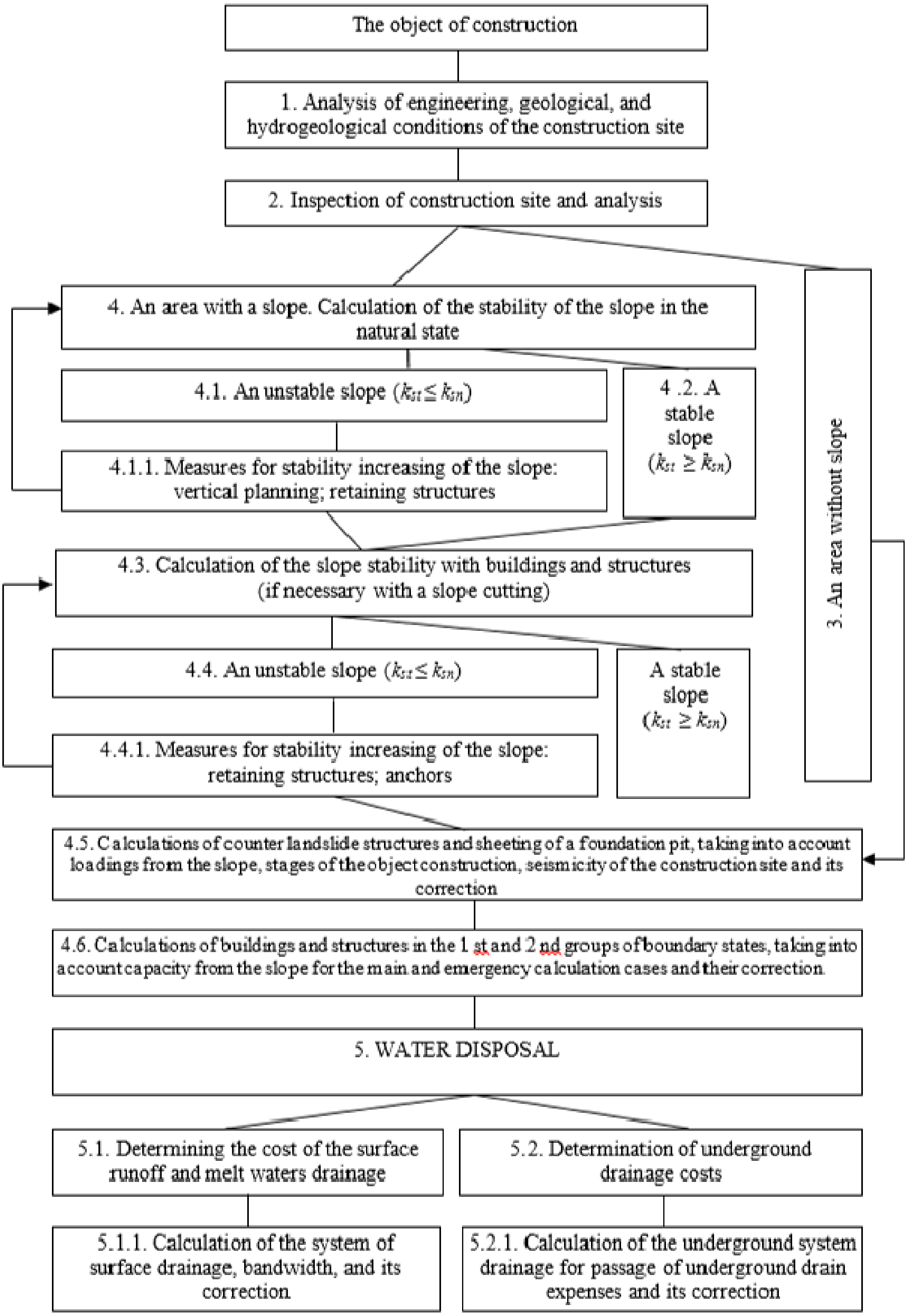

Figure 2 – The scheme of an integrated approach to the evaluation of construction projects in a landslide and landslide hazardous areas

The blocks method (the method of G. M. Shakhunyants enables to calculate slope stability from the natural to the project state at the first limit state - for the carrying capacity (under limit balance condition). Calculations are performed for the slope considering static, hydrodynamic and filtration capacities.

Assessment of slopes stability by the method of limit balance is recommended to perform in the following sequence:

1) analysis of engineering and geological conditions and choice of calculation section (sections);

2) compilation of the calculation scheme considering the features of the engineering and geological structure and physical and mechanical properties of soils;

3) generalization of the calculation scheme (simplification of the slope geometry by joining into one group the soils with the same or close physical and mechanical characteristics, removing non-essential forms of relief and geological elements, replacing the curvilinear sections of earth surface and depressions surfaces by straight lines);

4) determination of external additional capacities (magnitudes and directions of external additional capacities on the slope, application points of concentrated forces, boundaries and distributed nature of distributed forces);

5) determination of surface or sliding surfaces (sliding surface line (or set of such lines) considering peculiarities of engineering and geological structure and soils physical and mechanical properties (weak soils, the presence of hollows, peculiarities of hydrogeological conditions, etc.);

6) the landslide separation of the soil into blocks (compartments) according to the configuration of the slide surface;

7) determination of the stability coefficient by the formulas of the limit balance.

The method of finished elements considers the stress-strain state of the soil mass considering soil elastic and plastic behavior with building constructions and structures in the soil massif.

The essence of the finite element method is that the settlement system is replaced (approximated) by a system with a finite number of freedom degrees, that is, the discretization of the system is performed on separate elements that are interconnected by nodes. The work of a discrete system is determined by the interaction of individual finite elements.

The finite element method enables to perform calculations on the limit states of both groups in the same calculation scheme with one land model.

The indicator of slope stability degree in the finite element method is the reliability coefficient (slope stability coefficient k_{st})

$$k_{st} = \frac{c_v}{c_{cr}} = \frac{tg\varphi_v}{tg\varphi_{cr}}, \quad (2)$$

where c_v, φ_v are soils initial strength characteristics; c_{cr}, φ_{cr} are critical strength characteristics corresponding to plastic soil fluidity in the considered area.

Assessment of slopes stability by finite element method is recommended to perform in the following sequence:

1) choice of the sections calculation;

2) compilation of calculation scheme;

3) generalization of calculation scheme (unification of soils with similar physical and mechanical characteristics into a single group, neglecting the insignificant forms of relief and geological elements, replacement of curvilinear Earth surface spots and depression surfaces by straight segments);

4) determination of external additional loadings (value and direction of external additional loadings on a slope, application points of concentrated loadings, limit and distribution character of distributed loading;

5) calculation of underground water pressure;

6) calculation of domestic stresses in a soil;

7) determination of the reliability factor (stability coefficient of a slope k_{st}) utilizing the «reduction of c and φ» method.

For stress-strain state simulation of soil, the non-linear model is used which strength criteria is described by the Coulomb-Moor's law. The soil is considered to be the elastic and ideally plastic material which deformation occurs in accordance with the Prandtl chart.

As a result of calculations by two methods, the coefficients of stability reserve k_{st} are determined for the most characteristic cross-sections and compared to normalized values k_{sn}. Then conclusion as for slope stability is made (when $k_{st} \geq k_{sn}$ the slope is steady, when $k_{st} \leq k_{sn}$ the slope is unstable, and when $k_{st} \approx 1$ there is a state of the boundary equilibrium of the soil massif that, as a rule, causes landslide).

4.1. Considering unstable slope in natural condition ($k_{st} \leq k_{sn}$) the slope stability basic means and activities increasing are necessary to utilize, that is [1, 2]:

– retaining structures (retaining walls), pile foundations and deeply layered foundations;

– soil massif cementation by some of the conventional methods;

– foundations that are flowed around by soil sliding masses;

– bank protection structures for protection banks and slopes against erosion and leaching (sea, river, at reservoirs and ponds);

– regulation of underground drain (drainage of deep and shallow laying, wall drainage and catchment);

– regulation of surface drain (protection of slopes surfaces from the infiltration of rain and meltwater into the soil, installation of drainage structures);

– slope tilt reduction (soil cutting at slope the top and laying out of soil masses at the foot for an additional loading in the places of the expected soil bulging);

– anti-erosion constructions;

– artificial change of the slopes relief by adjusting the balance and planning the sliding surface and adjacent territory (cutting of various protrusions, banks, filling up the hollows, etc.);

– agroforestry (trees planting, bushes, perennial grasses);

– organization of a protective anti-landslide regime on the slope.

The means and activities of engineering protection choice for objects from landslides must be based on technical and economic comparison of variants, engineering calculations. It should consider city planning requirements, requirements for environmental protection and rational use of land resources, provide territories stability higher degree, reliable and uninterrupted functioning during the estimated service life of the objects under protection.

4.2. With a steady slope in the natural state ($k_{st} \geq k_{sn}$) buildings and structures are being "incorporated" into the slope (with a slope cutting if necessary). To facilitate the operation of retaining structures the part of land sliding pressure can be redistributed to building or structure foundation [1, 2].

4.3. Calculations and analysis of slope stability with buildings and structures are carried out (with a slope cutting when necessary). When calculating slope stability, it is necessary to consider position of a sliding surface that is lower than its calculation or deformed horizons zones, including zones under the supports (piles) lower ends. With calculation of a slope stability there must be performed calculations of the filtration strength of the slope soil at the spots of groundwater bulging, heterogeneous soils and soils that are easily exposed to suffusion, and at the contacts of soils with drainage sandbanks (filters) [1, 2].

4.4. When the slope is unstable ($k_{st} \leq k_{sn}$) along with buildings and structures (with a slope cutting when necessary) there must be utilized only the basic means for slope stability increasing (**section 4.1**).

4.5. Based on the initial data of engineering geological and hydrogeological surveys at a constructions site the calculation of excavation sheeting is performed. The calculation must consider slope loading, object construction stages, and area seismicity. When necessary, the design is corrected according to calculation results [3 – 5].

4.6. Calculation of buildings and structures is carried out according to the 1 and 2 groups of limit states for a basic and emergency design cases considering the slope loading, seismicity of the area, and, when necessary, the design is corrected according to calculation results [3 – 5].

5. The operation of the drainage system of surface and underground waters from the construction site is analyzed. One of the main causes of landslides is the soaking of soils from groundwater, which leads to load-bearing capacity decrease of the soil massif. Water dampens possible slide surfaces, which reduces frictional forces and contributes to a landslide.

The discharge of groundwater from the plateau occurs through the slopes. The speed of groundwater motion on or near the slopes increases while territory flooding. As a result, the pressure gradient increases, suffusion, and fluidity phenomena develop, mechanical properties of soils decrease, loess loams in particular. The development of these negative phenomena is often associated with the presence of hollows in the waterproof layer [11-13].

5.1. Calculation of surface runoff of rain and meltwaters from the catchment is performed. Measures to regulate surface runoff are the mandatory part of protective structures and devices complex aimed at increasing the general and local stability of slopes [14].

Estimated rainwater costs in the landslide zone should be determined by the method of limit intensities. The period of the one-time excess of the estimated rain intensity should be set at least 5 years, and with a proper technical and economic justification – not less than 10 years.

The bandwidth calculation of the surface drainage system from the construction site surface runoff and its adjustment is performed [14].

Laying of water-supply communications in landslide areas is not enabled. In exceptional cases and the relevant technical and economic justification the laying of water- supply communications is possible on the surface of the Earth in the passage or semi-passage channels, which should be beyond the landslide and landslide hazardous areas.

On areas adjacent to slopes, the surface runoff should be controlled using drainage channels, trays, as well as guarding shafts, which provide interception of surface waters [12, 13].

5.2. Calculation of underground runoff at the construction site is performed. Design of bases water protection, underground engineering structures, in-depth structures, and foundations is carried out for maintaining the durability of structures, eliminating the accelerated wear of reinforced concrete elements in watered environment. Requirements for water protection should be developed considering the influence of water [12, 13]:

– temporary influence due to infiltration of atmospheric precipitation, flood flooding, water supply disruptions;

– permanent influence due to the presence of moisture in the soil or groundwater.

The regulation of the underground runoff should be used in a complex of protective structures and measures in order to eliminate or weaken the action of groundwater on soils (reducing its density and strength), reduction or elimination of hydrostatic and filtration pressures, etc.

Types, sorts, designs, and dimensions drainage devices main elements should be assigned depending on the engineering-geological and hydrogeological conditions of the slope (slope), terms of the protected area use, conditions of work on the basis of water balance, filtration and hydraulic calculations, as well as technical and economic comparison of variants. In landslide zones, the following types of drainage devices should be used:

– horizontal drainage – trench (tubular drainage) with pipes (including drainage pipes with tubing filters), without pipes (drainage slots) and galleries; tunnels; stratum drainages;

– vertical drainage – drilling wells and mines;

– combined drainage systems – a combination of horizontal and vertical drainage.

Calculation of the underground drainage system from the construction site is performed and corrected to account for a bandwidth of underground waters runoff [12, 13].

The location of drainage systems should be linked to the general scheme of the general landslide countermeasures complex, considering possible change in the boundaries of landslide deformations, and the depth of laying the networks of these systems should be justified.

When adjusting groundwater levels, it is necessary to consider:

– interception and reduction of water levels to exclude bulging on landslide or landslide hazardous slopes;

– captation of water outlets on the slopes;

– drying-out the landsliding massif of soil;

– stabilization or reduction of water levels in contact with retaining foundations or structures.

Discharge of drainage waters from territories should, as a rule, be self-sufficient. In case of impossibility of such withdrawal, it is necessary to arrange pump stations.

The project must contain the necessary engineering solutions to preserve, protect or improve the environmental situation on the site of construction and adjoining territory [15].

Scientific and technical support should be provided for complex construction or reconstruction projects, special engineering-geological, hydro-geological, engineering-environmental conditions and complex relief; structures in the zone of influence (risk) of new construction (reconstruction) or areas where dangerous geological processes are possible [11, 16].

Monitoring is carried out at the stages of designing and construction as well as reconstruction and works on buildings preservation: significant consequences CC3 – in all cases, CC2 – in complex engineering and geological conditions, in areas of dense construction, in the influence zone of new construction or reconstruction [17].

Monitoring at the stage of construction and operation for functional purposes must contain visual-instrumental physical observations and surveys (including geodetic control) of structures, bases, territories, hydrogeological and ecological observing system, and analysis of results.

Conclusions. Application of normative documents and standards (DBN and DSTU) set for buildings and structures design in landslide and landslide hazardous areas in conjunction with the scheme of integrated approach to the evaluation of construction projects in these areas enables to perform buildings and structures design more efficiently and reasonably, to increase objects reliability and safety, to bring the practice of designing buildings and structures in landslide and landslide hazardous areas in accordance with modern requirements.

With further improvement of buildings and structures design in respect of normative documents and standards set and integrated approach scheme to the evaluation of construction projects, it is possible to expand them with the help of existing and new DBN and DSTU, as well as construction projects evaluation scheme improvement.

References

1. DBN V.1.1-46:2017. (2017). *Інженерний захист територій, будівель і споруд від зсувів та обвалів. Основні положення.* Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

2. ДСТУ-Н В Б.1.1-37:2016. (2017). *Настанова щодо інженерного захисту територій, будівель і споруд від зсувів та обвалів*. Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

3. ДБН В.2.1-10:2009. (2009). *Основи та фундаменти споруд. Основні положення проектування*. Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

4. ДБН В.2.1-10:2009. (2011). *Основи та фундаменти споруд. Основні положення проектування. Зміна 1.* Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

5. ДБН В.2.1-10:2009. (2012). *Основи та фундаменти споруд. Основні положення проектування. Зміна 2.* Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

6. ДБН В.1.1-45:2017. (2017). *Будівлі і споурди в складних інженерно-геологічних умовах. Загальні положення*. Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

7. ДСТУ-Н В Б.1.1-39:2016. (2009). *Настанова щодо інженерної підготовки грунтової основи будівель і споруд*. Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

8. ДСТУ-Н В Б.1.1-40:2016. (2017). *Настанова щодо проектування будівель і споруд на слібких грунтах*. Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

9. ДСТУ-Н В Б.1.1-44:2016. (2017). *Настанова щодо проектування будівель і споруд на просідаючих грунтах*. Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

10. EN 1990:2002, IDT .DSTU-N В V.1.2-13:2008. (2009). Єврокод 7. *Основи проектування конструкцій.* Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

11. EN-1997-1:2004, IDT DSTU-N В EN 1997-1:2010. (2011). Єврокод 7. *Геотехнічне проектування. Частина 1. Загальні правила.* Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

12. EN-1997-2:2010, IDN DSTU-N В EN 1997-2:2010. (2011). Єврокод 7. *Геотехнічне проектування. Частина 2. Дослідження й випробування грунту.* Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

13. ДБН А.2.1-1-2008. (2009). *Інженерні вишукування для будівництва*. Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

14. ДБН В.1.1-24:2009. (2009). *Захист від небезпечних геологічних процесів. Основні положення проектування.* Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

15. ДБН В.1.1-25-2009. (2009). *Інженерний захист територій та споруд від підтоплення та затоплення*. Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».
16. ДСТУ-Н Б В.1.1-38:2016. (2017). *Настанова щодо інженерного захисту територій, будівель і споруд від підтоплення та затоплення.* Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».
17. СНиП 2.01.14-83. (1985). *Определение расчетных гидрологических характеристик*. Москва: Госстрой СССР.
18. ДБН А.2.2-1:2003. (2017). *Склад і зміст матеріалів оцінки впливу на навколишнє середовище (ОВНС) при проектуванні й будівництві підприємств, будівель і споруд.* Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».
19. ДБН В.2.1-5:2007. (2009). *Науково-технічний супровід будівельних об'єктів*. Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».
20. ДСТУ-Н Б В.1.2-17:2016. (2017). *Настанова щодо науково-технічного моніторингу будівель і споруд.* Київ: Мінрегіонбуд України, ДП «Укрархбудінформ».

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Peculiarities of structures inspection by the example of a three-chamber navigation lock in Zaporizhzhia city

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Peculiarities of structures of hydro technical structures technical condition are considered. The problem concerning normative documentation on the inspection of hydraulic structures is considered, and the absence of standards for the inspection of this type structures is revealed. The issue of hydraulic structures inspection by the example of a three-chamber lockin Zaporizhzhia city is considered. The issue of the deformed state according to observations of past years are considered. The recommendations for the further exploitation, repair and completion of reconstruction of a three-chamber lockin Zaporizhzhia are given.

Keywords: hydrotechnical structure, inspection, evaluation of technical condition, safe exploitation

Особливості обстеження гідротехнічних споруд на прикладі трикамерного судноплавного шлюзу в м. Запоріжжя

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Розглянуто питання обстеження з визначенням технічного стану будівельних конструкцій гідротехнічних споруд. Указано основні причини виконання обстеження будівель та споруд. Наведено деякі приклади будівельних об’єктів, на яких виникли аварійні ситуації за останні декілька років. Розглянуто питання щодо нормативної документації з питання обстеження гідротехнічних споруд, виявлено відсутність діючих нормативів з обстеження споруд такого типу. Розглянуто питання обстеження гідротехнічних споруд на прикладі трикамерного судноплавного шлюзу в м. Запоріжжя. Проаналізовано життєвий цикл гідротехнічної споруди (зведення, експлуатація, відбудова, реконструкція). Наведено результати натурних дослідженнь міцності матеріалів конструкцій. Для визначення міцності бетонних конструкцій об’єкта обстеження було відібрано зразки бетону (зі стіни 1-ї камери шлюзу, днища 2-ї камери шлюзу, стіни 3-ї голови шлюзу). Виконано випробування зразків, за яким було встановлено міцність відібраних зразків на стиск. Визначено складні місця з діагностування технічного стану гідротехнічних споруд. Доведено, що якісною характеристикою технічного стану споруди є його деформований стан для оцінювання стану трикамерного шлюзу. Розглянуто питання деформованого стану за даними спостережень минулих років, наведено графіки деформацій та коливання рівня ґрунтових вод. Запропоновано рекомендації з подальшої експлуатації, ремонту та завершення реконструкції трикамерного судноплавного шлюзу в м. Запоріжжя.

Ключові слова: гідротехнічна споруда, обстеження, оцінка технічного стану, безпечна експлуатація

Introduction. Hydraulic structures and any engineering structure inspection is performed to determine object technical condition and its individual elements. The main types and reasons for the inspection are:

a) planned inspection has been performed before the design and execution of works on reconstruction, overhaul or restoration;

b) scheduled periodic inspections have been performed according to the established procedure and the term;

c) unscheduled inspections – performed at the inspected of strains, defects and injuries, in order to determine the causes and their elimination, as well as the restoration of damaged structures.

Analysis of recent sources of research and publications. Hydrotechnical structures represent potentially dangerous objects of failure which can lead to large losses (human, economic, environmental), therefore the question of determining these objects technical state is currently quite acute. Permanent occurrence of emergencies on different types of construction sites in the country are given such as:

– fall of a metal roof with an area of 2400 m^2 in early March 2018 at the shop of «ArsilorMittal Kryvyi Rih»;

– destruction of a new elevator near the city of Nikolaev in August 2017;

– fall of the bridge construction part in Kyiv in February 2017, and so on.

Thus, the inspection of buildings and structures, especially hydrotechnical ones, with increased danger, is one of the most important construction projects in Ukraine and abroad [9,10]

At present new regulatory documents on inspection [2, 4] have been developed and are in implemented in the construction industry, and the evaluation of buildings and structures technical condition for civil and industrial use.

Highlighting of unsolved aspects of the problem. In this regard, the hydrotechnical structures do not have normative documents on technical condition inspection and determination; at present, only order No. 252 , December 19, 1995 of the State Committee of Ukraine for Urban Development and Architecture [6] exists. Thus, the development and implementation of normative documents on inspection of hydraulic structures is extremely relevant, considering these objects resposibility degree.

Problem statement. To consider the inspection of hydraulic structures with the definition of object technical condition and its individual elements, for example, a three-chamber locklocated in the city of Zaporizhzhia.

Main material and results. The question of hydrotechnical structure inspection is considered on the real object of a three-chamber lock in Zaporozhye. The general type of gateway construction is shown in Figure 1.

In 2017 the specialists of Zaporizhzhia branch of State Enterprise «State Research Institute of Building Structures» and State Enterprise «State Research Institute of Building Structures», with the involvement of a professional executor (construction and hydrotechnical engineer) performed works on visual and instrumental inspection of this hydraulic engineering works. The purpose of the work was to inspect the construction structures, evaluate their technical condition and develop recommendations for their repair and completion of reconstruction, determine structures materials strength on selected samples, analysis of object deformed state according to observations during past years. Considering the lack of normative documents with the technical condition for such buildings, during the inspection norms for civil and industrial construction objects are used [2, 4]. The three-chamber lockis an integral part of the hydraulic unit in the gravitational dam of two hydroelectric power stations and two gateways (three-chamber (inspected, not exploited) and single-chamber (exploited)). The inspected object basis are the rocks represented by granites, which have a cracked structure. It should be noticed that the drainage tunnels of the new gateway pass under a three-chamber gateway. Thus, the inspection object can be attributed to the structures located in complex engineering-geological conditions. [1].

Figure 1 – General view of a three-chamber navigation lock in Zaporizhzhia city

The life cycle of this object can be divided into five stages:

– errection and putting into exploitation (1927–32, 1934 year) (Fig. 2);

– exploitation (1934 – 1941 year);

– post-war reconstruction (1945 – 1950 year);

– exploitation (1950 – 1993 year);

– reconstruction (1993 – 2018 year).

In general, the inspection object can be divided into the following construction elements:

– wharf and upper and lower canals;

– the heads of the lock (four heads);

– lock chambers (three chambers);

– culverts (pass from the upper underwater channel through the heads and bottom of chambers to the lower underwater channel);

– gate shafts (located in the heads of gateways by two on each head);

– premises for ships kinetic energy absorbers mechanisms (located in the first and second lock heads, two on each head)

– premises for gates control mechanisms (located two on each head);

– turning bridge premises (located on land lock wall of the first chamber in the first head);

– drainage path from the river side of 3 chamber;

– control panel towers (located at the second chamber of the lock on the river and land sides);

– unitized transformer substation building (erected near the tower of land side control);

– cable channels (arranged along the structure on both sides).

– to determine the concrete structures strength, the following samples were taken from the following structures:

– wall of the 1st lock chamber – 4 batches of 3 samples $\varnothing$94 mm (designation К1.1–К1.4);

– the bottom of the 2_{nd} chamber of the lock – 2 batches of 3 samples $\varnothing$94 mm (designation К2.1-К2.2);

– the walls of the 3_{rd} head of the lock – 2 batches of 3 samples $\varnothing$94 mm (designation Г3.1–Г3.2).

A general view of structure construction with sampling areas is shown in Figure 3. After sampling, the samples were tested, and testing results are given in Table 1.

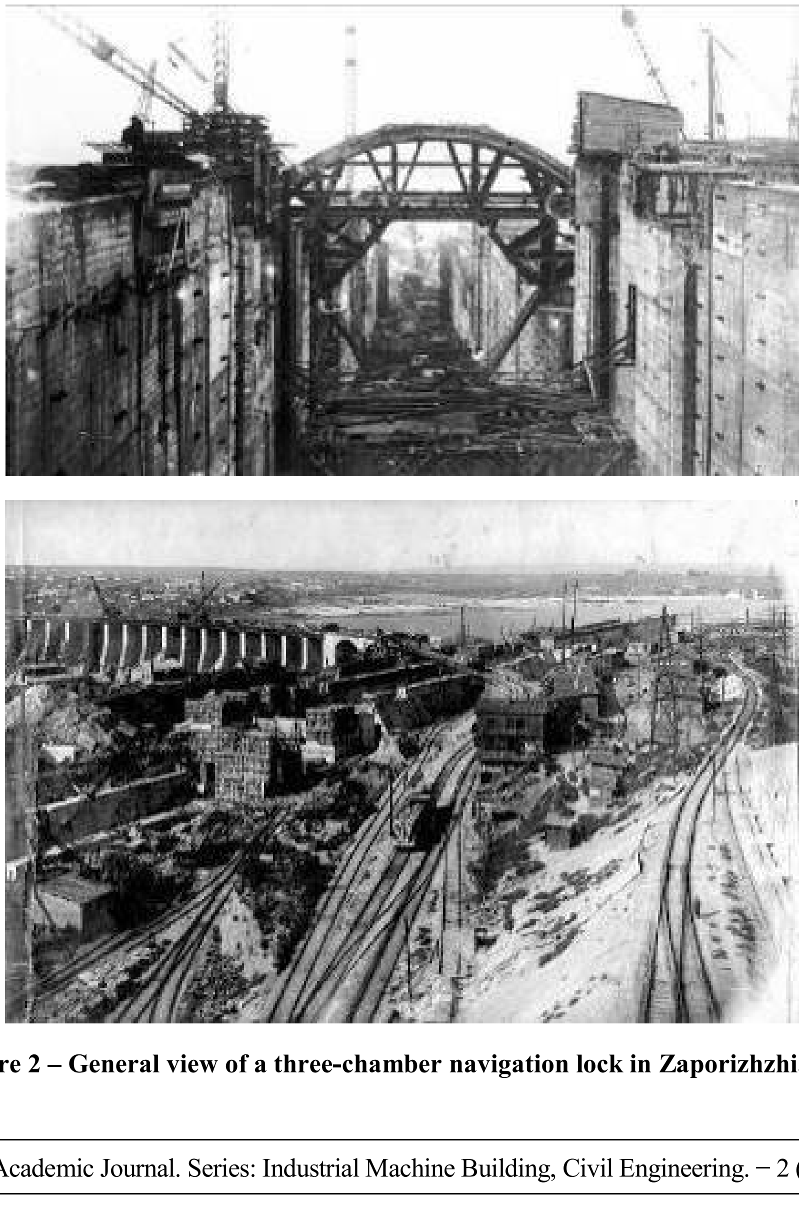


Figure 2 – General view of a three-chamber navigation lock in Zaporizhzhia city

Figure 3 – General view of construction site with sampling area

Table 1 – The results of concrete cores samples compression tests

№ batch of cores	Designation	Diameter, mm	Height, mm	η1	Cross-section area, cm²	Destructive load		α	Compression strength, given to the base size			
									one sample		average of 2 samples the largest on strength	
						kN	kgf		MPa	kgf / cm²	MPa	kgf / cm²
1	2	3	4	5	6	7	8	9	10	11	12	13
1	К1.1	94	82	0.96	69.36	492.30	50200	0.88	59.98	611.41	58.21	593.34
		94	83	0.96	69.36	455.04	46400	0.80	50.40	513.75		
		94	105	1.04	69.36	427.58	43600	0.88	56.43	575.28		
2	К1.2	94	85	0.96	69.36	411.89	42000	0.88	50.18	511.54	47.49	484.13
		94	90	1.00	69.36	323.62	33000	0.96	44.81	456.73		
		94	87	0.96	69.36	355.01	36200	0.88	43.25	440.90		
3	К1.3	94	86	0.96	69.36	415.81	42400	0.88	50.66	516.41	52.71	537.32
		94	97	1.00	69.36	431.50	44000	0.88	54.76	558.23		
		94	95	1.00	69.36	376.58	38400	0.88	47.79	487.18		

According to the tests, significant indicators of concrete selected samples strength in compression were identified.

When performing inspection work, a number of defects and damage was detected, which totality determined the technical condition of object construction structures:

• wharf and upper and lower canals; – satisfactory (category 2 [2]) with areas unsuitable for normal exploitation (category 3 [2]);

• the heads of the lock – satisfactory (category 2 [2]) from the pre-emergency area (stoplogs) (category 3 – 4 [2]);

• lock chambers – satisfactory (category 2 [2]) with areas unsuitable for normal exploitation (category 3 [2]);

• cuverts – satisfactory (category 2 [2]) with areas unsuitable for exploitation (absorber) (category 3 [2]);

• gate shafts – satisfactory (category 2 [2]) with in pre-emergency areas (staircases) (category 3 – 4 [2, 4])

• premises of the mechanisms of the kinetic energy absorbers of ships during the invasion - unsuitable for normal exploitation (category 3 [2]) with emergency (front cover) sections (category 3 – 4 [2]);

• the premises of the mechanisms of kinetic energy absorbers of ships during the invasion - unsuitable for normal exploitation (category 3 [2]) with pre-emergency areas (reinforced concrete floor) (category 3 – 4 [2]);

• turning bridge premises – unsuitable for normal exploitation (category 3 [2]);

• the drainage path from the river side consisting of 3 cameras – unsuitable for normal operation (category 3 [2]);

• control panel towers – satisfactory (category 2 [2]) with areas unsuitable for normal exploitation (category 3 [2]);

• unitized transformer substation building – satisfactory (2 category [2]);

• cable channels – unsuitable for normal exploitation (category 3 [2]).

The qualitative characteristic of structure technical condition is its deformed state. Considering that inspection object belongs to hydrotechnical category, significant influence on it is done by filtration processes in soil in-situ, with which it works along the line of hydraulic unit pressure front.

The customer provided with observational data on structure deformations, groundwater fluctuations, as well as deformations of adjacent sections of lock chamber walls. For optimal interpretation of the provided observational data, dependency graphs were constructed.

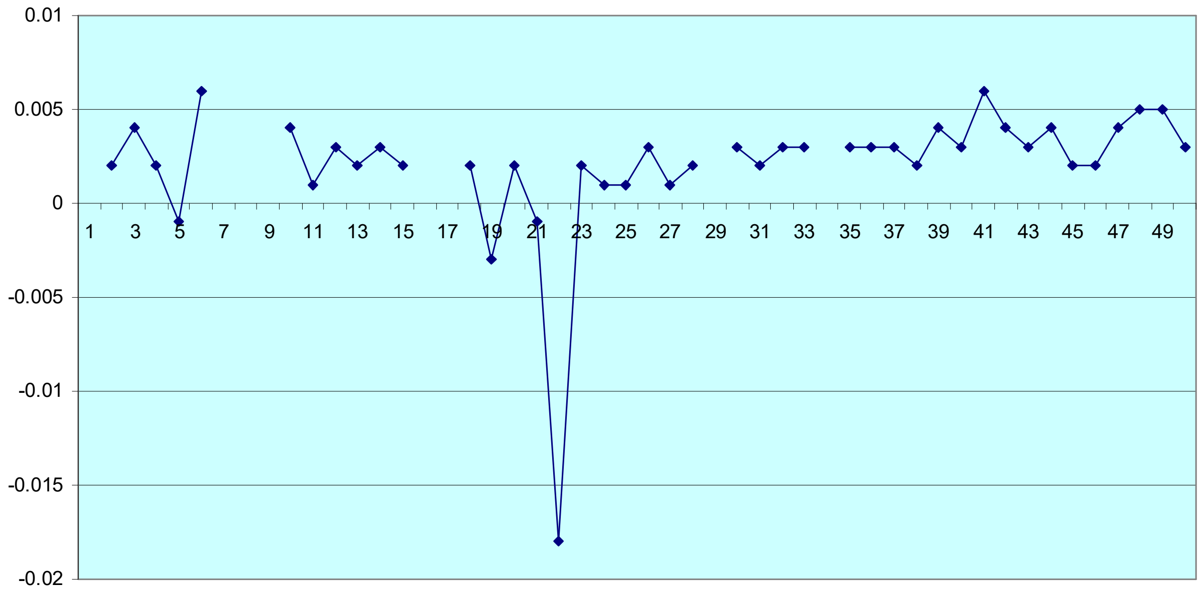
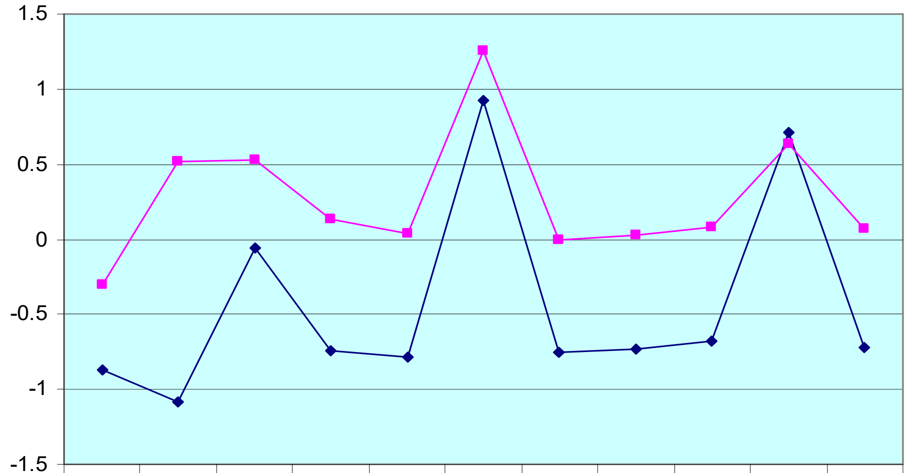
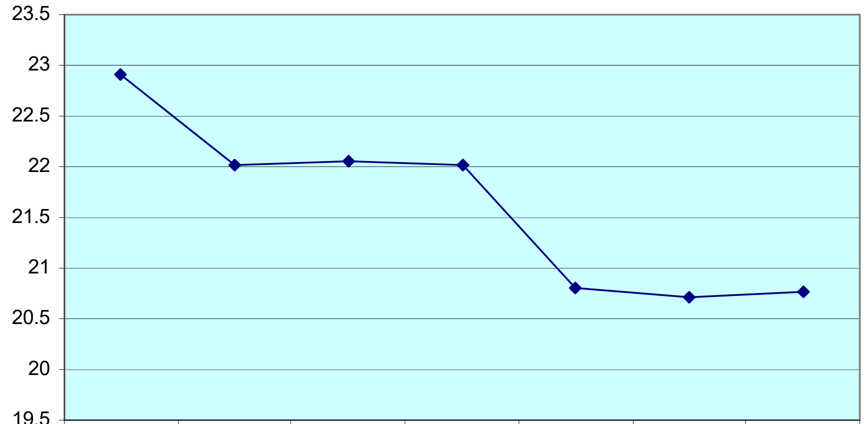


Figure 4 – Graph of deformation control benchmark change of three-chamber lock during the period from 1975 to 1985 (on the graph 1 ... 50 – mark numbers 0.01 ... 0.02 – change of marks in [m], breaks of the chart in the places of marks destruction)



	13.04. 2004р. (+15).	06.07. 2004р. (+22)	06.10. 2004р. (+14)	13.01. 2005р.	26.01. 2016р. (-10)	11.04. 2016р(+17)	12.07. 2016р. (+28)	25.10. 2016р. (+4)	25.01. 2017р. (0)	12.04. 2017р. (+14)	18.07. 2017р. (+25)
По осі Х	-0.87	-1.08	-0.06	-0.74	-0.78	0.92	-0.75	-0.73	-0.68	0.71	-0.72
По осі У	-0.3	0.52	0.53	0.13	0.04	1.25	-0.01	0.03	0.08	0.64	0.07

Figure 5 – Deformation graph along the joint 2-04 (on the graph: 1.5 ... -3 – step value in [mm]; 13.04.2004 – date of data readout; +28 ... 10 – outdoor air temperature)



	1987р. (січень)	1987р. (квітень)	1987р. (липень)	1987р. (вересень)	2017р. (січень)	2017р. (квітень)	2017р (липень)
Level of soils water	22.91	22.01	22.06	22.01	20.81	20.71	20.76

Figure 6 – Graph of groundwater level change by piezometers TR-3 (on the graph: 23.5 ... 19.5 – water level mark in [m]; 1987 ... 2017 – date of data readout)

For object deformed state evaluation, observation data analysisof object deformations and the fluctuations of groundwater has been carried out, which results are the following:

– the leveling data given by customer do not reflect inspected object deformed state;

– unidirectional deformations of adjacent sections during the observation period have not been recorded; the revealed increase of the measured values basically indicates concrete anays temperature deformation;

– for the period from 1987 – 2017 ground water level has decreased, which is due to the stopping of three-chamber lock exploitation and new depression curve formation.

The results of the inspection provided recommendations for further exploitation and the possibility of completing the reconstruction started in 1993, considering existing normative documents [3, 5, 7, 8]:

1). Reconstruction work should be carried out after defects and damage elimination recorded during the examination.

2). The work should be conducted on a separate project, developed by a specialized organization in the presence of operational plan.

3). The project should provide for section of work in-depth monitoring hydraulic engineering state required in accordance with [5].

4). When carrying out the work it is necessary to conduct thorough supervision of the author with the compilation of all necessary executive documentation.

5). To conduct scientific and technical support of works in accordance with the requirements [3, 7].

6). After the reconstruction (before exploitation) and lock chambers filling, perform a detailed inspection and evaluation of drainage system work with compilation of relevant acts and report on investigation results.

7). To establish permanent geodetic control of the construction before reconstruction, during reconstruction, and after commissioning. To do this, it is necessary to develop a geodetic monitoring program.

8). After reconstruction, before exploitation, object detailed inspection has been performed in accordance with the requirements [5].

9). The structure should be equipped with an automated system for monitoring the technical condition in accordance with current standarts requirements [3].

Conclusions. The issue of hydraulic structures inspection on the example of a three-chamber navigation lock in Zaporizhzhia city is considered. Problem (lack of regulatory documents) in the inspection of hydraulic structures is revealed. The analysis of observation data for three-chamber gateway structure deformed state is given. Difficult places to diagnose the technical condition of such structures are determined. Building structures and object technical condition is given. According to the performed research, recommendations for further exploitation and reconstruction completion possibility are given. Based on the recommendations, project was developed to restore damaged structures and complete the reconstruction of a three-chamber navigation lock in Zaporizhzhia.

References

1. ДБН В.1.1-45:2017. (2017). *Будівлі і споруди в складних інженерно-геологічних умовах. Загальні положення.* Київ: Мінрегіонбуд України, Укрархбудінформ.
2. ДСТУ-НБВ.1.2-18:2016. (2017). *Настанова щодо обстеження будівель і споруд для визначення та оцінки їх технічного стану*. Київ: ДП УкрНДНЦ.
3. ДБН В.1.2-14-2008. (2009). *Загальні принципи забезпечення надійності та конструктивної безпеки будівель, споруд, будівельних конструкцій та основ*. Київ: Мінрегіонбуд України, Укрархбудінформ.
4. ДСТУ Б.В.2.6-210:2016. (2017). *Оцінка технічного стану сталевих будівельних конструкцій, що експлуатуються*. Київ: Мінрегіонбуд України, Укрархбудінформ.
5. ДБН В.2.4-3:2010. (2011). *Гідротехнічні споруди. Основні положення*. Київ: Мінрегіонбуд України, Укрархбудінформ.
6. Наказ № 252 [1995.12.21]. (1995). *Про затвердження Методики обстеження і паспортизації гідротехнічних споруд систем гідравлічного вилучення та складування промислових відходів*. Київ: Держбуд України.
7. ДБН В.1.2-5:2007. (2008). *Науково-технічний супровід будівельних об'єктів.* Київ: Мінрегіонбуд України, Укрархбудінформ.
8. ДБН В.2.6-98:2009. (2011). *Бетонні та залізобетонні конструкції. Основні положення*. Київ: Мінрегіонбуд України, Укрархбудінформ.
9. Fell R., MacGregor, P., Stapledon, D., Bell, G. & Foster, M. (2015). *Geotechnical Engineering of Dams*. London: CRC Press. eBook ISBN 9780203387313
10. FEMA (2006). *Conduits through Embankment Dams. Best practices for design, construction,problem identification and evaluation, inspection, maintenance, renovation and repair*. US Federal Emergency Agency.

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Experimental research of retaining walls with structural surface

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The retaining walls are one of the most widespread types of engineering structures. Behaviour numerous studies of various soils with soaking have showed that their bearing capacity and compliance are closely related to their moisture content degree. To obtain information on the displacements and sediments of model structures and grounds, the hour-type indicators are used. The carried out researches have shown that with the same ground base, loading and boundary conditions, evident for a retaining wall with a structural surface, there is an inclusion in entire soil massif work. The uniformity of the structures and the ground base general deformations, in turn, provides retaining wall with a structural surface greater stability.

Keywords: experimental results, retaining wall, structural surface

Експериментальні дослідження підпірних стін зі структурною поверхнею

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Рассмотрено проектирование в условиях дефицита городских территорий, которое требует от инженера комплексного подхода для решения задач надежной эксплуатации зданий и сооружений и сохранения окружающей среды. Установлено, что подпорные стены – один из наиболее широко распространенных видов инженерных сооружений, которые нашли применение в строительстве. Изучено многочисленные исследования поведения различных грунтов при замачивании, при этом зафиксировано, что их несущая способность и податливость (жесткость) находятся в тесной зависимости от степени их влажности. Отмечено, что повышение влажности сопровождается снижением жесткостных характеристик основания, что может вызвать неравномерное оседание. С целью определения оптимальных конструктивных параметров предложенной конструкции подпорной стены со структурной поверхностью проведено экспериментальные исследования. Для получения информации о смещениях и осадках модельных конструкций и основания использованы индикаторы часового типа ИЧ-10, прогибомеры 6-ПАО. Определено, что при одинаковом грунтовом основании (геометрия слоев и физико-механические характеристики), нагрузке и граничных условиях очевидным для подпорной стенки со структурной поверхностью есть включение в работу всего массива грунта и равномерное перераспределение напряжений на контакте по лицевой и фундаментной плитам. Равномерность общих деформаций конструкций и грунтового основания, в свою очередь обеспечивает большую устойчивость подпорной стены со структурной поверхностью, а также повышает несущую способность основания за счет возникновения «арочного» эффекта (образование разгрузочных сводов и упругих ядер).

Ключові слова: результаты эксперимента, подпорная стена, структурная поверхность

Introduction. In connection with a significant increase in investment in the construction industry and, accordingly, production volumes in the conditions of a deficit in urban areas, especially in the last decades of Ukrainian economy development, the use of sites with complex terrain and hydrogeological situation has sharply increased. Design in such conditions requires the engineer to take an integrated approach to solve problems of buildings and structures reliable operation and preserve the environment, and construction on unsuitable territories is associated with solving social, economic and environmental issues.

The retaining walls are one of the most widespread engineering structures types, which have found application in industrial, civil, urban, road and railway construction. To the arrangement of the retaining walls, a number of requirements are presented, most of which are based on the territory geotechnical conditions study, which requires engineering protection.

According to expert estimates, 90% of Ukraine territory is characterized by complex engineering and geological conditions, deteriorating due to the impact of natural and man-made factors [1].

Analysis of the latest sources of research and publications. The current normative documents recommend that calculations for determining the walls position stabilityagainst shear, tilting, turning, determining base local strength and its load-carrying capacity should be carried out in calculating the retaining walls, structural elements and joints strength should be ensured. Calculations should be made on base deformations. But in the conditions of extra work areas and subsidence grounds, it is not always possible to implement the available technical solutions, since they are not suitable for working conditions. The existing retaining wall designs are not designed for additional forces from horizontal soil displacement, which causes stress concentration in the lower part of the faceplate, which leads to structure destruction [2, 3].

Behaviour numerous studies of various soils (loess subsidence, gypsum, hacked, karst, etc.) with soaking showed that their bearing capacity and compliance (rigidity) are closely related to the degree of their moisture content. At the same time, an increase in humidity is accompanied by a decrease in the base rigidity characteristics, which can cause uneven subsidence [4, 5].

Allocation of previously unresolved parts of a common problem. Experimental studies have shown that the stress-strain state of the substrate is largely determined by the structure, operating conditions and loading characteristics [2]. In this regard, there is a need to develop new design solutions for retaining walls capable of perceiving additional impact from an unevenly deformable base.

Formulation of the problem. The aim of the work is obtained during the experiment on the retaining walls with a structured surface study results processing and analysis. To process the experiment results, a software package MS Excel has been used.

In order to determine the optimal design parameters of a retaining wall with a structural surface proposed design, and to identify the qualitative patterns of its joint work with the base, an experiment was conducted.

The experiment was carried out on small-scale models in a specially designed chute (fig. 1). At modeling the method of the expanded similarity, where geometrical, mechanical and power analogues with a real object are maintained [6, 7], was applied.

a) *b*)

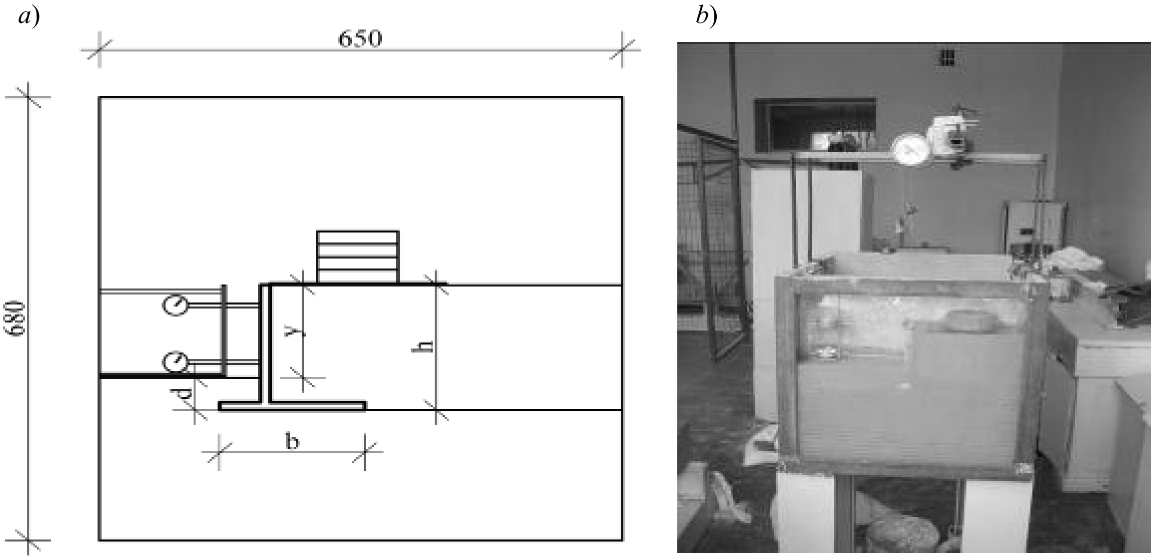

Figure 1 – **Experimental research:**
a) scheme for installing a retaining wall in the tray;
b) general view of the tray with installed wall and measuring devices

As a base soil in the models, a loam of broken structure was used. To create a uniform foundation, the soil was dried to a full loss of moisture and crushed by grinding in a mortar to a powdery state. Then the resulting powder was sieved through a sieve with a hole diameter of 0.5 mm. Considering the necessary soil moisture content, its density and volume determined the necessary amount of powder and water for its moistening. Humidification was carried out by a nebulizer with mixture constant stirring. Paste base was laid in layers of 15 mm, the compaction was carried out by a rammer made in the form of a rod with a welded base of a square cross section of 200 g. The purpose of preparing the model base was to obtain physicomechanical characteristics similar to natural soil.

Loams with the following characteristics were simulated (E = 13,5 MPa, c = 19,5 kPa, γ = 1,82 t/m^3, φ =22°).

The base model physical and mechanical properties were determined using the PLL-9 field laboratory in accordance with the methodology [8]. The strain modulus was determined with the help of a compression device of the Litvinov system. The coefficient of soil cohesion and the internal friction angle were determined by means of a shear device P10-S. Sampling was carried out from a tray with a step height of 150 mm, the results of certain characteristics are presented in (table 1), soils comparative characteristics are presented in (table 2).

Structural features of the retaining wall with a structural surface: a monolithic retaining wall of the angular type, which has voids on the contact surface of vertical and foundation elements, in the form of truncated pyramids of the same size and directed in a smaller base into the interior [9, 10]. With the development of the deforming load in time, that is, with soil vertical and horizontal movements with respect to the monolithic wall of the angular type, after its installation, soil gradual penetration into voids occurs.

Table 1 – Sample test results

№ point	The depth of sampling from the top of the array, m	Volume weight, γ, т/m^3	Modulus of deformation, E, MPa	Shift parameters		
				tgφ	φ°	c, MPa
1	2	3	4	5	6	7
1	0,15	1,826	9,3	0,38	21	0,014
2	0,15	1,812	9,5	0,32	18	0,019
3	0,3	1,831	8,9	0,36	20	0,016
4	0,3	1,822	9,1	0,46	25	0,012
5	0,45	1,816	8,5	0,48	26	0,009

Table 2 – Comparative characteristics of soils

Name of soil	Physicomechanical characteristics of the base			
	E, MPa	c, кPa	γ, т/m^3	φ, deg.
Full-face soil	13,5	19,5	1,82	22
Model base	5,62	6,8	1,71	22
Coefficient of transition	1/1,5	1/1,5	1	1

Premature filling of voids is prevented by sheets of elastic material. As a resiliently compliant material, a polyethylene film was used with the following characteristics: thickness of 200 μm, density of 916 kg / m^3, tensile strength of 165 kgf / cm^2.

Models of retaining walls were made using the method of layer-by-layer creation of a physical object using digital 3D model (fig. 2, a, b). For this, 3D printer Graber i3 was used.

The tray tests were carried out in a metal tray with a transparent front wall made of plexiglas. The tray dimensions are 600 × 650 × 680 mm. Its edges are made of corners 80 × 80 mm, the upper belt of steel strips with a width of 50 mm. All facets except for the front are made of chipboard 16 mm and rigidly fixed by two corners 60 × 60 mm. The working space for installing the retaining wall is fenced off by a partition of 16 mm chipboard. To prevent soil friction against tray wall, walls inner part was covered with an easily deformable polyethylene film in two layers with a layer of technical petroleum jelly [11].

The purpose of the first series of tests was to identify structural factors influence degree on the model bearing capacity of the proposed retaining wall of a special type.

The second series of tests ws conducted to compare anti-shear position stability of the retaining wall of the corner type and a retaining wall of a special type.

a) b)

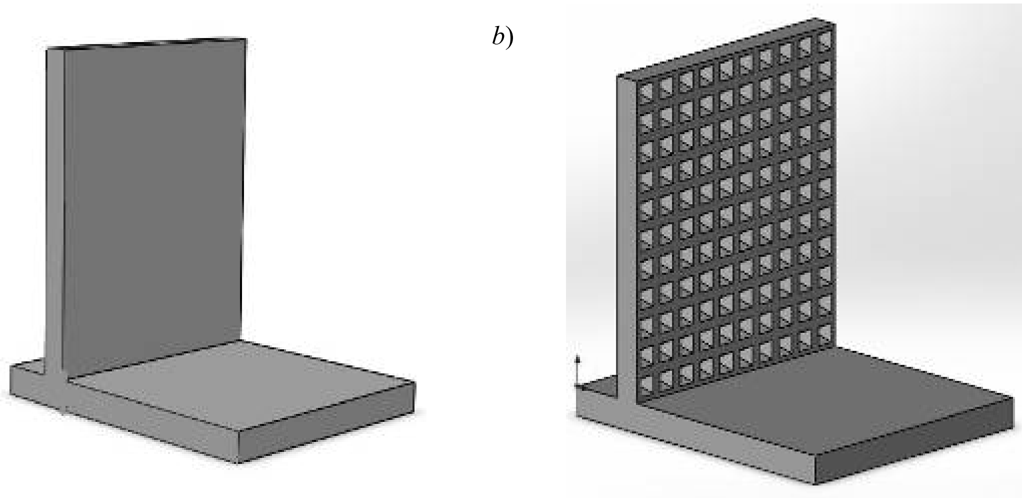

Figure 2 – **Experimental models:**
a) angled retaining wall; b) retaining wall with structural surface

Tray reparation for testing was carried out by installing it on the supports and the inner surface was covered with a polyethylene film. The second layer of polyethylene film was laid after applying a layer of technical petroleum jelly.

The base model preparation was carried out by layer-by-layer laying of pre-prepared ground paste. Each layer of paste was compacted by a rammer. When reaching a pre-marked height, the ground surface was planned, then a retaining wall was installed. Further, soil paste layered laying from the front side of the vertical retaining wall element continued until the mark indicated on the retaining wall. Backfilling was also performed using paste, the paste was laid layer by layer with a seal of 0.95 from the base under the foundation element to the top face of the retaining wall. A metal plate with dimensions of 150 × 200 mm was laid on the planned backfill surface.

To obtain information on the displacements and sediments of model structures and grounds, hour-type indicators IH-10, 6-PAO deflectometer were used, which were verified in the center of metrology, standardization and certification. Before retaining wall free surface, a bar with two clock-type indicators (fig. 3a) was rigidly installed to measure wall horizontal deformations (displacements) in two levels. Over the retaining wall, on a specially prepared console, there were installed deflectors (fig. 3, b), for measuring vertical deformations (displacements). All instrument readings were set to the initial values and recorded in the log.

Load on the platform was created in steps of 1.5 kPa. The load was maintained until soil conditioned stabilization. The model sedimentation rate, which does not exceed 0.1 mm in 30 min, was taken as a criterion for deformation conditional stabilization. Each subsequent stage of pressures was also maintained during the time of conditional stabilization.

Models loading models was carried out until the full loss of retaining walls stability. The devices values were recorded, then the graphs were constructed.

a) b)

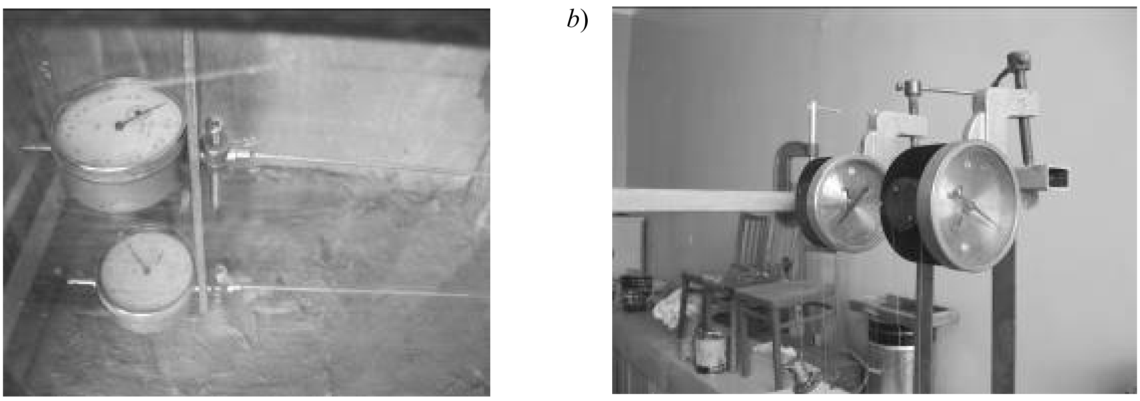

Figure 3 – **Measuring instruments:**
a) dial indicators IH-10; *b)* deflectometer 6-PAO

For retaining wall of angular type, the indications of hour-type indicators and deflectors indications were recorded (fig. 4, a, b); for the retaining wall with the structural surface, the indications of the hour-type indicators and deflectors indications were recorded (fig. 5, a, b).

Based on the data obtained, sediments of retaining walls (a corner wall type retaining wall, a retaining wall with a structural surface) were built (fig. 6).

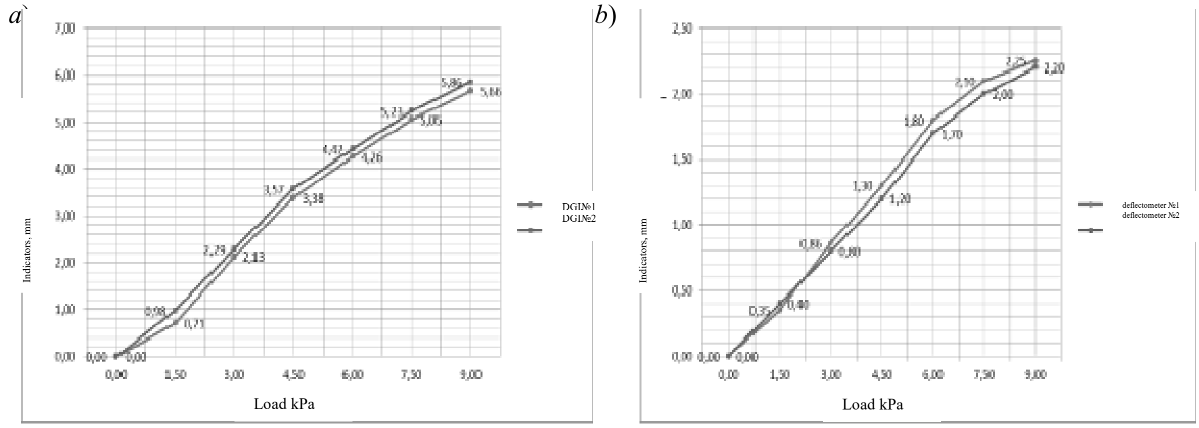

Figure 4 – Charts: *a)* indication of dial gauge indicators; *b)* indications of deflectometer

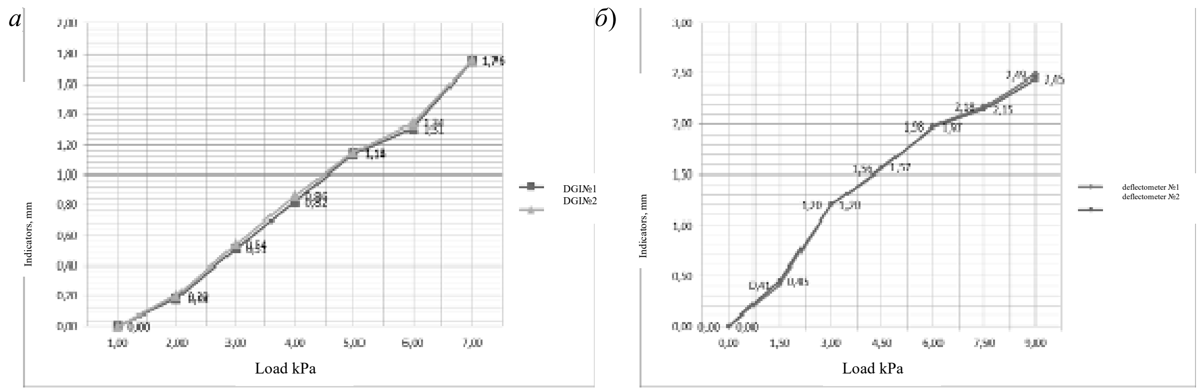

Figure 5 – Charts: *a)* indication of dial gauge indicators; *b)* indications of deflectometer

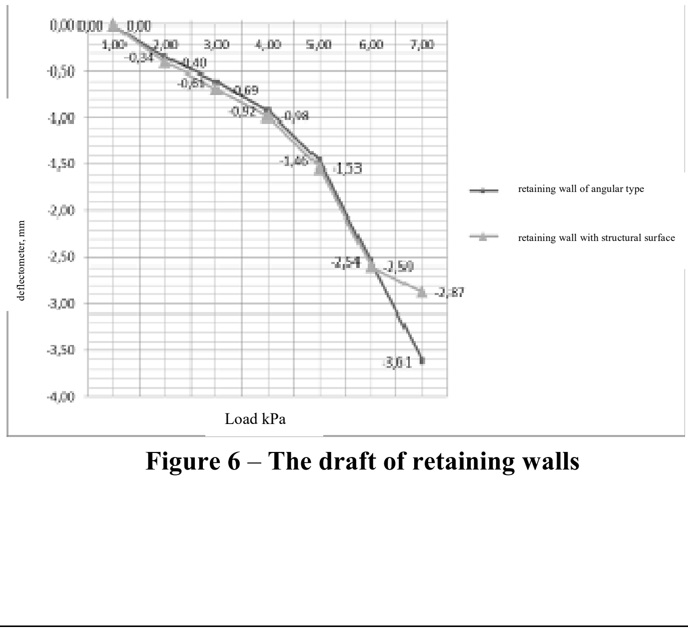

Figure 6 – **The draft of retaining walls**

It can be obvious from the graphs on (Figure 5) that the readings of hour-type indicators and deflectors have more similar indications than on the graphs (Figure 4), which indicates that the retaining wall with the structural surface moves and settles more evenly.

These graphs (Figure 6) show that the retaining wall with a structural surface has a large draft (by 18%) than the retaining wall of the corner type at the initial stage of loading the working platform, this indicates a gradual penetration of the soil into voids. With the passage of time, when the load increases, precipitation stabilization is recorded. With increasing load, retaining wall with the structural surface draft decreases, which is evident in the graph. In order to achieve the same draft of the retaining walls, 28% more load was applied to the retaining wall with a structural surface than to the retaining wall of the corner type.

Conclusions. The carried out researches have shown that with the same ground base (geometry of layers and physicomechanical characteristics), loading and boundary conditions, evident for a retaining wall with a structural surface, there is an inclusion in the entire soil massif work and uniform redistribution of stresses at the contact along the face and foundation slabs. The structures and ground base general deformation uniformity, in turn, provides greater stability of the retaining wall with a structural surface, and also increases base bearing capacity due to the appearance of an "arched" effect (the formation of unloading vaults and elastic cores).

References

1. Кривошеєв, П.І. (2001). Науково-технічні проблеми координації дій щодо захисту будівель, споруд і територій зі складними інженерно-геологічними умовами. *Будівництво України,* 6, 16-19.

2. Gubrynovicz, A. (1978). *Wychylenia z piony obiektow budow – lanych na tle nachylen tereny powodowanych wplywami eksploaji gornicze.* Conferencja naukowo-techniczna «Probleamy budow – nictwa na terenach gorniczych». Glivice, 75-82.

3. Gucunski, N., Najm, H. & Nassif, H. (2005). *Seismic analysis of retaining walls, buried structures, embankments, and integral abutments– Piscataway.* Dept. of Civil & Environmental Engineering Center for Advanced Infrastructure & Transportation (CAIT) Rutgers.

4. Тимченко, Р.А. & Кришко, Д.А. (2010). Работа плитных фундаментов-саморегуляторов (ПФС) на неравномернодеформируемом основании. *Современные проблемы строительства*. 8, 34-38.

5. Тимченко, Р.А. & Турабелидзе, Г.Л. (1989). *Работа саморегулирующихся фундаментов при заданных вертикальных деформациях основания.*Деп. в ВНИИС № 10159.

6. Иванов, Е.И. & Ерещенко, В.Е. (2015). *Методы подобия физических процессов.* Москва: МАДИ.

7. Гухман, А.А. (2016). *Введение в теорию подобия*. Москва: Издательство ЛКИ.

8. ДСТУ Б В.2.1-4-96 (ГОСТ 12248-96). (1997). *Грунти. Методи лабораторного визначення характеристик міцності і деформованості*. Київ: Держбуд України.

9. Тімченко, Р.О., Крішко, Д.А., Савенко, В.О. & Настич, О.Б. (2015). *Монолітна підпірна стінка кутникового типу* Патент України 100212 U. Київ: Державне патентне відомство України.

10. Тімченко, Р.О., Крішко, Д.А. & Савенко, В.О. (2016). Оптимизация конструктивного решения подпорной стены специального типа на основании линейной модели регрессии. *Вісник КНУ,* 41, 54-58.

11. Тімченко, Р.О., Крішко, Д.А. & Савенко, В.О. (2017). Методика експериментального дослідження підпірних стін спеціального типу. *Збірник наукових праць. Серія: Галузеве машинобудування, будівництво,* 2(49)-2, 221-226. https://doi.org/10.26906/znp.2017.49.846

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Mathematical modeling of the folded foundation interaction with the base by varying the structure stiffness

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The article describes a practice of using software system based on the finite element method for calculating shell foundations. It considers the peculiarities of the folded foundation interaction with subsoil mathematical modeling . It is found that during modeling, special attention should be paid to a purpose of the system initial parameters, to a choice of finite elements type, and to an optimal model of subsoil. Mathematical modeling of the folded foundation interaction with subsoil is performed under conditions of a plane problem. The foundation operation under various conditions of interaction with subsoil and with different stiffness parameters of the foundation is analyzed. Correlation between stiffness of a foundation structure and resulting equivalent stresses in subsoil under different conditions of interaction is determined. It is concluded that the obtained results represent a benchmark for subsequent calculations and modeling the interaction between a foundation and subsoil under a volumetric stressed condition.

Key words: shell foundation, finite element method, mathematical modeling, coulomb-mohr model

Математичне моделювання взаємодії складчастого фундаменту з основою при варіюванні жорсткості конструкції

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В статті наведено досвід використання програмних комплексів на основі методу кінцевих елементів при розрахунках фундаментів-оболонок. Зазначено, що активне використання методу кінцевих елементів пов'язане поєднанням трьох факторів: особливостями самого методу, наявністю сучасної обчислювальної техніки, розробкою математичних моделей досліджуваних явищ, що адекватні до реальних процесів із високим ступенем точності. Наведено приклади найрозповсюдженіших програм для вирішення геотехнічних задач, при цьому зазначено, що вони відрізняються способами завдання моделі ґрунтової основи, та мають свої певні інструменти для зміни та коригування вихідних параметрів та аналізу. Розглянуто особливості математичного моделювання взаємодії фундаментів та ґрунтової основи. Встановлено, що при моделюванні особливу увагу слід звертати на призначення початкових параметрів системи, на вибір типу кінцевих елементів і оптимальної моделі ґрунтової основи. Було виконано моделювання взаємодії складчастого фундаменту з основою в умовах плоскої задачі за допомогою програмного комплексу LiraSapr-2013. Проаналізовано характер роботи фундаменту при різних умовах взаємодії з основою та при різних параметрах жорсткості фундаментної конструкції. Було встановлено залежність між жорсткістю фундаментної конструкції та виникаючими еквівалентними напруженнями в ґрунтовій основі при різних умовах взаємодії. Встановлено, що саме у випадку із використанням гнучкої складки, яка працює як оболонка, і, залучаючи до роботи більше об’єму ґрунту у порожнину, досягається ефект перерозподілу напружень по всій ґрунтовій товщі. Зроблено висновок, що отримані результати слугують орієнтиром для послідуючих розрахунків та моделювання взаємодії фундаменту та основи при об’ємному напруженому стані.

Ключові слова: фундамент-оболонка, метод кінцевих елементів, математичне моделювання, модель Кулона-Мора

Introduction. Finite Element Method (FEM) is actively used to solve various engineering tasks, geotechnical ones in particular. The development of technology along with a wide software use when designing and calculating engineering systems urges the integration of software systems based on FEM in subsoil modeling, its interaction with foundation, and in the system analysis in general [1-3].

There are various software systems based on the finite element method, for example: Feadam, Sage-Crisp, Plaxis, Ansys, LiraSapr, Nastran, ABAQUS, etc. They differ in methods of subsoil modeling, and have their own specific tools for changing and adjusting source parameters and analysis. At the same time, the application of a software system for modeling foundation structures of off-standard forms and types is also relevant, i.e. stamps modeling. Today, the practice of using software systems for modeling complex foundation systems in conjunction with subsoil is not studied sufficiently [4-6].

Analysis of the latest research findings and publications. The finite element method is widely used due to a combination of three factors: the features of the method itself, the availability of modern computer technology, the development of investigated phenomena mathematical models that comply with the real processes with a high degree of accuracy.

National and foreign scientists actively use software systems based upon the finite element method for solving geotechnical problems in their research [1-10]. First of all, in research analysis, attention is paid to the study of software systems application in mathematical modeling of different types of shell foundations while interacting with subsoil. Mahmoud Samir El-kady and Essam Farouk Badrawi conducted experimental and numerical studies applying five square foundations, one of them was a flat-shaped foundation used as a reference sample and four shell foundations of a folded shape [1]. ABAQUS software was used for mathematical modeling. In a numerical model, a body of sandy soil is described by an 8-node brick element of a trilinear displacement. The sandy soil is modeled as a resilient plastic material model with a non-associated flow rule using the Coulomb-Mohr plasticity model. Foundation supports are modeled as a plastic material using an 8-node linear brick element with reduced integration (fig. 1). The minimum deviation of experimental and numerical results was found in conclusion.

Nisha P. Naik and Sabna Thilakan [5] investigated the operation of a column type shell foundation analyzing various angles of lateral surfaces inclination. The models of a foundation strain and soil properties were numerically modeled with OptumG2 software using finite elements. The Coulomb-Mohr sand model available in OptumG2 was used to simulate three soil conditions: loose, medium, and dense states. The models were subjected to a multiplier resilient and plastic analysis. As a result, general stresses, maximum vertical displacements were obtained. The results showed that the foundation with shell configuration had a greater bearing capacity than conventional flat slab foundation.

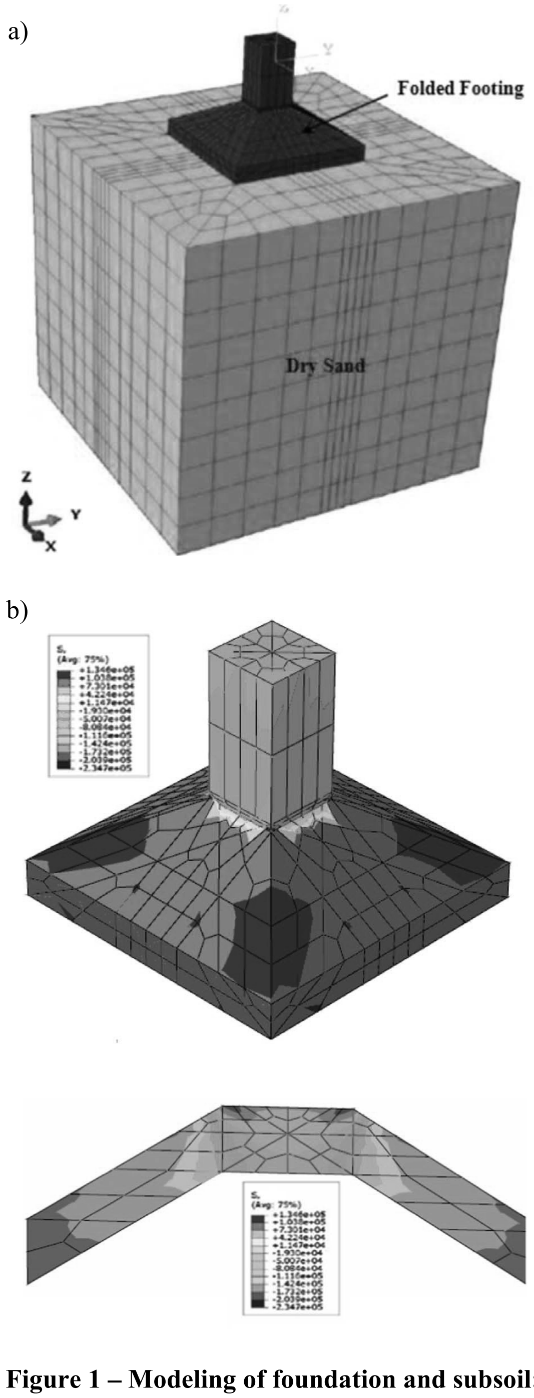

Figure 1 – Modeling of foundation and subsoil:
a) – finite element mesh of subsoil and foundation;
b) – shading contours of stresses for 30° folding angle

Mohammed Y. Fattah analyzed the operation of a conical shell foundation using ANSYS software system [7]. A three-dimensional brick element (Solid 65) was used to simulate concrete with or without reinforced bars. Steel reinforcing bars were represented using a 2-node bar (Link8 in ANSYS) and included in the property of 8-node brick element. The type of a

45 finite element is normally used for 3D modeling of solid structures. The element is determined by eight nodes with three degrees of freedom at each node. Using load symmetry, geometry and distribution of reinforcement, RPC-conical foundations were considered in the analysis of the finite element method. The selected segment was modeled using Solid 65 isoparametric hexahedral brick elements.

Chinese scientists Dongxue Hao, Rong Chen and Guangsen Fan were engaged in research on the ultimate bearing capacity of the foundations for power transmission towers on structurally unstable soils [9]. Mathematical modeling of subsoil and foundation was carried out in ABAQUS software system. The monolithic reinforced concrete foundation was modeled as a linear plastic material with a modulus of elasticity of 40 GPa and a Poisson coefficient of 0.2. The subsoil is considered as a homogeneous resilient and plastic material, which is described by the Drucker-Prager model. Resilient parameters: E = 30 MPa, ν = 0.3, γ=17 kN/m^3. The parameters of the Coulomb-Mohr subsoil model in a tensile stress state, specific soil cohesion and the angle of internal friction can be transformed into the parameters of the Drucker-Prager model. While conducting a finite element analysis of a deepened foundation with an extended basement near an enlarged base, large displacements and strains are inevitable. An application of traditional Lagrange elements in these areas may cause a decrease of their accuracy [10]. In contrast to the Lagrangian, Euler's analysis is a technique of finite elements in which material can flow through the boundaries of elements in a rigid mesh. Thus, Euler's technique can be very effective in processing tasks involving very large strains, material destruction, and liquid materials. In this paper, the Euler's method was used to study soil.

Determination of the unsolved parts of the general problem. One of the difficulties about mathematical modeling of folded foundations for power transmission lines is the modeling of joint operation of a foundation structure of a complex shape and subsoil, as it is necessary to choose a relevant subsoil model, consider all output parameters, simulate the operation of a foundation up to full involvement of soil into the operation.

Task setting. The research task is to analyze the operation and to select the optimal parameters of a folded foundation (stiffness) and subsoil (model choice) during mathematical modeling in LiraSapr-2013 software system.

Basic material and results. The research subject is a stressed and strained state of a foundation structure and subsoil under their contact interaction in specific operational conditions. A folded foundation is selected as a prototype of a new foundation shaped as separate thin-walled reinforced concrete folds joined with each other by a steel or reinforced concrete beam [11]. A folded foundation with an improved system of bearing beams and hinges is proposed as a new alternative solution [12].

Mathematical modeling of the interaction between a foundation structure and subsoil is carried out in the LiraSapr-2013 software system. As a test model, one of the foundations prismatic folds with known geometric parameters and physical characteristics was modeled [12], further on one typical prismatic stamp operation was studied under altered operational conditions and interaction with subsoil. To study the stress-strain state of subsoil under different interaction types of "foundation-subsoil" system, the task was solved in a plane formulation.

A finite element of type 2, corresponding to the finite element (FE) of the flat frame, was used for modeling a prismatic fold (there are 3 degrees of freedom in each node: X – displacement along X axis; Z – displacement along Z axis; UY – rotation around Y axis). Finite elements 21 and 22, which are rectangular and triangular FE of a plane problem (beam-wall), were used to model subsoil. This finite element is intended for a strength calculation of the plates loaded in their X1OZ1 plane; a priori this FE allows modeling a plane stress state. At the same time, the lower part of 1/3 soil thickness was modeled by finite elements of a larger size to optimize the calculation process.

Two conditions of a foundation structure and subsoil interaction were compared. In the first case, a prismatic folded plate contacts subsoil only within the horizontal bearing flanges. In the second case, subsoil is fully incorporated into the operation by filling the fold cavity. Silty sand was chosen as a reference soil sample having the following characteristics: c = 2 kPa, e = 0.75, E = 11 MPa, φ = 26°.

It is common knowledge that structures the outlines of which are similar to the arched ones transmit horizontal loads to the bearing parts, which in turn provokes tensile stresses at the base of a fold. In order to reduce the spill negative impact, which is transmitted to the soil, the rigidity ribs (diaphragms) are arranged in the fold. To simulate the operation of the folds with diaphragms and a step of their arrangement, calculations with different stiffness of the fold for both cases of interaction were performed.

To determine the main and equivalent stresses, the Coulomb-Mohr theory was chosen (a subsoil model) as a strength theory for assessing a bearing capacity of subsoil. The calculation results of the mathematical models are shown in Fig. 2 and 3.

As it can be seen in Figure 2, in the case where the prismatic fold rests on the soil only through the horizontal bearing flanges and the load is transmitted to the soil through them, the stiffness of the fold increases while the equivalent stresses reduce. This is explained by the fact that, when the stiffness of the fold increases, it ceases to operate as a thin-walled structure losing the properties of a shell, and its operation with the increased stiffness is similar to the operation of a conventional hard stamp (a slab).

However, in the second case (Fig. 3), when the soil is fully integrated into the operation by filling it into the fold cavity, the stiffness of the fold increases along with the increase of the equivalent stresses. It suggests that the application of a flexible fold operating as a shell and the integration of a larger soil amount in the cavity lead to the redistribution of stresses throughout the soil stratum.

The graphs of equivalent stresses values dependence from the alteration of the foundation structure stiffness (Fig. 4) clearly shows the consistent pattern of operation under different conditions of interaction between the foundation structure and subsoil.

Conclusion. The scientific novelty of the research is in determining the correlation between the stiffness of a foundation structure (folds) and values of equivalent stresses under different conditions of interaction between a foundation structure and subsoil. In turn, this affects the bearing capacity of a foundation and subsoil in general. The best results are demonstrated during co-operation of a foundation and soil. It can be explained by the fact that soil and foundation in this case act as a whole. The obtained results represent a benchmark for subsequent calculations and modeling of the foundation and subsoil interaction under a volumetric stressed state.

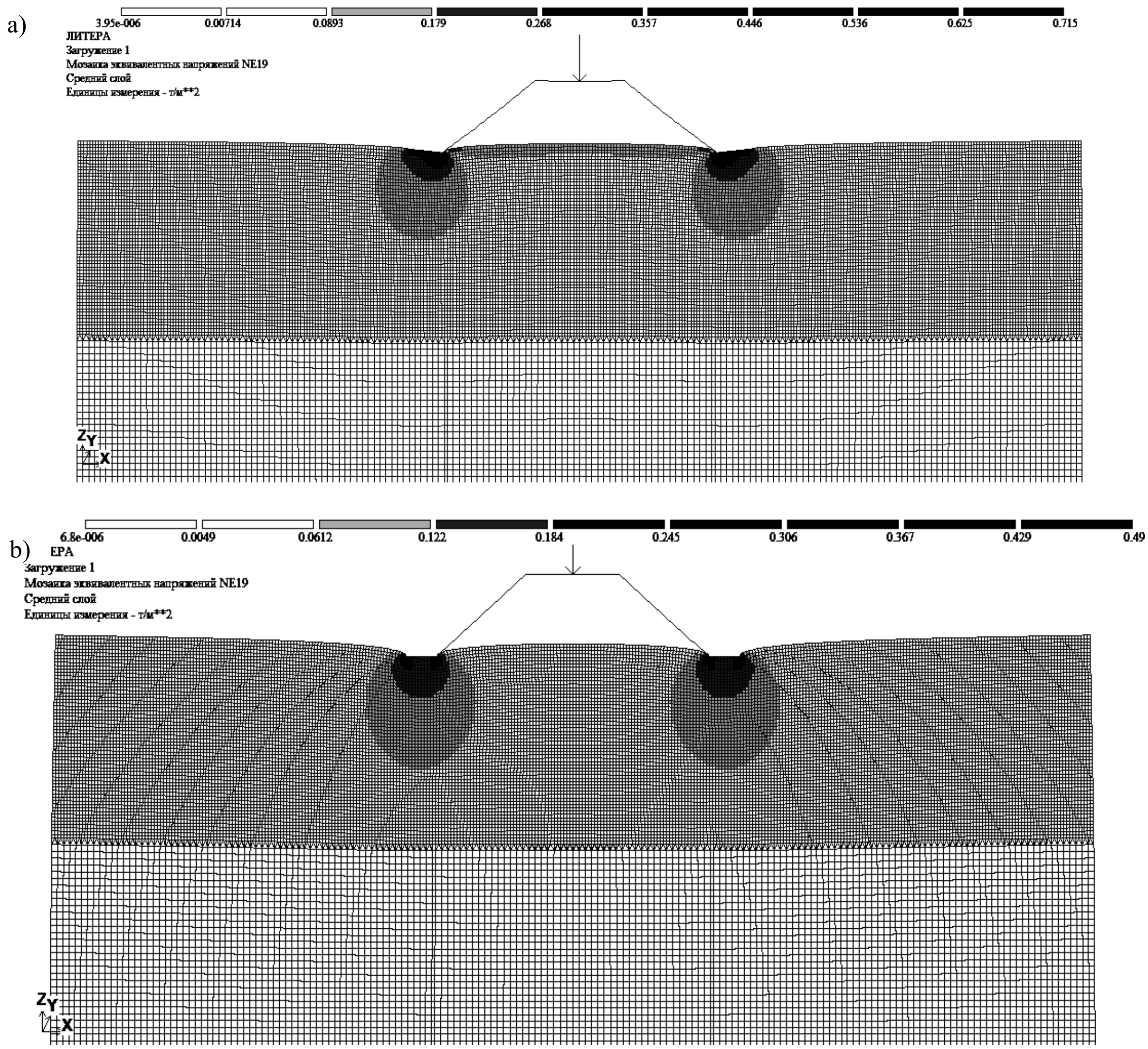

Figure 2 – Stresses shading contours under incomplete interaction of the foundation and the ground:
a) – the value of equivalent stresses at a coefficient of stiffness N=1;
b) – the value of equivalent stresses at a coefficient of stiffness N=4

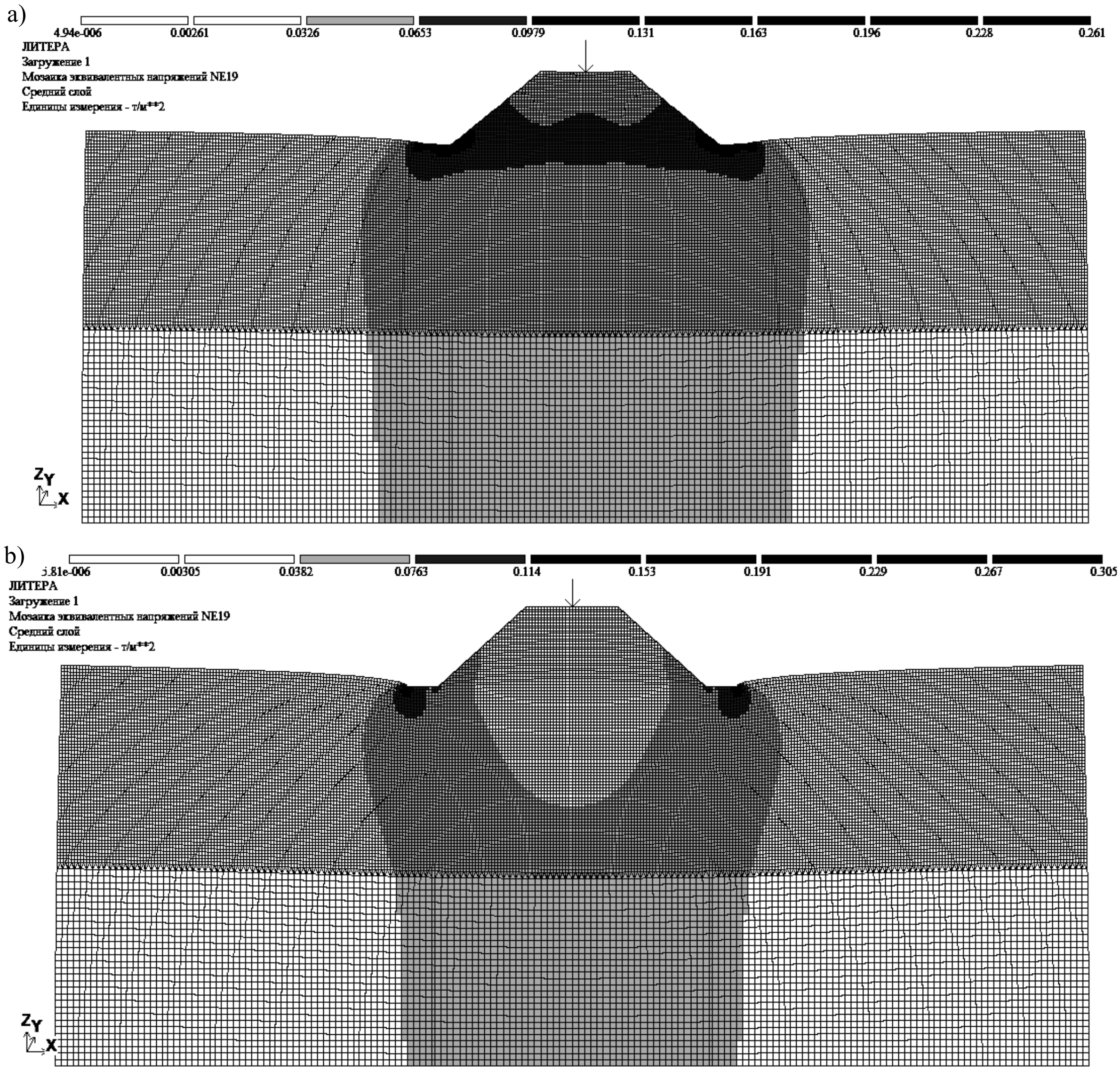

Figure 3. Stresses shading contours under complete interaction of the foundation and the ground
a) – the value of equivalent stresses at a coefficient of stiffness N=1;
б) – the value of equivalent stresses at a coefficient of stiffness N=4

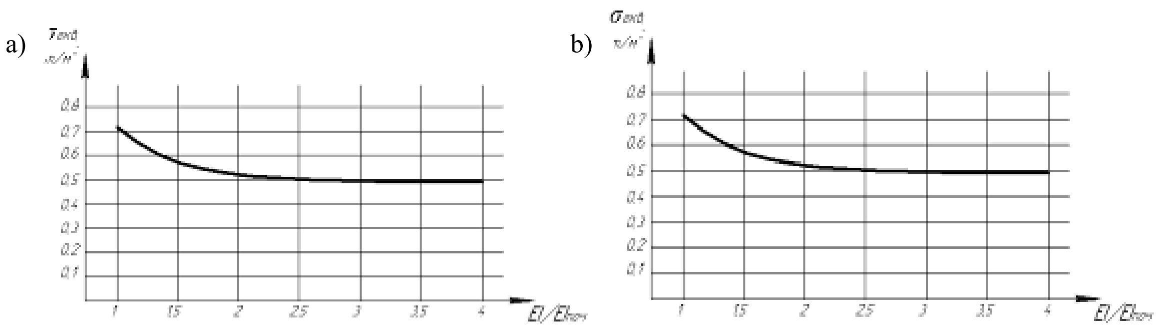

Figure 4. The graphs of equivalent stress values dependence from the stiffness change of the foundation structure

References:

1. El-kadya, M.S. & Badrawi, E.F. (2017) Performance of isolated and folded footings. *Journal of Computational Design and Engineering*, 4, 150-157. doi:10.1016/j.jcde.2016.09.001.

2. Пронозин, Я.А. & Епифанцева, Л.Р. (2011). *Экспериментальные исследования взаимодействия мембранного фундамента с грунтовым основанием.* Всероссийская науч.-практ. конф. «Стратегия инновационного развития, строительства и освоения севера», Тюмень.

3. Kurian, N.P. (1994). *Behaviour of shell foundations under subsidence of core soil*. Proc. 13 Int. Conf. Soil Mechanics and Foundation Eng. New Delhi, India.

4. Huat, B.K.B. & Mohammed, A.T. (2006). Finite Element Study Using FE Code (PLAXIS) on the Geotechnical Behavior of Shell Footings. *Journal of Computer Science*, 2(1), 104-108.

5. Naik, P.N. & Thilakan, S. (2015). *Geotechnical behavior of shell foundations*. 50th Indian geotechnical conference. Pune, India.

6. Colmenares, J.E., Kang, S.R., Shin, Y.J. & Shin, J.H. (2014). Ultimate Bearing Capacity of Conical Shell Foundations. *Structural Engineering and Mechanics*, 52(3), 507-523.

7. Fattah, M.Y., Waryos, W.A. & Al-Hamdani, M.A.E. (2015). *The Behavior of Conical Shell Foundation under Dynamic Loads.* The 2nd International Conference of Buildings, Construction and Environmental Engineering (BCEE2-2015). Beirut, Lebanon.

8. Esmaili, D. & Hataf, N. (2008). Experimental and Numerical Invetigation of Ultimate Load Capacity of Shell Foundations on Reinforced and Unreinforced Sand. *Iranian Journal of Science & Technology*, 32(B5), 491-500.

9. Hao, D, Chen, R. & Fan, G. (2012). Ultimate Uplift Capacity of Transmission Tower Foundation in Undisturbed Excavated Soil. *Energy Procedia*, 17, 1209-1216. doi:10.1016/j.egypro.2012.02.228.

10. Tong, L.C. (1995). *FE Simulation of Bulk Forming Processes with a Mixed Eulerian-Lagrangian Formulation.* (PHD thesis), Swiss Federal Institute of Technology, Zurich.

11. Тетиор, А.Н. (1975). *Проектирование и сооружение экономичных конструкций фундаментов*. Киев: Изд-во «Будівельник».

12. Timchenko, R.A., Krishko, D.A. & Khoruzhenko, I.V. (2017). Construction solution of folded-plate shell foundation for power transmission towers. *Academic Journal. Industrial Machine Building, Civil Engineering.* 2(49), 207-214.

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Resistance of tubular piles shear silk along surfaces

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The results of piles tests in sea muddy soils underlain by loams and clays are presented at port berths in Odessa region. Piles length 28.0 and 34.0 m have been made of metal pipes with a diameter of 1400 mm with a wall thickness of 16 mm. From the sea water surface to the silt roof is 15.0 m. The lower end of the two tubular supports (piles) has been located above the silt sole that enables the test piles to be brought to "breakdown" and determine its resistance (silt) along the outer and inner surfaces. According to the test results, the piles are increased by 6 m that enables to use the underlying soils as a bearing layer. Tests of piles have been carried out according to the standard procedure, for the reference system, two additional tubular supports with a diameter of 1400 mm were placed in the vicinity of the test subjects.

Keywords: metal tubular pile, sea muddy soils, muddy soils resistance to shear along trunk surface

Опір зрушенню мула вздовж поверхонь трубчастих паль

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Наведено результати випробувань паль у морських мулких ґрунтах, що підстилаються суглинками і глинами, на причалу порту в Одеській області. Палі завдовжки 28 і 34 м виконано з металевих труб діаметром 1400 мм з товщиною стінок 16 мм. Від поверхні морської води до крівлі мулу – 15,0 м, товща мулу – 13,2 м. Нижній торець двох трубчастих опор (паль) розташовувався вище за підошву мулу, що дозволило довести випробовувані палі до «зриву» та визначити його опір по зовнішній і внутрішній поверхнях. За результатами випробувань палі нарощено на 6 м, що дозволило використати як несучий шар розташовані нижче ґрунти. Випробування паль здійснено за стандартною методикою, для реперної системи було занурено додатково дві трубчасті опори діаметром по 1400 мм біля випробованих. Згідно з розрахунками середнє значення граничного опору зрушенню для двох паль, які розташовані в морських мулких ґрунтах, склало 17,3 кН/м^2 (17,3 кПа) і 15,4 кН/м^2 (15,4 кПа). Для порівняння дослідження опору обводнених лесовидних супісків показали, що їх величина трохи вища за морський мул і складає 20 кПа. Для визначення опору ґрунтів основи по поверхні паль застосовано методику циклічно-зростаючого навантаження. Граничне навантаження, урівноважене опором зрушенню паль, заглиблених у мул, із зовнішнього і внутрішнього боків склали 1980 та 1760 кН. З'ясовано, що після нарощування палі несуча здатність збільшилася за рахунок опору нижче торцевої частини. При повному навантаженні P=3000 кН осідання палі склало 5,45 мм. Установлено, що граничний опір ґрунтів зрушенню по поверхні стінок опори P_f = 1909 кН, а опір ґрунту під торцем стінок палі P_R = 1091 кН. Виявлено, що в процесі проходки трубчастої палі її стінками «прорізається» шар мулистого ґрунту.

Ключові слова: металева трубчаста паля, морські мулисті ґрунти, опір мулистих ґрунтів

Introduction. Designing the shallow foundations and pile foundations for buildings and structures, when there are engineering-geological elements from mud grounds in the earth cover, as a rule their resistance to the loads is usually not considered. Under the heavy thickness of these soils, knowing their resistance value, it is possible to partially reduce the effect of the load on the bearing soil layer.

Analysis of recent sources of research and publications. Constructing buildings and structures in the cities of Riga, Odessa and Sukhumi, K. Ehorov noted the special properties of highly compressible soils. At large thicknesses, the arrangement of sand cushions is not effective, they need to be cut with jointed piles.

Selection of previously unsettled parts of the general problem. In the tests, the resistance values of silk on the outer and inner surfaces of metal tubular piles were determined.

Task definition. To carry out the full-scale studies of the marine mud resistance.

Basic materials and results. Resistance characteristics of foundation soil to the loads, transmitted by pile, are shear strength over the shaft surface and the compression below the toe. Studying stress-strain state, conducted in the field conditions with the use of special equipment, two methods of soil resistance determining over the pile surface are used:

– Integral one, based on the average determination of soil shear strength over the entire surface;

– Differential one, which is based on soil shear strength determination in some spots with the help of strain gauges [1].

The applied technique of cyclic-increasing load enables to determine the soil resistance of the foundation over the pile surface [2, 3, 4]. It considers pile shaft elastic compression from each load stage without special equipment use.

The soil shear strength over the pile surface*(f)* appears after the load application. It is balanced on the compressible part of the length by shear resistance that depends on the soil properties. The elastic compression along the pile length occurs consistently with load increasing. A part elastic compression $s_{y,i}$ of the length $l_{f,I}$ corresponds to each load stage P_i. Within this length, the applied load is balanced by the shear resistance. Experiments proved that the shear resistance limit value, at the consequential increasing of the load, is retained in pile previous sections.

Elastic deformation consists of elastic-instantaneous and elastic-viscous parts. The elastic-instantaneous part is almost 90% of the total one and it disappears almost after the load is removed, and deformation elastic-viscous part relaxation lasts several hours. In the performed tests, deformation elastic-instantaneous part values were used.

The method of research is :

1. Piles tests for vertical or pulling static loads have been carried out in steps until the conditional stabilization of the settlement(*s*) is reached.

2. Once the stabilization of the settlement has been achieved and measured, the load is removed (reset to zero). The settlement residual part is measured at this stage (s_o).

3. The difference between settlements of stabilized stage (*s*) and residual one (s_o) is the elastic part (s_y).

4. The dependence graph $s_y = f\ (P)$ is constructed, where the breaking point of the line divides it into two branches and shows the size of the soil resistance (P_f) along pile ultimate load surface.

5. Shear resistance value (*f*) is determined as the quotient by dividing the load corresponding to the ultimate shear resistance (P_f) on the shaft surface area (A_f).

It is not always possible to use the method of cyclic-increasing load to determine the elastic settlement, so with some error it is possible to determine the ultimate load of soil resistance to the shear (P_f) on the dependency graph $s= f\ (P)$, lengthening two branches of the graph to their intersection.

At one of the port berths in Odesa region, the metal tubular piles were tested in mud ground, underlain by loam and clay. The ground conditions of the construction site are represented by the following layers (Fig. 1).

The averaged values of the soil physico-mechanical characteristics for each EGE are given in Table 1.

The pentration of metalpipes (piles) with the diameter of 1400 mm, the depth of 16 mm was carried out with a hydraulic chammer. Testing of control piles was carried out with the help of hydraulic jack *DV-400-200* with its support on the stop bar. For the installation of measuring instruments (deflection indicators), two additional supports were loaded near the test pile (Figure 2).

From the sea water surface to the silt top, the depth was 15.0 m. According to DBN B.1.1-12: 2014 (maps ZSR 2004), the area refers to the seismic zone with 6 earthquake intensitie sand10% probability, with 7 earthquake intensitie sand 5 % probability and to the zone with 7 earthquake intensitie sand 1% probability (earthquake intensity of the scale MKS-64).

The control tests were carried out according to the standard procedure. Each stage of the load was supported to the conditional stabilization, not exceeding 0.1 mm for the last 60 minutes of observations. Two piles (№№. 259, 215) with the length of 28.0 m were tested. The external static load, applied to the supports, is balanced by the sum of the shearing forces on the contact from soil inner and outer sides and the surface. Before overcoming the ultimate shear strength, the tubular support is compressed elastically (s_y), remaining stationary. At the load, exceeding the limiting resistance, there is a sharp increase of the settlement as a result of its movement.

The lower ends of the tubular supports (piles) were located above the silt bottom, which allowed to «break» the mand determine the soil resistance along the outer and inner surfaces (Fig. 3). The ultimate load, balanced by the pile shear resistance (strength), buried in the silt, from outer and inner sides was 1980 kN (Fig. 4).

According to the calculations, the average value of the ultimate shear resistance was 17,3 kN/m^2 (17,3 kPa). In comparison, the studies of flooded resistance sandy loam have shown that its value is much higher than sea mud and it is 20 kPa [4].

The ultimate load, balanced by the shear strength of the support (pile) № 215,buried in the silt, from the outer and inner sides was 1760 kN (Fig. 5). According to the calculation, the ultimate shear strength average value was 15.4 kN/m^2 (15.4 kPa).

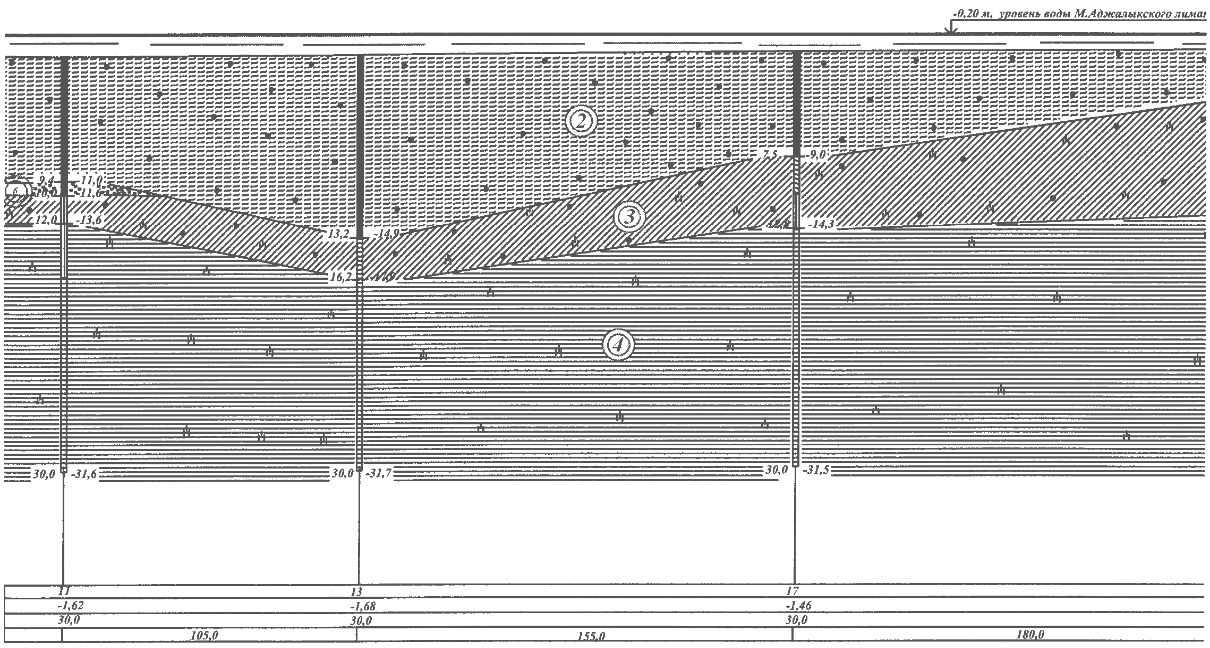

Figure1 – Engineering geological section:
EE-2 – seamud; EGE-3 – heavyloam; EGE-4 – light clay

EGE-2 – sealoamy mud is dark grey with sandy silt bands,the addition of the shells, an admixture of organic substances and hydrogen sulfideflavor; fluid.

EGE-3 – heavy loam is greenish-grey, brownish-grey, with the addition of land waste and crushed limestone; semisolid.

EGE-4 – light clay is light-grey, with ocher spots, ferruginous;semisolid.

Table 1 – Index of soilproperties

№ EGE	*Kind of soil*	ρ_s *g/cm*^{3}	ρ_d *g/cm*^{3}	*w*	I_L	S_r	*E, MPa*	φ	*c, MPa*
2	Sea loamy mud	2.45	0.01	0.56	1.552	0.97	2	4	0.012
3	Sandy loam	2.71	1.62	0.26	0.123	0.83	25	19	0.030
4	Clay	2.73	1.64	0.23	0.02	0.93	40	14	0.10

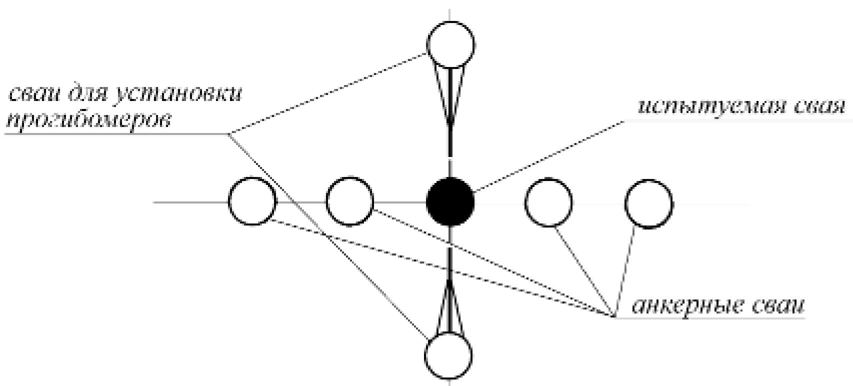

Figure 2 – Scheme of the test complex

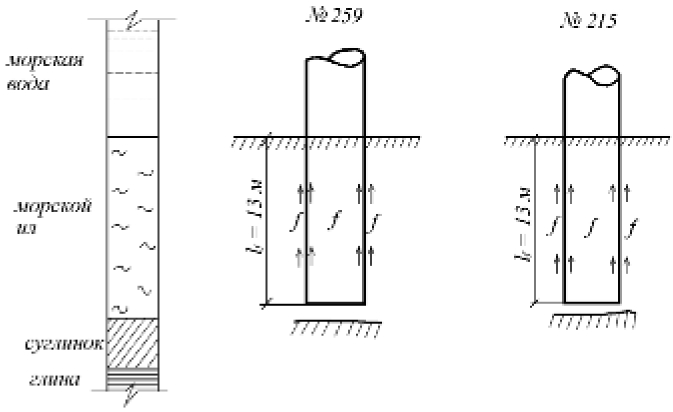

Figure 3 – The scheme of soil resistances (*f*) in tubularpiles

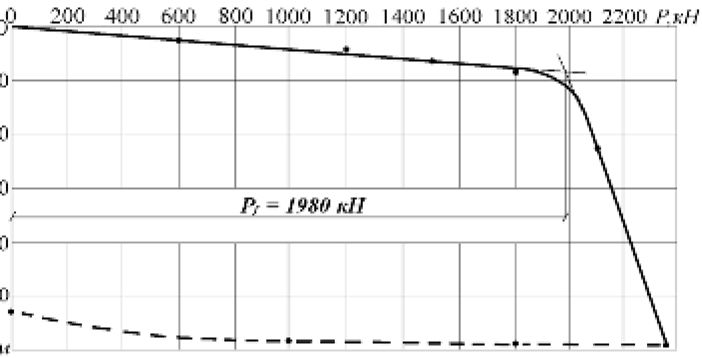

Figure 4 – Dependencygraph of settlementalong of theload for he support (pile) No. 259:
P_f is the loadwhich is equal to the ultimateshearstrength
at the contact of the support walls with the surroundingsoil

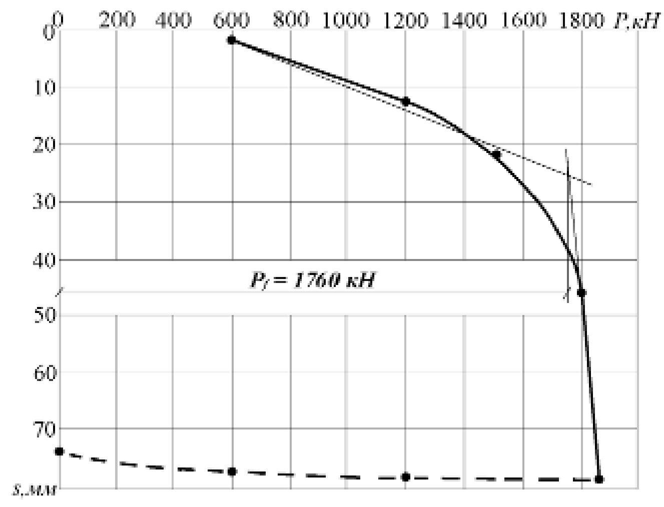

Figure 5 – Dependency graph of thesettlementalong of the support load (pile) № 215:
P_f is the loadwhich is equal to the ultimateshearstrength
at the contact of the support walls with the surroundingsoil

The test results showed that to achieve the design load on the piles, they must be extended. Pile № 254 was elongated by 6 m, which enabled its butt end to enter the soil layers lying below the silt bottom, and then its bearing capacity increased due to the resistance below the butt end (P_R).

During the penetration of a tubular pile, its walls «cut through» a layer of muddy soil. The top of the silt level from the outer and inner sides of the tubular (caisson) support ranged within ± 18 ... 20 cm. The research results of two piles, supported on muddy grounds, and the third one, which butt end buried into the underlying soils, are summarized in Table 2.

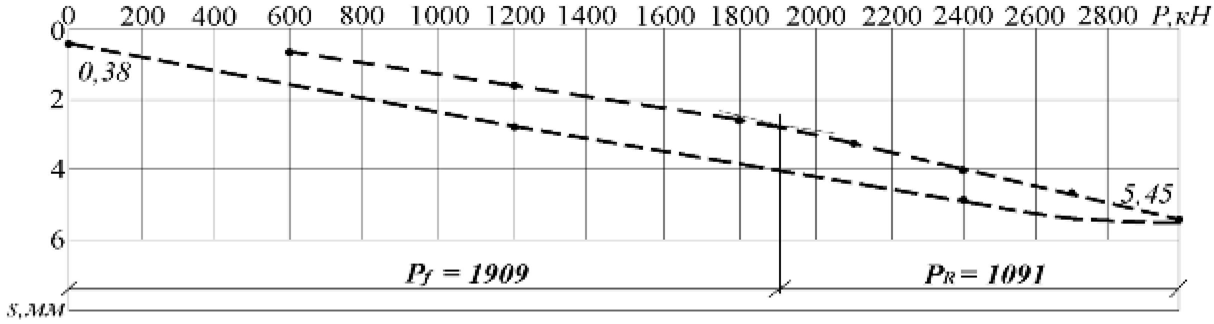

Figure 6 – Dependency graph of the settlementalong of the support load (pile) № 254:
P_f is the load which is equal to the ultimate shear strength at the contact of the support walls with the surrounding ground, the resistance below the butt end (P_R)

Table 2 – Key findings

№ Pile	*Pilelength, m*	*Load, kN*			*Settlement, mm*		*Ultimate shear strength of sea mud (atP_f), f, kPa*
		P	P_f	P_R	*Full*	*At* P_f	
259	28.0	2350	1980	370	59.87	8.94	17.3
215	28.0	1860	1760	100	79.7	25.1	15.4
254	34.0	3000	1909	1091	5.45	3.04	–

Notes: P– full load;*Pf–*ultimate soil shearstrength along the wallsurfaceof the tubular support;*PR–*soil resistanceunder the butt end of the support walls (piles).

Conclusions:

1. The butt ends of metal piles № 259 and 215 did not reach the sea silt bottom. The ultimate load, balanced by the sea mud shear strength from support outer and inner sides, was 1980 and 1760 kN.
2. Sea silt resistance along the outer and inner surfaces was17.3 and 15.4 kPa. For flooded sandy loam this value is 20 kPa.
3. The level of the sea silt top from the outer and inner sides of the pile varies within ± 18 ... 20 cm.
4. With sea silts considerable thickness and considering their resistance, it is possible to reduce partially the load effect on soil bearing layer.
5. According to the test results, all the piles were elongated that enabled their butt ends to bury the soil layers below the silt bottom.

References

1. Григорян, А.А. (1984). *Свайные фундаменты зданий и сооружений на просадочных грунтах*. Москва: Стройиздат.
2. Tugaenko, Y., Marchenko, M., Tkalich, A. & Mosicheva, I. (2015). Peculiarities of the soil deformation process at the bases of experimental settlement plates. *Technical journal Scientific professional journal of University North*, 9(1). 40-46.
3. Tugaenko, Y., Tkalich, A., Marchenko, M., & Loginova, L. (2015). Differential metod of estimation of soil resistance characteristic according to the pile test. *Scientific professional journal of University North,* 9(2). 180-185.
4. Тугаєнко, Ю.Ф. (Ред.), Марченко, М.В., Ткаліч, А.П. & Логінова, Л.О. (2018). *Природа деформування ґрунтів: монографія*. Одеса: Астропринт.

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Reinforcement of the foundation base of the building with horizontal elements of increased rigidity

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A method for reducing the uneven deformations of a two-story building, which provides improving the construction properties of weak soils that lie at the base, by reinforcing them with horizontal soil-cement elements (SCE) of increased rigidity, carried out using a sand mixing technology, is described. SCEs are created in the mass under the base of the foundations and are formed as a result of the destruction of soil natural structure with its simultaneous mixing and injection of cement mortar under pressure. It has been established that the manufacture of SCE has a number of advantages: using of local soil as a material for their manufacture; adding to the soil only water and a binder, without additional aggregate; installation of elements on the site.

Keywords: soil reinforcement, grouting element, fixing, boring and mixing technology

Армування основи фундаментів будівлі горизонтальними елементами підвищеної жорсткості

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Підтверджено, що за час експлуатації будівлі, зведені на просадочних ґрунтах, отримують нерівномірні деформації, причиною яких, як правило, є замокання з мереж водогінних комунікацій ґрунтів основи. Описано ефективний спосіб зменшення потенційних нерівномірних деформацій двоповерхового адміністративного будинку. З'ясовано, що цей спосіб передбачає поліпшення будівельних властивостей слабких ґрунтів, котрі залягають в основі будівлі, шляхом їх армування горизонтальними елементами підвищеної жорсткості, що виконуються за бурозмішувальною технологією. Ґрунтоцементні елементи (ГЦЕ) створюються в масиві безпосередньо під підошвою фундаментів й утворюються в результаті руйнування природної структури ґрунту з одночасним його перемішуванням і нагнітанням під тиском цементного розчину. Доведено, що після твердіння суміш перетворюється в армуючий елемент високої міцності та жорсткості, яка регулюється за рахунок процентного співвідношення «цемент – ґрунт». Установлено, що виготовлення ГЦЕ має ряд переваг: використання як матеріалу для їх виготовлення місцевого ґрунту; додавання в ґрунт лише води й в'яжучого без додаткового заповнювача; улаштування елементів на місці; уникнення виймання ґрунту з масиву; виключення динамічного впливу на навколишні споруди; можливість створення як вертикальних, так і похилих та горизонтальних елементів; можливість створення елементів різної довжини та діаметра, а також з різним кроком і схемою розміщення; досить низька собівартість робіт; екологічна безпека цього матеріалу. Установлено, що завдяки включенню в основу армуючих елементів відбувається поліпшення параметрів міцності та деформаційних властивостей ґрунту, підвищується опір основи до динамічних і статичних впливів.

Ключові слова: армування ґрунту, ґрунтоцементний елемент, закріплення, бурозмішувальна технологія

Introduction. Reinforcement of bases is an effective method for improving the mechanical parameters of bases, by introducing into the mass of inclusions with higher mechanical characteristics in comparison with soil [1, 2]. In geotechnics, there is a vertical, inclined and horizontal reinforcement of soil mass with cylindrical elements. They use reinforced concrete piles, sand and stone columns, soil-cement elements (SCE).

SCEs are made by mixing soil with a water-cement solution, as a result of which soil cement appears, which has higher mechanical characteristics in comparison with the natural soil.

Analysis of recent sources of research and publications. The production of SCE has several advantages [4 – 9]:

– use as a material for the manufacture of local soil SCE;

– adding to the soil only water and a binder without additional aggregate;

– installation of elements on the site;

– avoiding of excavation from the soil mass;

– exclusion of dynamic impact on surrounding buildings and structures, soil mass;

– the ability to install both vertical, inclined, and horizontal SCE;

– the ability to create elements of different lengths and diameters;

– SCE installation with different pitch and layout;

– fairly low cost of work;

– environmental safety of this material.

The sequence of SCE production depends on condition of the soil:

– dense soils are first loosened by drilling without supplying the solution, then after several approaches, the soil is mixed with the solution;

– weak soils usually do not need preliminary loosening.

This technology is called boring and mixing.

SCE are used for: strengthening foundation base; erection of pile foundations; erection of dividing walls; insulating of buried waste; strengthening the slopes of the pit; securing the slopes [3, 6, 7, 9].

Due to the inclusion of reinforcing elements in the base, the strength and deformation properties are improved, the resistance to dynamic and static effects increases, and the reduction of uneven sediment of buildings and structures is fixed. The mechanical characteristics of ground cement increase with time up to 2.5 times, respectively, it is ideal for long-term use as a base reinforcement material [5, 10].

A method of strengthening foundations with the use of horizontal reinforcing elements is becoming increasingly widespread annually during reconstruction and major repairs of buildings and structures [11, 12].

However, there is a problem with the introduction of manufacturing technology and the question of the justified use of the method of soil reinforcement with horizontal elements of increased rigidity in the course of reconstruction and major repairs of buildings and structures.

Highlighting of unsolved aspects of the problem. A significant drawback in the implementation of SCE is one-off production of equipment for their device, poor knowledge of soil-cement parameters under various geotechnical conditions, the lack of a regulatory framework for their calculation, complexity of quality control of the made elements.

Formulation of the problem. In the present work, the aim was to prove with a specific example the possibility of increasing the strength and deformation characteristics of weak soils lying at the base of the foundations to reduce potential uneven deformations of the base and foundation of the existing building.

Main material and results. The administrative building was built in the 70s of the last century and was a one-story L-shaped structure in the plan. The constructive scheme of the building is frameless with longitudinal and transverse load-bearing walls. The building was erected without a basement. Strip foundations from concrete foundation blocks and partially brickwork. The walls are made of brick, 380 mm thick, internal and 510 mm external.

The geotechnical section of the site to a depth of 12.0 m is represented by sandy and loamy soils of upper and middle Quaternary age of various genetic origin, which are covered from above by filled soils with a capacity of up to 1.9 m. The groundwater level during the survey period to the depth of 12.0 m is not opened, presumably the groundwater lies at a depth of 15.0 – 17.0 m.

The foundations of the building are based on weak natural grounds: sandy clay, humous, loess-like, subsidental, and sandy clay, loess-like, subsidental, carbonized. In places of prolonged local soaking, sandy clay acquired a flowing consistency. The total subsidence of soils from its own weight when soaked can be up to 9 cm [13].

In 2012, the reconstruction of this building was carried out, which consisted in redevelopment of premises, superstructure of the second floor, improvement of the adjacent territory. After a while, after the reconstruction in the central part of the building, deformations occurred in the form of vertical cracks in walls and partitions, deformations of the foundations and floors of the building arose. The cause of the deformations that occurred was the prolonged soaking from the water-bearing communication networks of the base soils with subsidence properties.

To reduce potential uneven deformations of the building, it is envisaged to perform the transformation of the building properties of weak soils under the entire building by reinforcing them with horizontal elements of increased rigidity. The reinforcement of the base is carried out by horizontal elements of increased rigidity (EIR) with a diameter of 300 mm. EIRs are created directly under the base of the foundations and are formed as a result of the destruction of the natural structure of the soil with its simultaneous mixing and injection under pressure of cement mortar.

Fixing the soils of the basement of the administrative building is made from two pits. Works on SCE reinforcing are performed in a certain sequence according to the drilling-mixing technology.

The reinforcing elements are located in three tiers. The distance between them is 500 mm, the pitch between the elements is 800 mm. Figure 1 shows the layout of the lower row of horizontal soil-cement elements, as well as sections 1-1 and 2-2.

Works on the SCE setting are performed in a specific sequence using a sandmill technology, the essence of which is the following.

The drilling machine with the help of a special tooling, including hollow drill rods, at the beginning of which a drilling mixer is fixed, destroys the natural structure of the soil and at the same time injects a water-cement solution that is thoroughly mixed with a crushed soil mixer. After hardening, the mixture turns into a reinforcing element of high strength – 2.5 ... 3.5 MPa (depending on the percentage of cement-soil) and rigidity (deformation modulus 80 – 110 MPa).

Reinforcing SCEs are arranged in three tiers. The entire volume of mass fixing is performed by sub-area, each of which provides for the following types and sequence of work:

– digging out pits to a designed depths with manual refining and bottom planning, arranging sump for collecting precipitation from the bottom of pits;

–preparation device from slag (capacity 100 mm);

– installation and fastening of rail tracks, connection of technological equipment;

– breakdown of the axes of reinforcing elements;

– preparation of a working water-cement solution using a mortar mixer;

– arrangement of horizontal SCE with the help of the UGB - 250A drilling machine by rotating the drill bit-mixer with a diameter of 300 mm and composite hollow rods Ø42 mm with a length of 1 m at a speed of 72 rpm and axial flow at a speed of 0.3 ... 0.75 m/min with simultaneous injection of the prepared solution through a swivel, rods and a crown using a diaphragm pump that creates pressure up to 0.5 MPa;

– removing the string of drill rods and boring bits from the well with repeated additional injection of the solution and mixing of the SCE material;

– plugging the wellhead, washing the sleeves, swivel and crowns-mixer;

– shutdown, rearrangement and connection of equipment to the next SCE.

After completion of the work on the device of the reinforcing elements of the lower tier, the pit is filled with local soil with compaction to the density of the soil in a dry state $\rho_d \geq 16.5$ kN/m^3 to a height of 500 mm and proceeds to the arrangement of the SCE of the middle tier. Similarly, go to the execution of the upper tier.

The SCE are carried out at air temperature above 50C. When the alternating air temperature, the GCE caps are insulated.

The composition of the water-cement mortar is «cement + water». Per 1 meter of soil-cement element the following are consumed:

– portland cement M400 – 25 kg;

– water – 80...100% by weight of cement (20...25 *l*.).

SCE are arranged next but one. The works on the missing element should start no earlier than in two days after the production of the nearby.

Before operation, in the process of fixing the base soils and during the SCE material strength set, the building is monitored, which includes high-precision control of changes in the spatial position of the fixed building as a whole, and individual structural elements separately, in order to determine possible uneven sediment of the foundations during work on the consolidation of the soil base.

Geodetic monitoring of the object will be carried out by a geodetic method and an automated complex using the «Monitoring» information-measuring system (once a week).

After completion of the work, the pits are filled up with a layer-by-layer compaction of the soil to the density of the soil in a dry state $\rho d \geq 16.5$ kN / m^3.

The operational control parameters for the SCE are:

– the planned binding of the axes of the wells, according to the design or adjusted parameters;

– the size of soil densification zone;

– linear speed of the mud mixer pulling back (not more than 0.3 m/min.);

– water-cement ratio in the range of 0.8 ... 1.0;

– cement activity (not lower than M400);

– cement consumption per 1m of wells (absorption);

– solution injection pressure (at optimal pressure there is no soil ejection from the well).

When arranging the first SCE, according to [14], the technology and their technical parameters are tested in each tier. In the process of tested elements manufacturing should be determined:

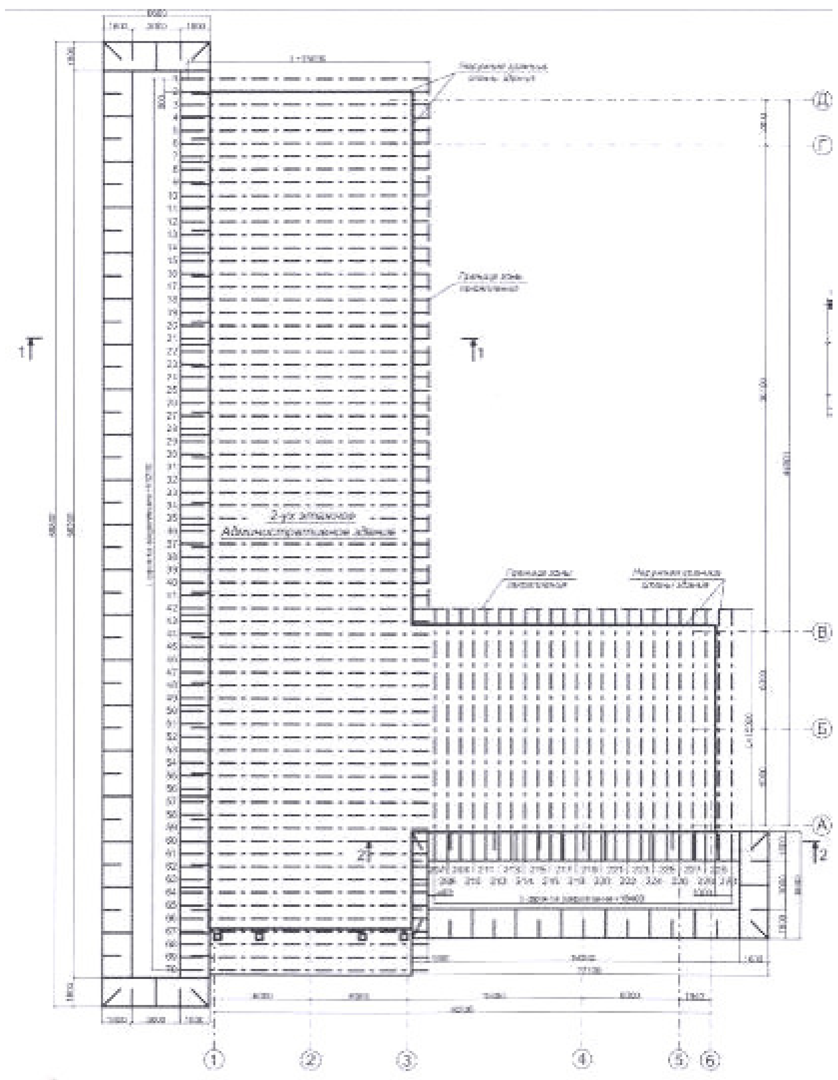
– drilling and pulling back speed;

– water-cement ratio of the solution;

– well absorption;

– discharge pressure of the solution.

To determine the quality of soil densification, samples of ground-cement reinforcing elements are sampled during their manufacture and at the age of 14 days a test is made in laboratory conditions, according to [15], and the cubic strength of the soil cement should be at least 1.5 MPa, and on the 28th day should be 2.0 MPa. The number of test items must be at least one for each sub-area.



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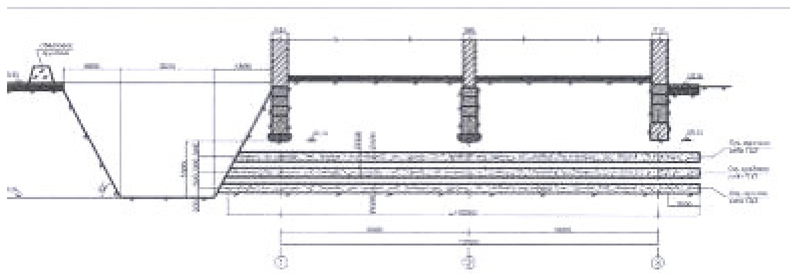

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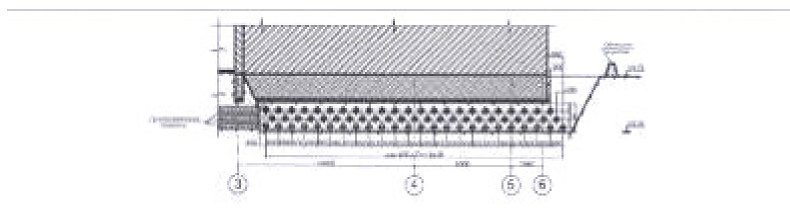

c

Figure 1 – Layout of lower row of SCE (a), sections 1-1 (b); 2-2 (c)

The control of the length and continuity of the SCE is carried out by the acoustic method. 10% of items are subject to control. After completion of the soil reinforcement of the foundation with horizontal SCE, considering that the type of geotechnical conditions of the territory and the site of the administrative building is referred to II by subsidence, all water-carrying communications should be reconstructed (replaced) and brought in line with the requirements of regulatory documents.

During exploitation of the building, it is necessary to inspect building structures, water-bearing communications and timely make preventive repairs. In the case of a leakage in the pipes of water-carrying communications it is necessary to rectify them immediately. Safe exploitation of the building can be ensured only with the prevention of ground soils watering.

Conclusions. The possibility of increasing the strength and deformation characteristics of weak soils lying at the base of the foundations of the building with the aim of reducing its potential uneven deformations by the method of reinforcing the soils with horizontal elements of increased rigidity, made by drilling and mixing technology, has been proved on the field site.

The stress-strain state of reinforced bases is influenced by the parameters of their reinforcement and the natural properties of the soil. Mechanical properties of reinforced soil mass are regulated by changing the parameters of reinforcement: step arrangement of elements; their diameter and length; layout scheme.

To reduce a base settlement, a method of their reinforcement with rigid horizontal SCEs, made by drilling-mixing technology, is promising, since the use of such elements improves the geotechnical properties of the soil, significantly reducing the settlement of the bases. Mechanical parameters of ground cement grow in time up to 2.5 times, respectively; it is highly effective for long-term use as a base reinforcement material.

References

1. Друкований, М.Ф., Матвєєв, С.В., Корчевський, Б.Б. та ін. (2006). *Армовані основи будівель та споруд*. Вінниця: УНІВЕРСУМ.

2. Тимофеева, Л.М. (1991). *Армирование грунтов. Теория и практика. Армированные основания и армогрунтовые подпорные стены*. Пермь: Пермский политехнический институт.

3. Зоценко, М.Л. (2011). Ґрунтоцементні основи та фундаменти. *Будівельні конструкції: міжвідомчий наук.-техн. зб. наук. праць (будівництво)*, 75-1, 447-456.

4. Denies, N. & Lysebetten, G.V. (2012). *Summary of the short courses of the IS-GI 2012 latest advances in deep mixing*. Proc. of the Intern. Symposium on Ground Improvement IS-GI. Brussels.

5. Zotsenko, N., Vynnykov, Yu. & Zotsenko V. (2015). Soil-cement piles by boring-mixing technology. *Energy, energy saving and rational nature use*. Oradea University Press.

6. Зоценко, Н.Л. & Тимофеева, Е.А. (2015). Шламовый амбар для отходов нефтегазовых скважин с грунтоцементным противофильтрационным экраном. *Вестник Пермского национального исследовательского политехнического университета. Строительство и архитектура,* 1, 7-20. doi:10.15593/2224-9826/2015.1.01.

7. Винников, Ю.Л. & Веденисов, А.В. (2015). Модельные исследования эффективности грунтоцементных разделительных экранов для защиты зданий от влияния нового строительства. *Вестник Пермского национального исследовательского политехнического университета. Строительство и архитектура,* 1, 51-63. doi:10.15593/2224-9826/2015.1.04.

8. Vynnykov, Yu., Voskobiinyk, O., Kharchenko, M. & Marchenko, V. (2017). *Probabilistic analysis of deformed mode of engineering constructions soil-cement grounds,* MATEC Web of Conf. Proc. of the 6th Intern. Scientific Conf. «Reliability and Durability of Railway Transport Engineering Structures and Buildings» (Transbud-2017). https://doi.org/10.1051/matecconf/201711602038.

9. Kryvosheiev, P., Farenyuk, G., Tytarenko, V., Boyko, I., Kornienko, M., Zotsenko, M., Vynnykov, Yu., Siedin, V., Shokarev, V. & Krysan, V. (2017). *Innovative projects in difficult soil conditions using artificial foundation and base, arranged without soil excavation,* Proc. of the 19th Intern. Conf. on Soil Mechanics and Geotechnical Engineering (Sep. 17 – 22, 2017 / COEX, Seoul, Korea). Retrieved from https://www.issmge.org.

10. Ezaoui, A., Tatsuoka, F., Furusawa, S., Yirao, K. & Kataoka, T. (2013). *Strength properties of densely compacted cement-mixed gravelly soil,* Proc. of the 18th Intern. Conf. on Soil Mechanics and Geotechnical Engineering (Paris, 2013).

11. Шокарев, Е.А., Шаповал, А.В., Шаповал, В.Г., & Шокарев, А.С. (2012). Опыт устранения неравномерных деформаций здания на лессовых просадочных грунтах. *Збірник наукових праць. Серія: Галузеве машинобудування, будівництво,* 4(34)-2, 291-295.

12. Самченко, Р.В., Щербина, Л.В., Степура, И.В., Шокарев, А.С., Юхименко, А.И. & Шокарев, Е.А. (2013). О проблемах реконструкции зданий и способах их решения. *Известия высших учебных заведений. Строительство,* 9(657), 115-122.

13. ДБН А.2.1-1-2014. (2014). *Інженерні вишукування для будівництва*. Київ: Мінрегіонбуд України.

14. ДСТУ-Н Б В.2.1-28:2013. (2013). *Настанова щодо проведення земляних робіт, улаштування основ та спорудження фундаментів (СНиП 3.02.01-87, MOD)*. Київ: Нацстандарт України.

15. ДСТУ Б В.2.7-239:2010. (2010). *Розчини будівельні. Методи випробувань*. – Київ: Нацстандарт України.

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Research of microelements content in the stratal waters

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The content of trace elements of iodine, bromine in the stratal waters of Chyzhivsk and Bilsk field has been analyzed. Deposits with high content of iodine and bromine ions are studied. The connection has been detected between the high content of iodine in underground water and their mineralization depending on the respected thermobaric conditions. It has been established that stratal waters of Chyzhivsk and Bilsk fields can be attributed to iodine-bromine. The analyzes has revealed that waters of the Chyzhivsk and Bilsk fields can be used for industrial removal of iodine and bromine. Having analyzed the data on the field, it has been determined that ground waters of the disclosed Chyzhivsk and Bilsk field complex cannot be used for amelioration purposes because of their high mineral content.

Keywords: trace element, iodine, bromine, stratal waters, oil basins, thermobaric conditions

Дослідження вмісту мікроелементів у пластових водах

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Відомо, що пластова вода є джерелом цінних мікроелементів, а саме йоду та брому. Підтверджено, що йод і бром – продукт єдиного процесу трансформації органічної речовини, що відбувається при високих температурах і тисках. Для визначення вмісту йоду застосовано йодометричне визначення з використанням гіпохлориту як окиснювача. Цей метод дозволяє визначити кількість іонів йоду з точністю до 0,02 мг в аналізованому об'ємі води без підготовки. Установлено, що пластові води приурочені до пісковиків глибиною від 3 – 14 до 30 м. Води представлено у вигляді розсолів. Визначено, що пластові води Чижівського та Більського родовищ можуть бути віднесені до йодо-бромного типу (бром, який складає щонайменше 25,0 мг/л, йод – принаймні 5,0 мг/л). Згідно з дослідженням вод Серпухівського горизонту Чижівського родовища, виявлено експоненціальну залежність між середнім умістом йоду та вмістом солей у пластових водах свердловин № 39, 50 Чижівського родовища. Вивчено пластові води Більського родовища. Серпухівські відкладення товщиною 200 м представлено щільними пісковиками та алевролітами. У пластових водах свердловин 104 і 105 виявлено бром, йод у кількостях, доцільних для промислового видобутку. Пластова вода має мінералізацію близько176 г л. Дослідження показали, що води Чижівського й Більського родовищ можуть бути використані для промислового вилучення йоду та брому, оскільки вміст йоду перевищує 10 мг/л, а вміст брому досягає 599 мг/л. З'ясовано, що ці пластові води є цінною сировиною для вилучення корисних мікроелементів, які зараз втрачаються.

Ключові слова: мікроелемент, йод, бром, пластові води, нафтові басейни, термобаричні умови

Introduction. It has been found that underground reservoir water can be a source of useful components. Later considerable concentrations of iodine were found in ground waters in nearly all world oil fields. However, in the oil fields waters the iodine content can vary widely. In many world countries industrial ground water is a major source of iodine. Over 70% of bromine production is provided by industrial waters [6, 10, 11]. The main features of iodine and bromine geochemistry were studied by V. Vernadsky. The scholar determined that endogenous rocks, ores and minerals iodine is only contained exceptionally in a dispersed state. The more distant from the sea area is, the higher it is located above its level, the lower the iodine concentration is in the soil, in the water and in the air. The studies were performed by P.M. Bilonizhka, V.I. Knesenko, O.M. Nikipelova [3 – 9] and others.

The study of the potential for iodine recovery from reservoir water has been started not long ago. However, to begin commercial iodine extraction from oil or gas fields waters, it was necessary to analyze the fields where iodine concentration is sufficient enough for industrial production.

The purpose of the research is to provide the content of trace elements in the stratal waters of Chyzhivsk and Bilsk deposits and analyze fields of the deposit with the content of ions iodine, bromine.

The main material of the research. Minerals containing iodine are individual or mixed halides. The sustained cycle of iodine is natural. Iodine is essential for both the biosphere and the noosphere. Its role in industry, medicine is growing every year. Iodine minerals are easily diluted; therefore iodine is easily bloomed out of rocks, carried to the sea, where it is partially accumulated in laminaria (kelp) algae [1].

Stratal water is a valuable source of minerals, bromine and iodine. Stratal water is underground water circulating in rock layers. In oilfield geology, stratal water is water present in the oil stratum (marginal, bottom, middle water). Iodine concentration in Chyzhivske gas condensate field stratal waters is studied. Most of the Poltava Region territory (northern and central) is located within the limits of Dnieper- Donets depression. It has a rather complicated geological structure. Within the Poltava Region territory it includes the southwest relatively smooth slope, separated series of fractures and the lowered central part (palaeorift or Don-Dnieper graben). The border between the south-western slope and the graben of the Dnieper-Donets depression is made along the line of Pyriatyn – Khorol – Bilyk, which roughly corresponds to the depth of the crystalline basement 1500 m. Different depths of hydrocarbons deposits, and hence different pressure, temperature and other geological conditions favored the formation of gas condensate, gas, petroleum, oil-and-gas, gas-and oil, oil-and-gas condensate fields [1].

Chyzhivske oil field is located on the territory of Gadyach and Lokhvitsa districts of Poltava Region, 15 – 20 km to the east of Gnidyntsi oil-and-gas field. In the vicinity of the field the following towns are located: Gadyach, Lokhvytsya, Romney, Glynsk, Pryluky, Bakhmach, Zinkiv, Myrgorod and the villages of Petrivka-Romenska, Krasnoznamenka, Chervonozavodske, Yaroshovka and others. Chyzhivske raise is a complication of Glinsko-Rozbyshivsky shaft, and it is located northwest of Pogarshchynske on the same axis with it, but it is more sunk. Chyzhivske raise is a crypto-diapiric structure with pre-carboniferous deep deposits of Devonian salt core. Chyzhivske raise, in tectonic terms, is located in the central part of the Dnieper – Donets shaft and is confined to the smooth anticline structures belt. To the west of Chyzhivske raise Avdiyevska structure is located, and to the east of it there is Komyshnivska structure. On the south of the structure belt, Lokhvytsko-Yarivsky and Zhadanivsky saggings are adjacent [2].

According to the schemes of detailed seismography and deep drilling, Chyzhivska structure is an anticline fold stretching northwest as to its productive T horizon’s roof. In the vault its axis is archwise bent, resulting in the north-west stretching of the structure being changed in the sub-latitudinal direction. The length of the fold on the long axis is 12 km and its width is 9.5 km. The amplitude of the elevation makes 700 m. West pericline is more extended, deepened at angles of 12 – 14°. The south wing is somewhat smoother than the north one, its inclination degree makes 10 – 11°. In the north wing a coherent faults zone is traced, consisting of two tearing faults. According to the structure schemes of the Chyzhivske raise, its Visean deposits preserve all the morphologic features. In the younger sediments ranging from the Middle Carboniferous, the uplift is gradually incurving, and according to the Mesozoic-Cainozoic sediments a sloping northwest stretching monocline is silhouetting.

The Visean layer is divided into the lower and upper sub-layers. The lower part of the sub-layer is formed of dark grey and black argillites bands of limestones, siltstones and mortars. The upper part of the sub-layer is composed of dark grey very dense limestones. The Serpukhov layer consists of the lower and upper sub-layers. The lower one consists of dense dark grey argillites. The upper one is an argillous greenish strata with bands of sandstone, limestone and coal. The Middle Carboniferous section is reaching on the rocks the Lower Carboniferous layers with stratigraphic and angular incoherence and it is presented by the Bashkir and Moscow layers. The Upper Carboniferous section is a chain of argillites and sandstones, sometimes with bands of siltstone and limestone. To characterize the water saturation of the horizons and for qualitative assessment of the aqueous rocks collectors properties, the stratal water flow discharges were calculated for some horizons using the layer recovery curves. The stratal pressure in aqueous horizons was measured by means of the depth gauges. The stratal temperatures were measured by means of electric thermometers. The water viscosity was determined with account of its temperature and salinity. In the upper part of the section, in the active water exchange zone the aqueous horizons of the Cainozoic and Cynoman Low-Cretaceous sediments are located. Aqueous are loams, anisomerous sands and sandstones with bands of argillous sands and clays. The rock filtration properties vary widely (F_f = 0.6 – 3.6 m/day, according to the experimental pumpings) and are defined by their lithologic composition, homogeneity degree and consistency in length . Aqueous horizons contain fresh water of sodium-hydrocarbonate composition with mineralization of 0.4 – 1.2 g/l, which is widely used for drinking water supply. The lower occurring aqueous strata are located in the zone of slow water exchange. The relative productive horizons of the section include the Moscow and Bashkir layers of the Middle Carboniferous, Visean layer of the Lower Carboniferous and the Turney-Devonian periods sediments confined to sandstone strata. The depth of the aqueous Middle Carboniferous horizons varies from 2 – 5 to 20 – 30 m, the porosity is 20 – 24%. The water content of the complex is high: at testing the Moscow layer deposits in well No. 12, the obtained inflow of stratal water made 214 m^3/day , with the productivity factor of 0.37. The smaller inflows of stratal water were obtained at testing the Bashkir layers, where they are 20 – 34.8 m^3/day.

The significant inflows of stratal water from the wells, where the Middle Carboniferous sediments were tested indicate that gas deposits confined to this complex are under the conditions of highly active hydrodynamic system. In the oil and gas fields a slight overpressure is traced, exceeding hydrostatic pressure by 0.5 – 1.9 MPa.

Developed under the Turney-Devonian conditions, thermodynamic water-pressure systems normally have limited contact with the gas deposits. The rocks water saturation in gas contour areas of the productive horizons is not very high, although their capacity and filtration parameters within the gas content loop are significant enough. The water obtained from this facility is a highly metamorphized, practically sulfate-free solution (SO_4^{2-} content makes 16 mg/l) of calcium-chloride type with the level of mineralization making 232 g/l. It its salt composition the abnormally high iodine (50.8 mg/l) and bromine (39.43 mg/l) concentrations are observed. The obtained data may indicate the possibility of the stratal waters ingress through the tectonic shift from the Devonian intersalt sediments, characterized by a high content of the above trace elements [2].

Also the stratal water in the Bilske fields are studied. The bilgeous gas condensate field is administratively located in Zinkivsky district of the Poltava region and Ohtira district of the Sumy region. In orygrography, the Bilske fields is in the midst of the Vorskla rivers in the east and Grun in the west.

Among the most representative in hydrogeological terms, directly in the field, are wells 105 and 104, where from the early coal deposits deposits of stratal water. Serpukhov deposits with a thickness of 200 m are represented by densely packed sandstones and siltstone with low gas saturation. Of the microcomponents are bromine, iodine and boron, the content of which is respectively 224 mg/l, 12 mg/l and 17 mg/l. Stratal water is calcium-chloride type, with mineralization – 176 g/l.

To analyze the deposits, no special studies have been conducted in the field. The actual material has been accumulated in the course of testing productive horizons of coal deposits. In the process of obtaining the water inflow the following activities were exercised: a) determination of the layer recovery curve to the static position; b) measurement of stratal pressure and temperature; c) water sampling for chemical analysis and sampling of water-soluble gas. Water sampling for chemical analysis was performed either in the mouth or at self-filling by means of depth samplers. In addition to the chemical composition of waters, the gas concentration and water-soluble gases composition were determined. In most cases, iodine in the field stratal waters is contained as a simple anion (I^-).

However, in the mineralized ground waters, iodine occurs partly in the form of free iodine (I_2). To determine iodine in the hydro-chemical practice the colorimetric method has been used for a long time, basing on the iodine ions oxidation by sodium nitrite to the final I_2 and extracting the latter with chloroform. This method gives satisfactory results only with the waters free from reducing agents (organic matter, H_2S, Fe_2 +, etc.).

The waters of Chyzhivske field contain ions of iron, therefore this method was not used (Tab. 1). The most convenient and accurate method of determining iodine and bromine is iodometric determination of iodine using hypochlorite as the oxidant [2]. This method can determine the amount of 0.02 mg I- in the analyzed volume of water without any preparation. Electrometric determination of iodine and bromine gives quite accurate results for a wide range of concentrations, but this method is time-taking and labor-consuming.

For the analysis of stratal water samples the following reagents were used: methyl orange (Fig. 1), sulfuric acid, potassium phosphate, potassium hypochlorite, sodium formate, potassium iodide, starch 1% (if a sample has changed its color to dark blue (Fig. 2), there is iodine present in it). To determine the amount of iodine, the solution was titrated. The aqueous system is located under this layer, including aqueous strata, confined to the sandstone bands with the depth from 3 – 14 to 30 m. The waters are represented as high salinity brines. The chemical composition of the water is calcium-sodium chloride. The temperature range of the system bedding is 110 – 120 °C [2].

Table 1 – Chemical composition of stratal waters fields

Field	Mean depth of the deposit location, m	Iodine content, mg/l	Bromine content, mg/l	rNa/rCl
Chyzhivske	3950 (C-5)	15.86 – 34.90	289 – 373	0.82 – 0.83
Chyzhivske	2980 (B-6)	7.2 – 17.98	250	0.77 – 0.79
Bilske	4470 (B-16) well 104	40.18	599	–
Bilske	4465 (B-16) well 105	27.49	125.21	–

Figure 1 – Adding methyl orange to the water samples to determine the content of iodine

Figure 2 – Titration of the water samples taken to determine the content of iodine

The highest concentrations of iodine and bromine are observed in the chloride- sodium waters of extra salinity (Fig. 1, 2). Iodine-bromine mineral waters tend to be located in the tearing faults zones, which serve as path ways for deep ground waters [5].

Iodine and bromine are also the products of the single process of organic matter transformation taking place at high temperatures and pressures. Then carbohydrate solutions and their accompanying deep sodium chloride solutions containing iodine and bromine move through the zones of large tectonic shifts into the higher areas of the Earth crust to the depths where lithologic structural conditions are favorable for the formation of oil, gas and the accompanying iodine-bromine waters accumulations. The latter are localized in artesian basins confined to large tectonic structures.

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The latter are localized in artesian basins confined to large tectonic structures. Thus, according to the Serpukhov horizon's waters study, the exponential correlation was revealed between the mean iodine content and the stratal waters salinity in wells No.39, 50 of Chyzhivske field according to the data obtained in 2012 – 2013 as shown in Fig. 3:

$$M = 12047e^{0.0071I}$$

where M – stratal water salinity, mgEq/l;
I – iodine content in stratal water, mg/l.

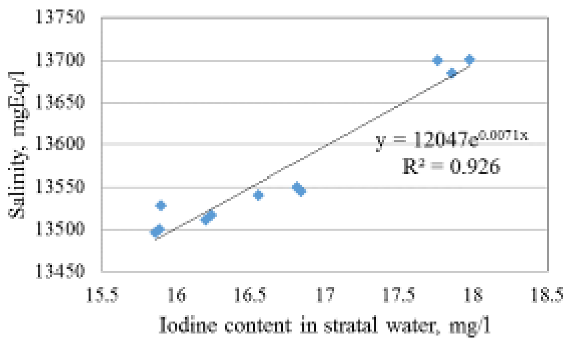

Figure 3 – Diagram of the iodine content mean values correlation with mineralization of wells No. 36, 50 of Chyzhivske field in 2012 and 2013

The correlation is constructed considering the reliability degree of R^2 approximation, which is the highest (0.92. Thus, the iodine concentration values of about 18 mg/l are peculiar for well No.50, and the values from 16 to 16.8 mg/l are specific for the stratal waters of well No.36 of Chyzhivske field. The water of well No.50 are having greater mineralization of about 13750 mgEq/l. That is, in the analyzed wells, the iodine concentration grows with the stratal waters' salinity increase. The formation of ground waters with high content of iodine was significantly affected by the powerful sedimentary strata and the respective thermobaric conditions.

It is determined, that the lower temperature limit of iodine evaporation from organic-mineral complex of sedimentary rocks and its accumulation in ground waters is 35 – 50°C. However, the most intensive processes of the iodine containing organic compounds destruction take place at temperatures above 125 – 150o C [3].

Conclusions. Stratal water is a source of valuable micro-elements, namely iodine and bromine. It is determined, that the stratal waters of Chyzhivske and Bilske fields can be attributed to the iodine-bromine type (bromine making at least 25.0 mg/l, iodine – at least 5.0 mg/l) [2]. Thus, according to the Serpukhov horizon waters study, the exponential correlation was revealed between the mean iodine content and the stratal waters salinity in wells No.39, 50 of Chyzhivske field according.

The research has shown that waters of the Chyzhivske field horizons can be used for industrial extraction of iodine and bromine, because the iodine content exceeds 10 mg/l and the bromine content reaches 599 mg/l. Due to the low temperatures on the surface and the unspent water absorption of deposits, they can not be used for heat-energy purposes. However, these stratal water is a valuable raw material for the extraction of useful micronutrients that are now lost.

References

1. Іванюта, М.М. (Ред.). (1998). *Атлас родовищ нафти і газу України*. Львів: «Центр Європи».
2. Bandurina, H. (2014). Analysis of trace elements content in the stratal waters of Chyzhivske field. *Energy, energy saving and rational nature use*, 2(3), 48-52.
3. Білоніжка, П.М. (2009). Йод у підземних водах нафтоносних басейнів як показник органічного походження нафти. *Вісник Львівського університету,* 23, 121-125. http://www.vuzlib.com.ua.
4. Будзиновская, Т.К. & Гордиенко, В.П. (1989). Тенденции развития йодобромной промышлености. *Химические технологии и инжиниринг производств неорганических соединений йода, брома и марганца,* 7, 3-5.
5. Бабкина, О.А. (1998). *Эколого-бальнеологические свойства йодо-бромных минеральных вод в восточной части Воронежской области*. Вопросы региональной экологии: тез. докл. III Регион. науч.-техн. конф. Тамбов: Тамбовский государственный университет.
6. Виноградов, А.П. (1939). Йод в морских илах. О происхождении йодо-бромных вод нефтеносных районов. *Труды биогеохимической лаборатории АН СССР,* 5, 19-32.
7. Нікіпєлова, О.М., Лемко, І.С., Драгомирецька, Н.В. & Гайсак, М.О. (2001). *Застосування бальнеогідротерапії та мінеральних вод*. Взято з www.vafk.com/ukr/l_11.doc
8. Бакиев, С.А., Калабугин, Л.А., Калабугин, А.Л., Гафуров, Т.А. & Умаров Р.Б. (2007). Изменение концентрации йода в подземных водах в связи с разработкой нефтяного месторождения Крук. *Геология и минеральные ресурсы*, 3, 54-57.
9. Кнезенко, В.И. & Стасиневич, Д.С. (1995). *Химия и технология брома, йода и их соединений*. Москва: Химия.
10. Новиков, А.Н. & Новикова, А.А. (2003). *Извлечение брома и йода из рассолов с помощью полимерных сорбентов.* Фтористо-полимерные материалы: фундаментальные, прикладные и производственные аспекты (9–11 августа 2003). Истомино.
11. Суярко, В.Г. & Безрук, К.О. (2010). *Гідрогеохімія (геохімія ґрунтових вод)*. Харків: Харківський національний університет ім. Каразіна.

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Concrete pump working capacity determination in the composition of small-sized technological set equipment for the wet method gunite work

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The small-sized technological set equipment with a universal, non-porous hose concrete pump is shown, that is verified during the execution of wet method gunite work. Dependencies for determination of basic universal hose concrete pump work parameters are proposed. The stand surface concreted with wet-cracking method, by the using of the concrete mixture composition, which has been checked during the gunite work execution on the construction site. A check of the resulting rubber-concrete coating strength has been carried out. The stability of the offered technological package operating conditions is confirmed.

Keywords: small-sized technological set equipment, gunite work, universal hose concrete pump, gun concrete.

Визначення працездатності бетононасоса у складі технологічного комплекту малогабаритного обладнання для проведення торкрет-робіт мокрим способом

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Запропоновано ряд технологічних комплектів малогабаритного обладнання для виконання торкрет-робіт мокрим способом, один з них наведено в цій статті. Показано технологічний комплект малогабаритного обладнання з універсальним безпоршневим шланговим бетононасосом, який використовується як базова машина такого комплекту. Запропоновано залежності для визначення основних параметрів роботи універсального шлангового бетононасоса: продуктивності, витрат потужності на транспортування бетонної суміші, тиску всмоктування суміші з бункера та тиску її нагнітання до торкрет-сопла. Перевірено роботу шлангового бетононасоса в складі технологічного комплекту при виконанні торкрет-робіт мокрим способом. Забетоновану способом мокрого торкретування стендову поверхню з використанням складу бетонної суміші перевірено при виконанні торкрет-робіт на будівельному майданчику. Одержане торкрет-бетонне покриття після 28 діб твердіння в природних умовах було випробувано на міцність на стиск. За результатами проведених досліджень підтверджено стабільність умов роботи технологічного комплекту, що пропонується. Установлено, що новий універсальний безпоршневий шланговий бетононасос придатний для використання його у складі технологічного комплекту малогабаритного обладнання та виконання торкрет-робіт мокрим способом.

Ключові слова: технологічний комплект малогабаритного обладнання, торкретування, універсальний шланговий бетононасос, торкрет-бетон

Formulation of the problem. In modern construction, monolithic concretization plays a central role. There are a lot of different equipment types used in these works for the concrete mixes preparation and transportation [1, 2].

Recent research analysis. The studies of these machines are represented in this works [3, 4].

Identification of previously unsettled parts of the general problem. Considered structures of machines, used in monolithic concreting, have disadvantages that relate to the prepared mixture quality and the operation reliability in comparison with the machines, offered in this work.

The experience of small-sized technological set equipment multiple use at various construction sites shows their efficiency and suggests that the wide-scale use of these machines is viable. The set content includes new machines and equipment, protected by the Ukrainian patent. This includes concrete grout pumps of different design, concrete mixers operating in cascade mode and gunned nozzles with ring tips [5, 6].

Statement of assignment and methods of its solving. It is proposed to include a universal hose concrete pump, which is developed at the Department of the Mechanization of Construction Processes of the Kharkov National University of Construction and Architecture [7], in the small-sized technological set equipment.

Study results and their discussion. Possibility to carry out a gunite work wet process using a universal hose concrete pump has been investigated in the small-sized technological set equipment (Fig. 1) [8].

Small-sized technological set equipment in accordance with the basic scheme in Fig. 1 is shown in Fig. 2.

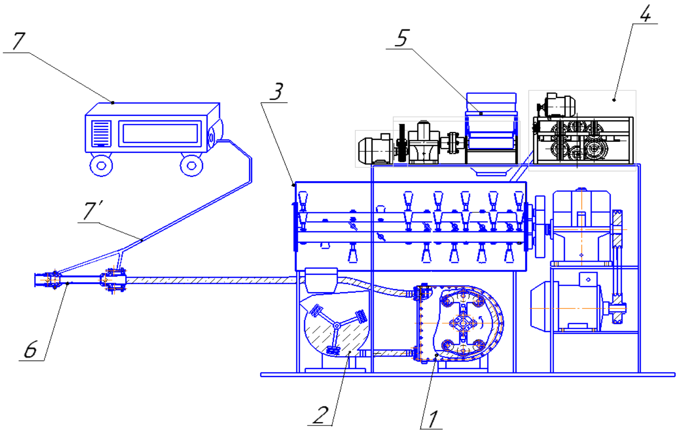

Figure 1 – Principal scheme of the small-sized technological set equipment with the use of universal hose concrete pump:
1 – universal non-piston hose concrete pump; 2 – receiving tank for concrete pump; 3 – long-time concrete mixer; 4 – automatic machine-cutter of synthetic fibers; 5 – ribbon feeder; 6 – a nozzle with a circular nozzle; 7, 7 ' – respectively compressor installation and hoses from the compressor to the nozzle

Figure 2 – Small-sized technological set equipment for carrying out a gunite work by wet method:
1 – universal non-piston hose concrete pump; 2 – receiving tank for concrete pump;
3 – long-time concrete mixer; 4 – automatic machine-cutter of synthetic fibers; 5 – ribbon feeder;
6 – a nozzle with a circular nozzle; 7' – hoses of compressed air supply from the compressor to the nozzle

The universal hose concrete pump is used as a base machine in the technological kit. To determine its main operating parameters, the dependencies are proposed::

$$\Pi_{tech} = 3600\, F_{hose}\, v_{av}\, k_1\, k_2\, k_3 \qquad (1)$$

where F_{hose} – cross-sectional area of the hose in the concrete pump housing, m^2;
v_{av} – average speed of the concrete mixture through a flexible hose, m/s;
k_1 – coefficient that takes into account the gradual buildup of the force created by the pinch rollers of the rotor when compressing the outside of the hose in the pump casing;
k_2 – coefficient that takes into account the reliability of the hose in the pump casing, taking into account the ultimate stress state;
k_3 – coefficient that takes into account the conditions for the mixture to be delivered by a concrete pump via a flexible hose, taking into account the presence of its reverse currents:

– the power consumed for the process of transporting the concrete mixture to the nozzle shaft can be determined by the dependence:

$$P_g = \frac{3.45 \cdot \left(G_{bsum} \cdot k_{tr} + F_{prut} \dfrac{k_{tr.koch}}{r_{rol}} \right) n \cdot R_{sr}}{1000 \cdot 30 \eta_{bn}} + \frac{S_{shl} \cdot \Delta p \cdot v_{sr} \cdot k_{dl}}{1000 \eta_g} \qquad (2)$$

where G_{bsum} – the weight of the concrete mixture, which is under the influence of clamping rollers, H;
k_{tr} – the coefficient of friction that occurs as a result of the wall effect between the inner wall of the flexible hose and the concrete mixture in the course of its movement;
F_{prut} – the force of pressing the rollers to the outer surface of the flexible hose in the working space of the pump, H;
$k_{tr.koch}$ – friction coefficient of rolling of rollers on the surface of the hose, м;
r_{rol} – roller radius;
n – rotor speed, min^{-1};
R_{sr} – average value of the distance between the rotor axis and the end face of the roller;
η_{bn} – coefficient of efficiency of the concrete pump;
Δp – pressure drop at the ends of the transport pipeline, Pa;
k_{dl} – coefficient taking into account the length of the transport pipeline;
η_g – hydraulic losses in the transport pipeline;

– the suction pressure of the concrete mixture from the hopper of the concrete pump is determined by the following dependence:

$$p_{vs} = \rho_0 \cdot g \cdot H_{sum} + p_{at} - (1 + \varsigma_\Sigma)\rho_0 \frac{v_{sum}^2}{2}\ , \qquad (3)$$

where H_{sum} – height of the concrete mix in the bunker, m;
p_{at} – atmospheric pressure, Pa;
ς_Σ – total loss of pressure when sucking a concrete mixture;
v_{sum} – speed of the mixture through the inlet, m / s,

– the pressure which creates a universal hose concrete pump at injection of a concrete mixture to a nozzle shaker is defined as [9]:

$$p_{nL} = \frac{4 \cdot m \cdot \omega^2 \cdot R_r \cdot \varphi \cdot f}{\pi (D_{shl} - 2\delta_{shl})^2} + \lambda \cdot \frac{L}{d_{tr}} \cdot \rho_0 \cdot \frac{v_{sr}^2}{2}\ , \qquad (4)$$

where m – mass of the rotor that overlaps the flexible hose in the pump housing;
ω – angular speed of the rotor of the concrete pump;
R_r – the radius of the rotor of the concrete pump on the end of the roller;
φ – zone of working process of injection of concrete mix, rad;
f – coefficient of friction of the concrete mixture on the inner wall of the working hose in the pump body;
D_{shl} – outer diameter of the flexible hose;
δ_{shl} – hose wall thickness;
λ – coefficient of resistance of the concrete mix along the pipeline;
L – the length of the pipeline on which the concrete mixture is transported;
d_{tr} – internal diameter of the transport pipeline;
ρ_0– average density of concrete mix;
v_{sr} – the average speed with which the concrete mixture is transported.

Using the aforementioned set of equipment, the concrete surface was wetted by means of wet gunitework.

For the vertical surface gunite, the following concrete mixture composition was used for 1 m^3: cement – 363 kg, sand – 1107 kg, rubble – 711 kg, water 183 l, plasticizer Sikaplast 520 in volume of 1.5%.

With the small-sized technological set equipment help, the concrete mixture, which was prepared in a long-time concrete mixer, was applied to the stand surface using a nozzle. In this case, the surface was concreted gradually when applying the concrete mixture to the layers. Each subsequent layer was applied after the previous hardening of the previous layer. Generally, the thickness of the layer on the cracked surface is δ = 30 ... 40mm (Fig. 3).

Figure 3 – The stand surface which was concreted by wet gunite method

After 28 days of concrete mixture deposited layer solidification in the natural environment, it was tested for strength using the concrete strength meter Beton Pro CONTROL, which is presented in Fig. 4.

Figure 4 – Beton Pro CONTROL device for measuring concrete strength

Fig. 5 shows the concrete surface, which is wetted by wet cracking, and tested for compressive strength using the device shown in Fig. 4. The strength of the gunite concrete was determined at different points of the concrete surface.

The obtained test results are listed in Table 1.

The average value of the obtained concrete strength indicators with the device help is 47.7 MPa. Spread of data in Table 1 is 15%. This testifies to the formation of a homogeneous gunite concrete structure.

Thus, the results of the conducted research showed that when working universal piston hose concrete pump to the nozzle with a ring nozzle on the hose to monounsaturated concrete mixture. This is evidenced by the lack of its bundle in the process of transportation. The stability of the operating conditions of the technological package, offered with the universal hose concrete pump, is confirmed.

Figure 5 – The process of gunite concrete strength checking using the «Beton Pro Control» device:
1 – an example of a concrete surface gunited by wet method;
2 – Beton Pro CONTROL device for measuring concrete strength

Table 1 – Pressure concrete strength to compression

Metering number	f_{com1}	f_{com2}	f_{com3}	f_{com4}	f_{com5}	f_{com6}	f_{com7}	f_{com8}	f_{com79}	$f_{com\ av}$
Strength compressine values, MPa	46.7	52.5	54.8	42.9	51.5	43.9	41.4	40.8	54.5	47.7

Conclusions

1. The efficiency of a small-sized technological set equipment with a universal, non-porous hose concrete pump is shown.
2. Operating stability conditions confirmation of the technological kit with the hose concrete pump that is proposed for the test of the gunite concrete obtained on compression: the difference of the compressive strength results does not exceed 15%.

References

1. Moran, S. (2016). *Process plant layout*. Oxford: Butterworth-Heinemann.

2. Klespitz, J. & Kovács, L. (2014). *Peristaltic pumps – a review on working and control possibilities*. IEEE 12th International Symposium on Applied Machine Intelligence and Informatics (SAMI 2014)/ Herl'any, Slovakia. http://real.mtak.hu/27727/1/37_sami2014.pdf.

3. Valigi, M.C. & Gasperini, I. (2007). Planetary vertical concrete mixers: Simulation and predicting useful life in steady states and in perturbed conditions. *Simulation Modelling Practice and Theory,* 10-15, 1211-1223.

4. Henikl, J., Kemmetmiller, W. & Kugi, A. (2016). Estimation and control of the tool center point of a mobile concrete pump. *Automation in Construction*, 61, 112-123.

5. Ємельянова, І.А., Задорожний, А.О., Гузенко, С.О. & Мєлєнцов, М.О. (2011). *Двопоршневий розчинобетононасос для будівельного майданчика: монографія*. Харків: Вид-во Тимченко А.М.

6. Ємельянова, І.А., Аніщенко, А.І., Євель, С.М., Блажко, В.В., Доброходова, О.В. & Мєлєнцов, Н.О. (2012). *Бетонозмішувачі, що працюють у каскадному режимі: монографія.* Харків: Team Publish Group.

7. Ємельянова, І.А., Задорожний, А.О., Клименко, М.В. & Чайка, Д.О. (2016). *Універсальний безпоршневий бетононасос.* Патент України 112585. Київ: Державне патентне відомство України.

8. Emeljanova, I., Blazhko, V., Shatohin, V., Chayka, D. & Kobanets, D. (2017). Features of creation of universal technological sets of the small-sized equipment for conditions of a building. *Науковий вісник будівництва (Scientific Bulletin of Civil Engineering)*, 4(90), 136-145.

9. Чайка, Д.О. (2018). Дослідження можливостей транспортування бетонної суміші універсальним шланговим бетононасосом. *Вчені записки ТНУ імені В.І. Вернадського. Серія: технічні науки,* 29(68), 25-31.

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Determination of the pressure in the cylinder of the automotive internal combustion engine by the installation of tensommetric sensors

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The analysis of well-known methods of displaying and their cost has made it possible to conclude that the methods of direct and indirect displaying have a number of serious shortcomings. Therefore, the diagnostic method for the condition of the internal combustion engine according to the value of gas pressure in the cylinders of the engine has been proposed. This method allows determining its indicator ratios, assessing the engine`s health of the ICE, the regulator (control) performance, efficiency and environmental cleanliness. This article describes the general principle of operation of the device and components of the process of measurement and processing of the obtained data. The authors of the article provided the connection diagram and substantiated the efficiency of the proposed method for obtaining information on internal cylinder processes. The results of the performed calculations indicate the suitability of the proposed calculation method for the simulation of internal cylinder processes inside internal combustion engines, including the modern ones.

Keywords: internal combustion engine, indicator diagram, diagnostics, gas pressure, cylinder-piston group, stud, stretching

Визначення тиску в циліндрі двигуна внутрішнього згоряння автомобіля шляхом установлення тензометричних датчиків

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Виконано аналіз відомих способів відображення та їх вартості, це дозволило зробити висновок, що методи прямого і непрямого відображення мають ряд серйозних недоліків. Запропоновано методику діагностування стану двигуна внутрішнього згоряння за значенням тиску газів у циліндрах двигуна. Визначено, що тиск газів у циліндрах двигуна є найбільш інформативним діагностичним параметром, що характеризує стан його поршневої частини. Обробка індикаторних діаграм дозволяє отримати відомості про якість робочих процесів досліджуваного двигуна, встановити значення його індикаторних показників, оцінити технічний стан двигуна внутрішнього згоряння, якість регулювання, економічність, шумові та вібраційні показники й екологічну чистоту. Зазначено, що реалізація методу непрямого відображення полягає в тому, що під гайку або болт кріплення головки блока циліндрів поміщають датчик тиску, який являє собою сталеву шайбу із закріпленими на ній тензорезисторами. Зусилля, що виникають від дії сил тиску газів у циліндрі двигуна, передаються через головку блока циліндрів шпилькам або болтам кріплення головки блока циліндрів до самого блока. При цьому датчик тиску сприймає ті ж самі зусилля, перетворюючи їх на електричний сигнал. Таке рішення істотно спрощує пристрій датчика і робить його конструкцію універсальною. Наведено опис принципу роботи пристрою та складових процесу вимірювання й обробки отриманих даних. Подано схему підключення та обґрунтовано працездатність запропонованого способу отримання інформації про внутрішньоциліндрові процеси на прикладі реальних вимірювань і обробки отриманих даних. З’ясовано, що результати виконаних розрахункових досліджень свідчать про придатність запропонованої методики розрахунку для моделювання внутрішньоциліндрових процесів двигунів внутрішнього згоряння, в тому числі й сучасних.

Ключові слова: двигун внутрішнього згоряння, індикаторна діаграма, діагностування, тиск газів, циліндропоршнева група, шпилька, розтягнення

Introduction. The major vehicle component part, which accounts for the largest number of failures, is the internal combustion engine (ICE). The reliability of engines operation depends not only on their design, manufacturing technology, service conditions of cars, but to a large extent on the organization and the quality of their service. The perfection of any repair technique and any method of servicing is determined by the extent to which it provides the interaction between the objectively existing process of changing the technical condition of the object and the process of its technical operation. The traditional routine-preventive method of maintenance and repair, based on the performance of certain preventive maintenance through pre-scheduled intervals of time or work, regardless of the system conditions and parts status, provides weak interaction between these processes. Closer relationship between them is provided by the methods of maintenance and repair of the condition [1, 2]. The basic principle of the methods of servicing and repairing as required is the principle of preventing the failure of the car systems and their individual most important units and parts while ensuring their maximum possible replacement life [1, 3]. The methods of servicing and repairing as required provide for continuous or periodic control and measurement of parameters that determine the technical state of functional systems and functional units, that is, the implementation of continuous or periodic diagnostics of these objects [4].

The development of the ICE design, as well as the means of measuring and processing of the received information, requires continuous improvement of the existing diagnostic methods and the development of new, more advanced methods based on the use of the latest means of obtaining information about the technical state of the engines [5, 6].

Actual scientific researches and issues analysis. The analysis of various methods of ICE diagnosis and the practical experience of their use allow us to conclude that the existing methods have a large number of deficiencies, and for correct diagnosing of the ICE's condition it is necessary to use the combination of diagnostic methods [1, 7]. This circumstance requires the improvement of the existing methods (for modern ICE at least the modernization of the existing methods is necessary) and the development of new, more advanced and informative methods for obtaining information on the technical condition of ICEs that realize the possibility to monitor diagnostic parameters.

The gas pressure in the engine cylinders is the most informative diagnostic parameter [8, 9]. The processing of indicator diagrams allows you to get information about the course of the work processes of the diagnosed engines, to determine their ratios, to evaluate the technical state of the ICE, the quality of their regulation, efficiency and environmental cleanliness [10].

Selection of previously unsettled parts of the general problem. The analysis of well-known methods of displaying has made it possible to conclude that the methods of direct displaying have high cost and do not provide the necessary accuracy of the obtained data, in addition, their application for automotive engines is practically impossible because they do not have indicator cocks. Famous methods of indirect pressure measurement in ICE cylinders also have a number of disadvantages [2, 8], as a result of which they have not received mass distribution, and most of them are used as separate experimental works with limited precision and rather narrow scope of use. Very few methods, both direct and indirect, allow continuous monitoring of pressure in ICE cylinders [9, 10] necessary to obtain operational information about changes in its condition, timely response to emerging malfunctions by performing appropriate maintenance operations, which is the most important condition for predictive maintenance.

Setting objectives. The above mentioned circumstances make it necessary to develop new methods and means of displaying engines, while focusing on the development of methods and means of indirect displaying.

The main material and the results. The proposed method of indirect display of internal combustion piston engines is based on the measurement of the voltage acting in the elements of connection of body parts (studs, bolts), connecting the cylinder caps (head of the cylinder block) and the engine block. The choice of coupling elements as a measuring object is based on the fact that they perceive only the forces of gas pressure and, thus, allow indirect displaying of the engine characteristics. The calculation scheme of coupling elements of base members is presented in Fig. 1. In the non-operating state of the ICE, the power studs are loaded with the force of pre-tightening (the minimum force strains the stud).

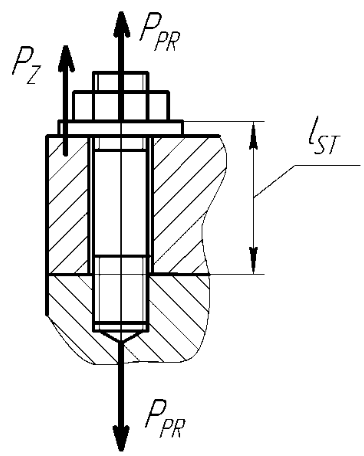

Figure 1 – Analytical model of joint members of nonrotational parts:
P_{PR} – force of preliminary tightening of hold-down studs; P_Z – the force that stretches the stud under the influence of the gas pressure; l_{ST} – the ffective length of the stud

$P_{PR} \approx m\,(1-\chi)\,P'_{Zmax}$, MN, (1)

where m – the tightening factor of the stud;

χ – coefficient of the main load of the threaded fastening;

P'_{Zmax} – force of gas pressure during combustion, which falls on one stud, MN.

Under the influence of the force of the previous tightening, the stretching of the stud and the compression of the connecting parts takes place. At the same time, the stud is lengthened by the value λ_{STmin}

$$\lambda_{ST\min} = \frac{l_{ST} \cdot P_{PR}}{E \cdot F_0}\text{, m,} \tag{2}$$

where l_{ST} – the effective length of the stud, m;

E – module of elasticity of a stud's material, MPa;

F_0 – the area of the cross-section of the stud's plug, m^2.

When the engine works, the pressure force of gases at burning causes additional stretching of the stud and head and compression of the head, the stud will be influenced by the power P_Z:

$P_Z = P_{PR} + \chi\, P'_{Zmax}$, MN, (3)

Under the influence of the force P_Z the stud is lengthened by the value λ_{STmax}:

$$\lambda_{ST\max} = \frac{l_{ST} \cdot P_Z}{E \cdot F_0}\text{, m,} \tag{4}$$

The implementation of the indirect displaying method is the following: a pressure sensor, which is a steel washer with resistive-strain sensors, fixed on it, is placed under a screw or a hold-down bolt. Efforts arising from the action of the forces of gas pressure in the engine cylinder are transmitted through the cylinder head component to the studs or bolts of the cylinder head components to the block itself. In this case, the pressure sensor takes the same efforts, turning them into an electrical signal.

This solution greatly simplifies the sensor device and makes its design a versatile one. An example of a pressure sensor is shown in Fig. 2

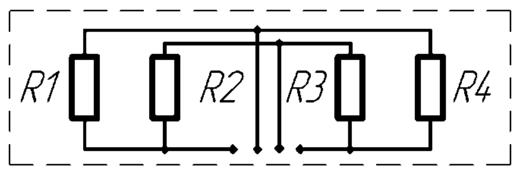

Figure 2 – The developed pressure sensor for the implementation of the indirect displaying method

There are four resistive-strain sensors, installed on the steel washer, that form a complete measuring circuit. Resistive-strain sensors R1 and R3 are set up for the perception of axial deformations; resistive-strain sensors R2 and R4 are used for thermocompensation.

Under the influence of the pressure change in the engine cylinder, the steel washer is compressed and stretched, being in the zone of elastic deformation, in this case resistive-strain sensors deform along with it, also changing their resistance. The signal from resistive-strain sensors, proportionally to the pressure in the ICE cylinder, with the help of the measuring circuit is converted into voltage, the value of which through the amplifier and the ADC is recorded by the computer, where it is further processed and analyzed. The following engines were chosen as the objects of research: engine GAZ-560; engine GAZ-5601; engine VAZ-2111.

In order to substantiate the efficiency of the proposed method of obtaining information on internal cylinder processes, the authors of the article have made the calculation of elongation of cylinder cover attachment studs (engines GAZ-560 and GAZ-5601), block bolts (VAZ-2111 engine) and compression of washes under them under the influence of forces from the previous tightening and gas pressure in the ICE cylinder. The results of the calculations show that the stretching of the cylinder cover studs (bolts) of the cylinder cap (block heads), as well as the compression of the washers underneath them while the engine is operating, are in the range available for registration by resistive-strain sensors. In particular, for the selected objects of investigation, the difference between the extension of the stud (bolt) with the maximum tensile strength and elongation at the force of the preliminary tightening is from 1 to 20 microns; the difference between the compression of the washer at maximum compressive strength and the compression with the force of the previous tightening is from 0.02 to 0.80 microns. Naturally, the most qualitative indirect indicator diagram can be achieved during the installation of resistive-strain sensors on the body of the cylinder cap mounting stud (or cylinder head bolt). However, in this case, the following problems arise: firstly, it is technologically difficult to install a resistive-strain sensor and to arrange the transmission of a signal from it, and secondly, the resistive-strain sensor is mounted on an unscrewed stud (bolt) of the cylinder cover (head of cylinder block), and in this case, the stud, together with the glued resistive-strain sensor, will be loaded with the force of pre-tightening P_{PR}. This force will stretch the stud and a resistive-strain sensor up to the value, that may exceed the permissible deformation of the resistive-strain sensor. In addition, when the engine is warmed, due to the thermal deformation of the parts, the load on the studs will increase. Thus, the main studies were related to the measurement of the stresses in the washers under the nuts of the studs of cylinder head fastener groups to the engine block (engines GAZ-560 and GAZ-5601) and in washers under the bolts of the cylinder head components (VAZ-2111 engine).

The general measuring scheme of the method of indirect displaying is presented in Fig. 3. Resistive-strain sensors 2, installed on the washer 3, compressed with a nut 1 of the stud 5 in the cylinder head fastener group 4 to the engine block 6 are connected to the measuring circuit. The signal from the measuring circuit is amplified in the amplifier 14 with a gain of 100 or 1000, and, through an analog-to-digital converter 15, is transmitted to the computer 16, where its processing is carried out.

During tests on engines GAZ-560 and GAZ-5601 simultaneously with the registration of the indirect indicator diagram a direct dispaying was carried out using the GT-20 sensor of the company Autronica and the diagnostic complex K-748. The signal from the proximity sensor 9 (see Fig. 3) was recorded in parallel with the signal from the resistive-strain sensor 2. In this case, the indicative diagrams taken with the GT-20 sensor, were considered as reference ones, and the diagrams, taken by the method of indirect displaying, were compared with them and their accuracy was estimated. The VAZ-2111 engine eliminates direct displaying, which in turn complicates the acquisition of absolute values of pressure in the cylinder, therefore the signal, registered from the resistive-strain sensor 2, proportional to the pressure in the cylinder, was considered as relative.

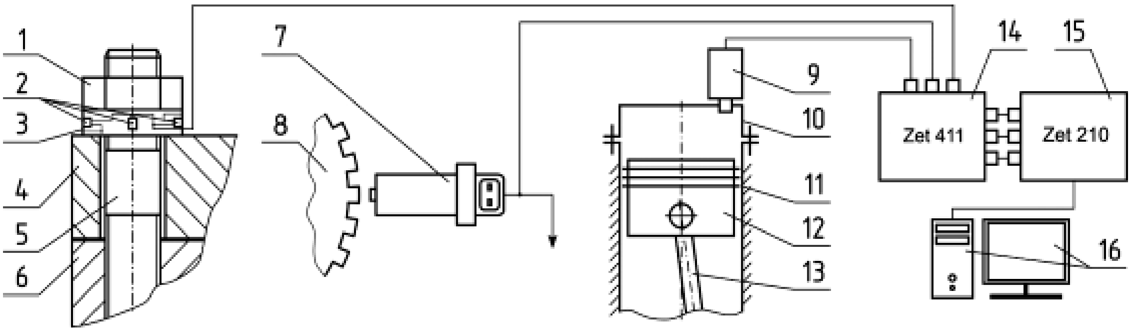

Figure 3 – The overall measurement scheme of the indirect displaying method:
1 – nut for attaching the cylinder head fastener group to the engine block; 2 – resistive-strain sensors;
3 – Washer under the nut of the the cylinder head fastener group to the engine block;
4, 10 – cylinder cover; 5 – attachment stud for cylinder head fastener group to the engine block;
6, 11 – engine block; 7 – position sensor of the crankshaft; 8 – toothed crown of the crankshaft pulley;
9 – direct displaying sensor; 12 – piston; 13 – connecting rod;
14 – signal amplifier; 15 – analog-digital converter; 16 – computer

The measurements on the engine GAZ-560 were carried out in four modes: in idle run, at the load of 14, 26 and 36 kW. The measurements on the engine GAZ-5601 were conducted in three modes: idle run, with a load of 42 and 70 kW. The measurements on the VAZ-2111 engine were conducted in idle run.

An important point in measuring the pressure in the ICE cylinders is to determine the position of the UDP in the diagram between the compression stroke and the expansion stroke. For the engines GAZ-560 and GAZ-5601 to determine the UDP at each mode of inspection, the fuel supply was switched off in the diagnostic cylinder and there was recorded compression-expansion pressure without the combustion process. According to the received diagram, the upper dead point was determined; then this diagram was superimposed on the pressure diagram with the combustion process (with the maximum coincidence of the polytrop of compression) and the moment of the UDP was determined. For the VAZ-2111 engine, in parallel with the indirect displaying, a signal from the normal inductive position sensor of the crankshaft 7 (see Fig. 3) was recorded to determine the upper dead point between the compression stroke and the expansion stroke.

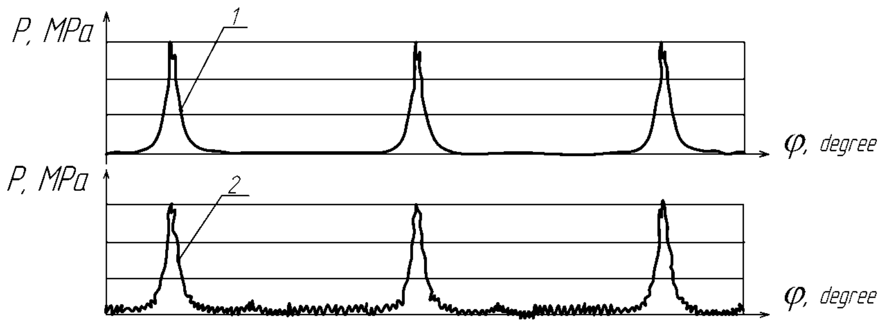

Figure 4 – Indicator diagrams of the engine GAZ-560:
1 – direct displaying; 2 – indirect displaying; three consecutive loops;
idle run; n = 630 rpm; sampling rate of 20 kHz; signal without processing

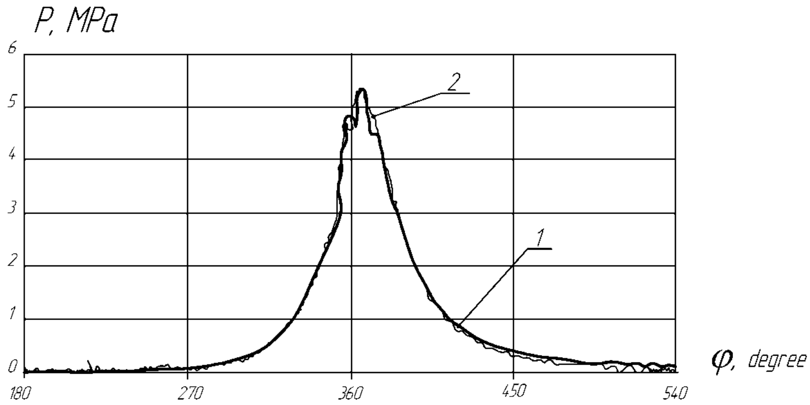

Figure 5 – Indicator diagrams of the engine GAZ-560:
1 – direct displaying; 2 – indirect displaying; load 36 kW;
n = 582 rpm; average for 24 cycles

We will consider the results of the studies, obtained by the method of indirect displaying, on the example of the engine GAZ-560. In Fig. 4 there are shown indicator motor diagrams GAZ-560, obtained by direct 1 and indirect 2 displaying on idle run (n = 630 rev / min); on Fig. 5 there are shown average 24-cycle indicator motor diagrams GAZ-560 obtained by methods of direct 1 and indirect 2 displaying, while working under the load of 36 kW (n = 582 rpm).

Conclusion. The gas pressure in the engine cylinders is the most informative diagnostic parameter that characterizes the condition of its piston part. The processing of indicator diagrams allows you to get information about the quality of the work processes of the engine under investigation, to determine the value of its indicator ratios, to assess the regulator performance, efficiency and environmental cleanliness. The analysis of well- known methods of displaying and their cost has made it possible to conclude that the methods of direct and indirect displaying have a number of serious shortcomings that do not allow them to be used in practice. And, thus, there is a need to develop new methods and means of displaying engines.

References

1. Малышев, В.С., Бабошин, А.А. & Корегин, А.Ю. (2009). Диагностирование двигателей транспортных средств с использованием методов косвенного индицирования. *Автотранспортное предприятие,* 2, 48-50.

2. Бабошин, А.А. & Малышев, В.С. (2009). Анализ методов измерения давления в цилиндрах ДВС и обоснование необходимости разработки методов косвенного индицирования. *Автотранспортное предприятие*, 9, 42-44.

3. Citron, S., O'Higgins, J., Chen, L. (1989). *Cylinder by Cylinder Engine Pressure and Pressure Torque Waveform Determination Utilizing Speed Fluctuations.* SAE Technical Paper, 890486. doi:10.4271/890486.

4. Lapuerta M., Armas, O. & Hernández, J.J. (1999). Diagnosis of DI Diesel combustion from in-cylinder pressure signal by estimation of mean thermodynamic properties of the gas. *Applied Thermal Engineering*, 19(5), 513-529. doi:10.1016/S1359-4311(98)00075-1.

5. Brown, T.S. & Neill W.S. (1992). Determination of Engine Cylinder Pressures from Crankshaft Speed Fluctuations. *Journal of Engines,* 101(3), 771-779. www.jstor.org/stable/44611250.

6. Payri, F., Olmeda, P., Guardiola, C. & Martín, J. (2011). Adaptive determination of cut-off frequencies for filtering the in-cylinder pressure in diesel engines combustion analysis. *Applied Thermal Engineering*, 31(14-15, 2869-2876. doi:10.1016/j.applthermaleng.2011.05.012.

7. Isaev, A. & Shchelkunov, A. (2008). Low frequency hydrophone calibration with using tensometric pressure sensor. *The Journal of the Acoustical Society of America*, 123(5), 1880-1882. doi:10.1121/1.2933907.

8. Бабошин, А.А. (2012). Результаты исследования метода косвенного индицирования поршневых двигателей внутреннего сгорания. *Автотранспортное предприятие,* 8, 42-46.

9. Scafati, F.T., Lavorgna, M., Mancaruso, E. & Vaglieco, B. (2018). Use of in-Cylinder Pressure and Learning Circuits for Engine Modeling and Control. *Nonlinear Systems and Circuits in Internal Combustion Engines*, 5, 55-71. doi:10.1007/978-3-319-67140-6_5.

10. Svete, A., Bajsić, I. & Kutin, J. (2018). Investigation of polytropic corrections for the piston-in-cylinder primary standard used in dynamic calibrations of pressure sensors. *Sensors and Actuators A: Physical,* 274, 262-271. doi:10.1016/j.sna.2018.03.019.

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The definition of the direction of forces arising during the interworking of a car's steer wheel with chassis dynamometer

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The article gives the theoretical substantiation of the forces determination that arise during the steerable vehicle wheel with chassis dynamometer interaction taking into account the wheels angles setting in relation to the longitudinal, vertical and transverse vehicle axes. The transition from mobile to fixed coordinate system using Euler angles is considered. The transitions comparison from the stationary coordinate system to the moving one in the system an aircraft axes and ship axes. It made it possible to move to the fixed coordinate system for a steerable vehicle wheel. A table of transition between moving and stationary reference systems has been made. The table provides an opportunity to determine the projections of forces that arise when the steerable vehicle wheel is interacting with the bearing surface when the angles of its installation are relative to the frame, are changed.

Keywords: steerable wheel, vehicle, chassis dynamometer, Euler's angle

Визначення напряму сил, що виникають при взаємодії керованого автомобільного колеса з біговими барабанами

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Проаналізовано способи перевірки кутів установлення коліс на автомобілі та виявлено недоліки способу регулювання у статичному стані величини кутів, які є однаковими для всіх автомобілів певної моделі й не враховують індивідуальні відхилення в рульовому керуванні автомобіля. Розглянуто спосіб, який дозволяє здійснювати перевірку та регулювання кутів установлення керованих коліс при рухові автомобіля й імітації такого руху за допомогою бігових барабанів. Наведено теоретичне обґрунтування визначення сил, що виникають при взаємодії керованих коліс з біговими барабанами з урахуванням кутів установлення коліс стосовно поздовжньої, вертикальної та поперечної осей автомобіля. Розглянуто перехід від рухомої системи координат до нерухомої за допомогою кутів Ейлера. Виконано порівняння переходів від нерухомої системи координат до рухомої в системі осей літака й корабельних осей, що дало можливість перейти до нерухомої системи координат для керованого колеса автомобіля. Складено таблицю переходу між рухомою та нерухомою системами відліку, яка дає можливість визначати проекції сил, що виникають при взаємодії керованого колеса автомобіля з опорною поверхнею при зміні кутів його встановлення відносно остова.

Ключові слова: кероване колесо, автомобіль, бігові барабани, кути Ейлера.

Introduction. The operation practice and scientific research show that the correct installation steerable vehicle wheels depends to a large extent on its handling, course stability, fuel consumption and tire wear [1 – 3]. Due to the tires deformation by motion, steerable wheels of a vehicle are installed at certain angles to the longitudinal, vertical and transverse car axes (longitudinal inclination angles and dislocation) [4 – 6]. These angles are assigned by the manufacturer for each individual car model and must be maintained within the specified limits throughout the service life.

Existing inspection methods include the wheels installation in a static state with angular angles, which is the same for all vehicles of this model [7, 8]. However, in each case there is a deviation from the norm. This is especially noticeable on cars that were in operation, in which there are kingpin strikes, bushings, joints steering rods, front axle beams deformation, subsidence front springs.

Typically, these deviations are compensated by the rather wide allowable limit in adjusting the wheels setting angles. In this case the optimal values are not stored, of course. In addition, when driving a car, the gaps in the steering control are selected, which leads to the change in the wheels initial position. In these cases, the recommended wheel angle setting cannot be the same for all vehicles of this model.

Actual scientific researches and issues analysis. Methods are also known which enable to check and adjust the steerable wheel of a vehicle angles when the car is driving [8] and simulating this motion by chassis dynamometer [9].

When designing chassis dynamometer stands, it is necessary to determine the forces acting on the drum from the wheel. In works [9, 10] the determination of forces is carried out taking into account only the ascent angle. Given that steerable wheels are installed at the same time at three angles, this method of determining forces is inaccurate [11].

Selection of previously unsettled parts of the general problem. Given that the steerable wheels of a vehicle are installed at certain angles relative to the car frame, there is a need to create a mathematical establish. It will allow to determine the forces projections that arise when the wheel is interacting with the support surface while taking into account the three steering wheel angles set [12, 13].

Setting objectives. The article purpose is the theoretical justification for determining the forces projections that arise when the steering wheel is interacting with a chassis dynamometer, taking into account the wheel angles mounting relative to the vehicle frame.

The main material and the results. Since the steerable wheel of a vehicle is set at certain angles to the longitudinal, transverse and vertical car axes, then to determine the projections of the forces applied to the wheel, it is necessary to make a transition from the moving coordinate system, which is tightly connected to the steering wheel and can change its position depending on changes in the wheel mounting angles and a fixed coordinate system that is rigidly tied to the support surface. This transition is carried out through a series of successive a moving coordinate system turns around the axes of a fixed system and their new positions.

There are several ways to move from a fixed to a moving coordinate system [14]. Often in such cases, the transition is carried out using the Euler's angles (Fig. 1).

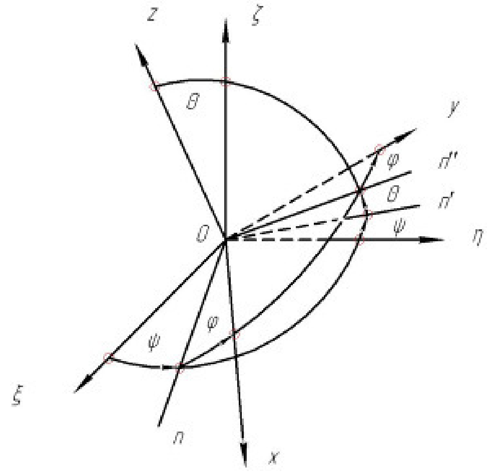

Figure 1 – The transition from a mobile to a fixed coordinate system using the Euler's angles

If, when moving from a moving coordinate system to a fixed one and taking advantage of the Euler's angles, one can notice that at small angles for the installation of driven wheels, there are small deviations axes coordinate systems moving from its original position. At the same time, the nutation angle Θ and the sum of precession angles ψ and the proper rotation φ remain small, although the angles ψ and φ themselves acquire arbitrary values within 2π. I.e. $\cos\ \Theta \approx 1$; $\cos(\psi - \varphi) \approx 1$.

In this case, the choice for the transition of Euler's angles is inconvenient, since using large angular values in determining the moving axes slight variations from the initial position is inconvenient due to the complexities that arise during mathematical transformations.

In some cases it is more convenient to use other orientation angles of the mobile reference system [14]. Thus, in the aircraft dynamics (Fig. 2), the Ox axis directed along the axis of the aircraft from the tail to the pilot cabin, the Oy axis – in the plane of symmetry of the aircraft, and the Oz axis is perpendicular to this axis (by wingspan) to the right of the pilot.

The transition from a stationary coordinate system to a moving one in the aircraft axes system is carried out by means of three turns: the first around the axis $O\eta$ on the angle ψ, the second – around the new position of the axis $O\zeta$ (line On) on the angle Θ and the third – around the new position of the axis $O\xi$ (axis Ox) at the angle φ. The angles of orientation are: ψ – the angle of rotation, Θ – is the pitch angle and φ – is the angle of the roll.

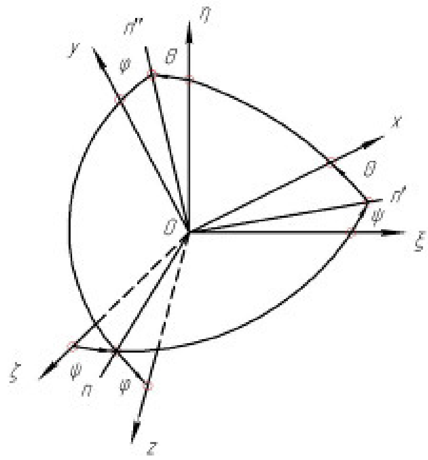

Figure 2 – The transition from mobile to fixed coordinate system of the aircraft system

Ship axles differ from the aircraft axes only by the designation. At the Krylov suggestion [14], the *Ox* axis is directed from the stern to the nose, the *Oy* axis is to the left side, and *Oz* is in the diametric plane of the ship (Fig. 3). Now the angle ψ defines the divergent, the angle Θ – the roll, the angle φ – the rigging of the ship. The transition from a stationary coordinate system to a moving axle in the ship system is carried out using three turns: the first around the axis $O\eta$ at the angle ψ, and the second one around the new position of the $O\xi$ (line *On*) on the angle Θ and the third – around the new position of the axis $O\zeta$ (*Oz* axis) at the angle φ.

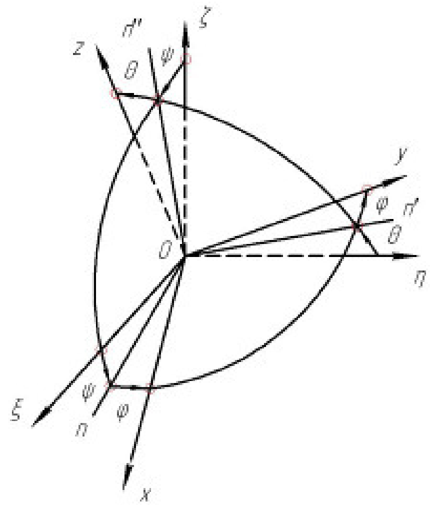

Figure 3 – The transition from mobile to fixed coordinate system of the ship system

To move from mobile to fixed coordinate system for steerable wheel of a vehicle we will use a moving coordinate system similar to the ship. The *Ox* axis is directed along the longitudinal vehicle axis, the *Oy* axis in the direction perpendicular to the vehicle axis, and the axis *Oz* vertically upwards.

Start the countdown to match the axis of rotation of the wheel. In the transition to a moving coordinate system (Fig. 4) let us consistently make another turn: the first axis *Oy* at an angle α, the second – around the new position of the axis *Ox* (line *n*) at an angle β and the third – around the new position of the axis *Oz* (axis $O\zeta$) on the angle γ. As an orientation angles there appear: β – a the angle of inclination of the longitudinal rotation axis (pivot), β – the angle of the wheels collapse and γ – angle of ascent of wheels. Thus the axis *Oy* passes into the axis $O\eta$, the axis *Ox* – in the axis *O*, and the axis *Oz* – in the axis $O\xi$.

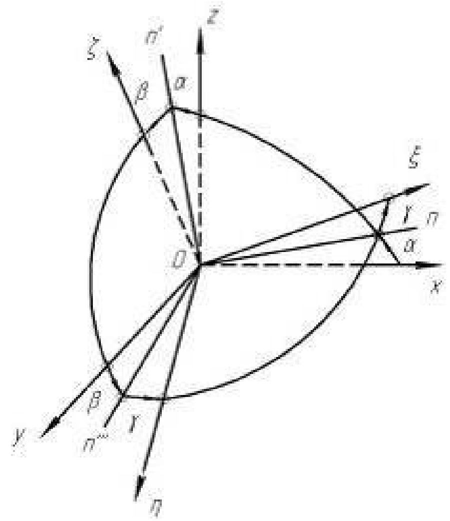

Figure 4 – The transition from mobile to fixed coordinate system for the steerable wheel of a vehicle

With such a sequence of turns, the transition from a moving coordinate system to a motionless one can be represented as Table 1.

Table 1 – Transition from moving to fixed coordinate system

Symbol	X	Y	Z
ξ	$\cos\gamma\cos\alpha - \sin\gamma\sin\beta\sin\alpha$	$-\sin\gamma\cos\beta$	$\sin\gamma\sin\beta\sin\alpha + \cos\gamma\cos\alpha$
η	$\sin\gamma\cos\alpha - \cos\gamma\sin\beta\sin\alpha$	$\cos\gamma\cos\beta$	$\sin\gamma\sin\alpha - \cos\gamma\sin\beta\cos\alpha$
ξ	$-\cos\beta\sin\alpha$	$\sin\beta$	$\cos\beta\cos\alpha$

This transition table from a moving to a fixed coordinate system allows you to determine the projections of the forces acting on the chassis dynamometer on the part steerable wheel of a vehicle, taking into account his installation angle.

Conclusion. The result of this research is the table of transition between moving and immobile reference systems. This table enables to determine the projection of forces that arise when the steerable wheel of a vehicle is interacting to the supporting surface by changing its installation angles relative to the vehicle frame.

References

1. Хачатуров, А.А. (Ред.). (1976). *Динамика системы «дорога – шина – автомобиль – водитель»*. Москва: Машиностроение.
2. Gillespie, T.D. (1997). *Fundamentals of Vehicle Dynamics.* SAE International
3. Великанов, Д.П. (Ред.). (1977). *Автомобильные транспортные средства*. Москва: Транспорт.
4. Гришкевич, А.И. (1968). *Автомобили: теория*. Минск: Выша школа.
5. Wong, J.Y. (2008). *Theory of Ground Vehicles.* NYSE: John Wiley & Sons Inc.
6. Taghavifar, H. & Mardani, A. (2017). *Off-road Vehicle Dynamics*. Springer International Publishing
7. Сирота, В.І. (2005). *Основи конструкції автомобілів.* Київ: Арістей.
8. Лудченко, О.А. (2003). *Технічне обслуговування і ремонт автомобілів*. Київ: Знання-Прес.
9. Заикин, Г.М. (1960). Стенд для проверки схождения колес. *Автомобильный транспорт,* 7, 26-27.
10. Othman, N.A. & Daniyal, H. (2015). Investigation on Chassis Dynamometer with Capability to Test Regenerative Braking Function. *International Journal of Power Electronics and Drive System (IJPEDS),* 6(3), 657-664. http://umpir.ump.edu.my/id/eprint/11368.
11. Sayers, M.W. & Han, D. (1996). A Generic Multibody Vehicle Model for Simulating Handling and Braking. *Vehicle System Dynamics,* 25(1), 599-613. doi:10.1080/00423119608969223.
12. MacAdam, C. (1988). *Development of Driver-Vehicle Steering Interaction Models for Dynamic Analysis*. University of Michigan Transportation Research Institute. https://apps.dtic.mil/dtic/tr/fulltext/u2/a208244.pdf.
13. Hong, C.W. & Shio, T.W. (1996). Fuzzy control strategy design for an autopilot on automobile chassis dynamometer test stands. *Mechatronics*, 6(5), 537-555. doi:10.1016/0957-4158(96)00010-4
14. Лурье, А.И. (1961). Аналитическая механика. Москва, Техиздат.

Збірник наукових праць. Серія: Галузеве машинобудування, будівництво
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Hydroulic stand for testing automatic seats model development

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A new design scheme for testing the car seats with hydraulic drive has been proposed, a mathematical model of stand work process has been developed, the movement of mass center in the car seat has been theoretically proved. It is determined at the expense of what factors it is necessary to develop special stands necessary for an adequate various seats check or seat configurations, which simulate long-term seats use during a short time period. The special stands car seats of different configurations work analysis simulation close to the real operation conditions on cars is carried out. The basic parameters and operating modes that characterize working processes in the conditions of operation are established. The mathematical dependence, which characterizes the work process of the stand for testing the car seat with a hydraulic drive, is obtained. The movement of car seat masses center depending on the influence factors is determined.

Keywords: model of car seat with hydraulic drive, mathematical model of the stand, coordinates of mass center movement, radius, speed

Розроблення моделі стенду для випробування автомобільних сидінь з гідроприводом

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Розглянуто різноманітні конструкції сидінь, які, як правило, перевіряються на потенційне використання в транспортному засобі з метою розробки місць, і є стійкими до деформацій та вібрацій. Визначено, за рахунок яких чинників необхідно розробити спеціальні стенди, необхідні для адекватної перевірки різноманітних сидінь або конфігурацій сидінь, які імітують багаторічне використання сидінь протягом короткого періоду часу. Проведено аналіз роботи спеціальних стендів імітації автомобільних сидінь різних конфігурацій в умовах, близьких до реальних умов експлуатації їх на автомобілях. Встановлено основні параметри та режими роботи, які характеризують робочі процеси в умовах експлуатації. Визначено необхідність забезпечення плавності регулювання процесів, підвищенні ефективності перевірки автомобільного сидіння, зменшенні рівня енергоспоживання при використанні стенду для випробування сидінь. Розроблено, спроектовано та досліджено стенд для випробування автомобільних сидінь з гідравлічним приводом робочих органів; досліджено імітацію дорожніх умов та підвищено ефективність перевірки. Запропоновано нову конструктивну схему стенду для випробування автомобільних сидінь з гідравлічним приводом; розроблено математичну модель робочого процесу стенду; теоретично доведено переміщення центру мас на автомобільному сидінні. Одержано математичну залежність, яка характеризує робочий процес стенду для випробування автомобільного сидіння з гідравлічним приводом. Визначено переміщення центру мас автомобільного сидіння залежно від факторів впливу.

Ключові слова: модель автомобільного сидіння з гідроприводом, математична модель стенду, координати руху центру мас, радіус, швидкість

Formulation of the problem. The constant increase in the requirements for the quality and comfort of the car seat characterizes the modern automotive industry state. The car seat plays an important role in the car. The quality of the car seat depends on comfort, driver fatigue while driving, and the driver's health.

Other functions are also assigned to the seat: it should provide a reliable fixing of the driver, and also enables long journeys to be carried out without fatigue.

Improving car seat quality seat is due to high requirements for car seats (DSTU, technical documentation and UNECE). Therefore, in order to ensure safety and comfort of the driver, it is necessary to check the car seat on specialized stands. Checking the car seat enables to check it for compliance with the conditions, which are specified in the technical documentation, DSTU and UNECE.

A promising direction is the development of new structures and improvements to existing stands for testing car seats. Application in such machines of a hydraulic drive provides an opportunity to ensure smoother working processes adjustment, improve the car seat checking effectiveness, reduce energy consumption.

The article is devoted to the development, design and modeling for the car seats testing stand with a hydraulic drive of working bodies. This setting enables to adjust the amount stand of motion in a wide range, the ability to simulate certain road conditions and improve the test effectiveness.

Recent research analysis. There are several designs of existing equipment for testing car seats.

Patent US 6386054 B1 "Mannequin assembly and method of testing seats which utilizes the assembly" (Mannequin and seating method using a stand).

This design uses a mannequin that can create conditions for accelerated seat length. The mannequin provides a force that simulates both the weight of the person against the attached seat part and the forces created against the seat back by the human movement [3].

In order to significantly minimize or reduce the appearance of scratches or deformations, various configurations or "seat designs" are typically checked for potential use in the vehicle in order to develop spaces that are, in fact, resistant to such deformations [3]. To adequately test these diverse seats or seat configurations, tests and stands should be designed to simulate the use of seats for a short period of time.

One device used to simulate a car seat is a mannequin or mannequin unit that has a common human form. In particular, the mannequin is stationary or passively placed inside (on the seat), which needs to be tested, and then the seat is tested for vibrational stability, since the dummy passively lies inside the seat.

Another characteristic example is the device for testing seat is patent US 6386054. The device belongs to mechanical testing products field, in particular for testing the seats of the wheeled vehicles.

The technical result is the seat back movement dependence on the attachment of effort when tested in accordance with the method described in the regulatory documents [4].

This technical result is achieved by the fact that seat load mechanism is located on a rigid basis, the seat can be installed with the vehicle floor part, which enables the most reliable way to reproduce the seat fastening mechanism. Transfer of load from load device to load cells is carried out using cables with the blocks system. The stand design enables to adjust the height and force application angle to ensure the seats widest range testing possibility [4].

The device belongs to the products mechanical testing filed, in particular for testing the wheeled vehicles seats, [4]. Its disadvantage is cumbersome design.

It is known the device US 5373749 A "Vehicle head restraint tester" The car headrest tester is taken as a prototype. At the stand, forces are applied to the selected parts of the car seat. The stand includes the transmission parts of the effort, which are fitted with the ability to rotate around the overall axis of rotation. Drives connected to the transmission parts of an effort, apply effort to any selected configuration seat back segments - simultaneously or sequentially [4].

This stand is designed to assess the suitability of seats for the Federal Motor Vehicle Safety Standard requirements № 202 and can not be used to assess compliance with UNECE Regulation requirements 80 and 14. As the requirements of these rules provide for other loading regimes and other parts are loaded. The patent does not describe how to attach seats, although the test results depend on their fixation method. In addition, the loading unit is designed for a fairly complex scheme with several worm gearboxes and hydraulic motors. To overcome the aforementioned shortcomings, the stand scheme rooted in the scientific work is proposed.

Automobile seats must meet the strength and safety requirements by the established regulatory documents, GOST, Technical Regulations, UNECE Regulations, etc. Therefore, there is a need for the development of a specialized stand to carry out tests on the seats conformity to these requirements. [4].

Identification of previously unsettled parts of the general problem. Analytical and operational studies of existing stands for testing car seats have shown that for today there is a need for the development of such equipment that have a wide capabilities range, are effective, energy-saving and able to handle high performance indicators. Therefore, a new test bench for hydraulic seats with a hydraulic drive is suggested.

Hydraulic drive use enables to have fine adjustment within wide limits, increasing the efficiency of the test process.

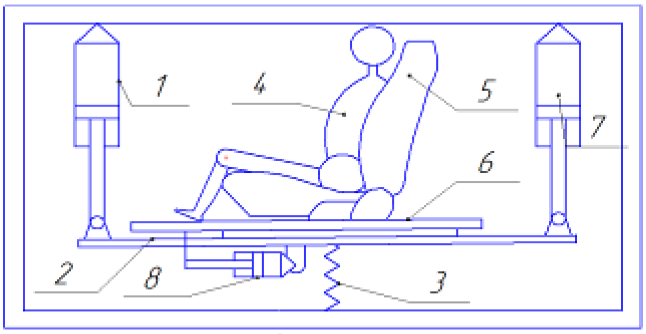

Figure 1 - Constructive scheme of the test bench for hydraulic seats with hydraulic drive
1,7, 8 - hydraulic cylinder; 3 - elastic element; 4 - mannequin; 5 - car seat; 6 - platform

Assignment statement and methods of its solving. To achieve smoothness of process control, increasing the effectiveness of the car seat check, reducing the power consumption when using the bench while testing the seats, the study is formulated as follows:

- to create, design and test a stand for testing car seats with a hydraulic drive of working bodies;
- to investigate the imitation of road conditions and improve the inspection effectiveness;
- to study the possibility of adjusting the motion amount in a wide range.

Study results and their discussion. The stand with a hydraulic drive operation principle is as follows (Figure .1). Mannequin 4, which is made in the form of a human body, is made in order to simulate human movements as much as possible while driving, securely attached to the car seat 5. Mannequin 4 is made of cast-iron or cast aluminum. Mannequin 4 itself is assembled from several parts that resemble man's bones. On the mannequin 4, sensors are installed, where test indicators are removed and transmitted to the computer. With the help of hydraulic cylinders 1,7,8 (5 hydraulic cylinders are installed on the stand), the stand can simulate different road conditions, as well as check the lower car seat part 5 for durability.

The hydraulic pump moves out the liquid from the water boiler and flows through the pipeline to the hydraulic cylinder 1. Hydraulic cylinder 1 loads platform 2, all other hydraulic cylinders remain stationary, then one side of platform 2 falls down to some angle. The car seat masses center 5 shifts towards the application of forces. The car seat 5, along with the mannequin 4, where the sensors are mounted, also leans and the sensors send data to the computer. The spring 3 resists this force, it can try to turn platform 2 into its original position. This example simulates a car hit in a pit with one wheel.

If two front hydraulic cylinders are loaded and the hydraulic cylinder 8 is loaded and the remaining hydraulic cylinders remain stationary, the front part of the platform 2 is lower evenly and the moving platform 6 moves in the rod direction. The car seat 5 along with the mannequin 4 also bends to a certain angle. Sensors that are installed on the mannequin 4 respond and send data to the computer, where it is possible to see the amount of movement and how far the masses center has shifted. The spring 3 also counteracts this force and tries to turn the platform back to its original position. This is an example of loads that mimics getting into the car front wheels pit.

Load of hydraulic cylinders 1, 7, when all other hydraulic cylinders remain motionless, incline platform 2 on the side. The car seat 5, along with the mannequin 4, also lean toward the side, the sensors mounted on the mannequin will react to a certain extent and transmit measurements to the computer. From these measurements it is fond out the number of system movements in this position and how much the car seat masses center shift. The spring 3 will attempt to turn platform 2 back to its original position. This example of loads imitates the car movement on an uneven road.

The front hydraulic cylinders sharp loading and the hydraulic cylinder 8 sharp loading, which is then gradually rotated to its original position, will cause the platform 2 to drop sharply downwards, and the moving platform 6 shifts horizontally towards the action of the hydraulic cylinder 8, after which it l returns to its original position together with the hydraulic cylinder 8. It causes the car seat 5 along with the mannequin 4 to move sharply in the hydraulic cylinders action direction. Sensors installed on the mannequin respond and record the system movement amount, and are also sent to the computer. Where it can be seen how much the center of the masses has shifted. Data loads immitate an accident of the car, which crashed from behind.

If load the front hydraulic cylinders and smoothly load the hydraulic cylinder 8, other hydraulic cylinders remain stationary. The front portion of the platform 2 is lowered downward, and the moving platform 6 moves smoothly towards the action of the hydraulic cylinder 8. The car seat 5 accepts some loads and the mannequin 4, where the sensors are installed, recording these movements. Due to the sensors installed on the mannequin 4, it can be seen the amount of the system movement and at what distance the center of the masses has shifted from the original position. This load example simulates the car braking at a certain speed.

This stand serves to check the car bottom seat for durability and compliance with the DSTU requirements, technical regulations and UNECE regulations, and also due to this stand the system movement amount and car seat masses moving the center under certain conditions can be found.

The test bench for hydraulic seats with motor can simulate any road conditions, so it can be observed how the car seat behaves in one or another situation. Thus, the proposed design provides car seat convenient and efficient inspection. The stand includes a hydraulic system that helps to more accurately capture data more accurately and has a large range of regulation. Additionally, angle measurement devices for measuring seat deflection, power adjustment devices, controllers and automatic control devices are added to the stand. This system reduces the time to test the car seat while maintaining the test results accuracy.

In stand test purpose view for motorized seats equipped with hydraulic cylinders and hydraulic drive, the following requirements should be followed when constructing a mathematical model of work. In order to achieve the stand maximum efficiency, one should strive to minimize energy consumption, and the stand must meet the requirements of DSTU, technical regulations, UNECE regulations, etc.

Proceeding from the foregoing, in order to minimize energy consumption, it is necessary to adhere to the rules, according to which at each moment of time the power stand consumption is as small as possible, but that the condition for achieving a sufficient fluid supply in the hydraulic cylinder.

The work principle is as follows. From the water tank 6, the liquid is absorbed through an unregulated pump 5 with constant flow direction and fed to a hydraulic distributor with electromagnetic control 2 (figure .2). In the neutral position of the hydraulic distributor spool, when the pump is running, the pressure on the pipeline between the pump and the distributor begins to increase, with the safety valve 4 running and the liquid merges back into the tank. When changing the position of the spool open passage sections in the distributor and the liquid begins to enter the hydraulic cylinder 1 piston cavity. From the hydraulic cylinder rod cavity fluid through the drainage line passes through a hydraulic distributor, regulated chokes with a servo 3 and a filter 7, falls into the drains in the tank.

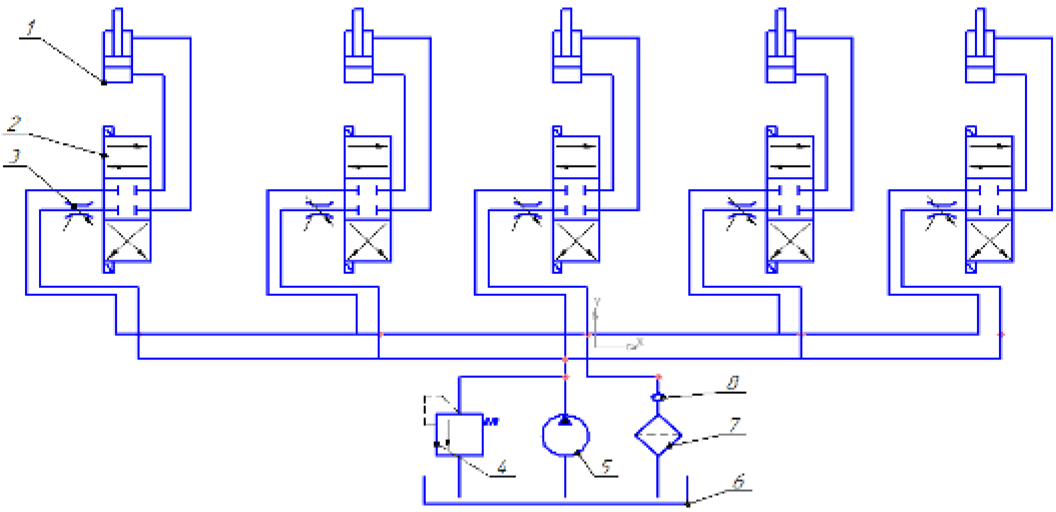

Figure 2 - Hydraulic diagram of the test bench for car seats
1 – hydraulic cylinder; 2 – hydraulic distributor; 3 – servo-choke; 4 – safety valve;
5 – hydraulic pump; 6 – a water jacket; 7 – filter; 8 – check valve

The translational movement velocity of the hydraulic cylinders rods is controlled by the chokes. Rods reversal is carried out by switching the positions of the hydraulic distributor. In stem emergency stop (for example, an insurmountable effort) the pressure in the system increases, thereby causing the opening of the safety valve and the discharge of the working fluid into the tank.

Due to modern capabilities and the complex electronic control systems development, the implementation of such a system in the hydraulic cylinders managing process can ensure the system reliability (long inter-repair period, control and shutdown during excessive pressure on the system, interconnection with the onboard computer, etc.), profitability (ensuring the equipment operation within the rational parameters limits and operating modes), ergonomics (ease of use) and safety techniques (many types of hearting during system operation occurrence prevention).

To achieve such a requirement, it is necessary to determine the energy source and all consumers in the system, there nature, principle and regularities. The next step should be to determine the total amount of energy consumed at different system operating modes and to obtain patterns according to which to adjust the amount of energy generated by the source according to the required value at each time point.

Checking the car seat on the stand using a hydraulic drive is a complex process that should be analyzed as the operation of the hydraulic cylinder and the hydraulic motor. Also, the amount of energy used to load the car seat is considered.

In this booth (Figure 3), the spring acts as a leveling device so that the stand returns to the starting position.

Since the forces acting on the stand force the center of mass to shift, then there is a need to calculate the point of mass center moving.

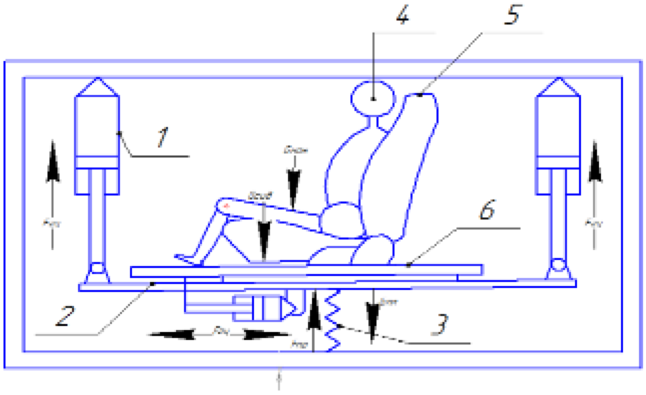

Figure 3 – Scheme of forces acting on the test bench for testing a car seat with a hydraulic drive
1 – hydraulic cylinder; 2 – platform; 3 – a spring; 4 – a mannequin with sensors;
5 – car seat; 6 – mobile platform

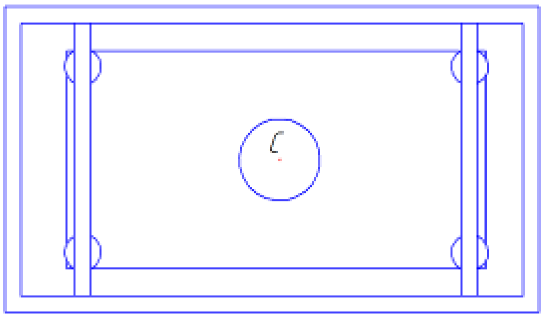

Figure 4 - Test stand mass center scheme for hydraulic seats with hydraulic drive

To determine the stand mass center displacement, the system movement amount should be known. The system motion amount is called the geometric sum of all system material points motion

$$\overline{K} = \sum m \cdot \vartheta \quad (1)$$

where $\overline{K}$ – amount of system movement; according to it motion amount projections on the coordinate axis are obtained.

$$\begin{cases} \overline{K}_x = \sum m \cdot \vartheta_x = \sum m \cdot \frac{dx}{dt} = \sum mx \\ \overline{K}_y = \sum m \cdot \vartheta_y = \sum m \cdot \frac{dy}{dt} = \sum my \\ \overline{K}_z = \sum m \cdot \vartheta_z = \sum m \cdot \frac{dz}{dt} = \sum mz \end{cases} \quad (2)$$

The theorem on the system movements amount is formed as follows. Vector derivative from the system motion amount is equal to the main vector of all external forces applied to this system

$$\frac{d\overline{K}}{dt} = \overline{R}^{(b)} = \sum \overline{F}^{(b)} \quad (3)$$

The system mass center is called the geometric point C, whose coordinates are determined by the formula:

$$X_c = \frac{\sum mx}{M} \quad Y_c = \frac{\sum my}{M} \quad Z_c = \frac{\sum mz}{M} \quad (4)$$

where M – the system mass, while x, y, z – the material points coordinates of this system.

Assuming that in the center of masses is the system entire mass, then the system motion amount is equal to the mass center motion amount. Thus, iy is got

$$K = M \cdot \vartheta_c \quad (5)$$

Here:

$$\begin{cases} K_x = M \cdot \vartheta_{cx} = M \cdot x \\ K_y = M \cdot \vartheta_{cy} = M \cdot y \\ K_z = M \cdot \vartheta_{cz} = M \cdot z \end{cases} \quad (6)$$

If the mass center acceleration C is equal to F, then it is got

$$M \cdot \overline{\omega}_c = \overline{R}^{(b)} = \sum \overline{F}^{(b)} \quad (7)$$

Proceeding from this, the final formula according to which this system is calculated, has the form:

$$K_x = \frac{P}{q} \cdot V_c \cdot \cos\varphi \quad (8)$$

where

$$\frac{P}{q} = m \quad (9)$$

$$V_c = r \cdot \omega \quad (10)$$

where r – is the circle radius; ω – mass center angular velocity.

$$\omega = \cos\varphi \cdot t \quad (11)$$

where t – time for which system will change its position. We accept $t = 30$ s.

Hence, the final formula has the form

$$K_x = m \cdot r \cdot \omega \cdot \cos \omega t \quad (12)$$

Based on Figure 5, to find out the masses center movement an equation should be made

$$\begin{cases} X_l + Y_l = L^2 \\ Y_l = tg\varphi \cdot X_l \\ X_l \lhd 0 \end{cases} \quad (13)$$

where L – length to mass center; $tg\varphi$ – tangent of the mass movement center; X_l, Y_l – displacement coordinates.

Then, if this system is solved, it is got:

$$X_l + tg\varphi \cdot X_l^2 = L^2 \quad (14)$$

Proceeding from this equation, it is obtained

$$X_l + (1 + tg\varphi) = L^2$$

$$X_l = \sqrt{\frac{L^2}{1+tg\varphi}} = -L\sqrt{\frac{1}{1+tg\varphi'}} \quad (15)$$

$$Y_l = -tg\varphi \cdot L\sqrt{\frac{1}{1+tg\varphi'}} \quad (16)$$

Having solved these equations, coordinates are got. Use the formula to find out the displacement

$$S = \sqrt{(X_l - X_a)^2 + (Y_l - Y_a)^2} \quad (17)$$

where S – mass center displacement in the coordinate plane.

It is the final formula, according to which the masses center displacement is calculated.

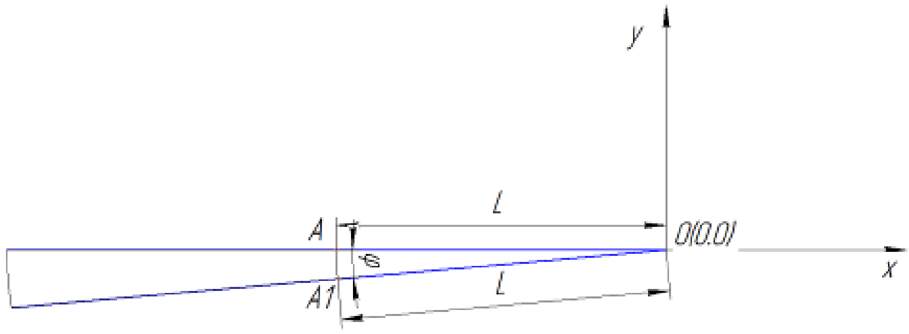

Figure 5 - Mass center moving scheme

Table 1 - Parameters that affect the movement of the mass center

№	Angle of platform φ^0	Radius R мм	Movement V	Amount of movement S, мм	V_1	S_1	V_2	S_2
0	0	75	75	0	75	0	75	0
1	1	76	75,99	3	76.99	4	77.99	6
2	2	77	76,95	5	78.09	7	80.95	10
3	3	78	77,89	7	80.89	10	83.89	13
4	4	79	78,808	8	82.8	12	86.79	16
5	5	80	79,7	9	84.67	14	89.64	19
6	6	81	80.55	10	86.52	16	92.49	22

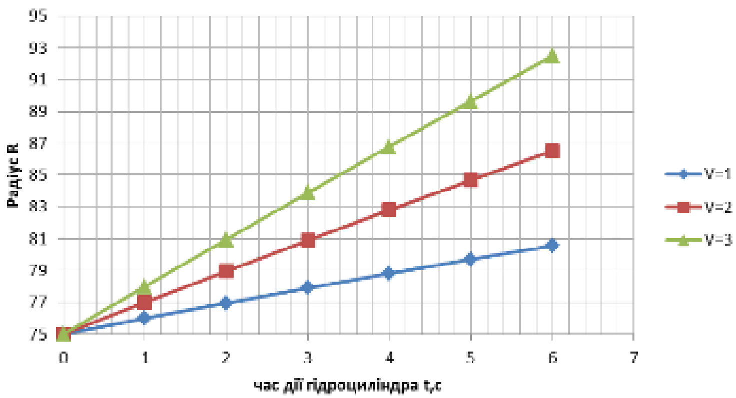

Figure 7 - Mass center moving graphical dependence of radius from speed

On this dependencies graph, the increase in the system movements amount depending on the circle trajectory under the linearly directed forces influence is clearly seen. At the same time, the masses center shifts along the whale trajectory. Characteristic is the nonlinearity, which is due to the quadratic effect of the circle trajectory.

The first curve (Figure 7) shows the center displacement of mass at speed V = 1 cm / s for a certain time period. In this curve there is observed a mass center slight displacement, because the hydraulic cylinder rod speed is small and the mass center point of moves to a small distance (Table 1).

The second curve (Figure 7) shows the mass center displacement at a speed V = 2 cm / s for a certain time period. In this case, the mass center displacement is greater than in the first case. Because at a speed of 2 cm / s, the radius that passes through the point over the same period of time is also increased (Table 1).

The third curve (Figure 7) shows the mass center displacement at a speed V = 3cm / s for a certain time period. On this curve, the mass center displacement is even greater than the second (Table 1).

Conclusions. The mathematical dependence, which characterizes the work process of the stand for car seat with a hydraulic drive test, is obtained. The car seat masses center movement depending on the influence factors is determined. According to these data it can be concluded that the smaller the seat masses center displacement itself, the less negative impact is on the person. It also determines the impact of impulses on both human health and the car seat itself. Therefore, it is very important that the seat complies with the technical documentation, DSTU and UNECE. Indeed, from the car seat depends on the driver health and the transport efficiency.

Therefore, the presented stand use gives the opportunity to test the car seat, check it on the mass center displacement and control the pulse values, so that the seat design meets all the parameters.

References

1. Бойко, Ю., Сухенко, Ю., Дубинець, О. & Сухенко, В. (2011). *Технологія автомобілебудування*. Київ: Університет Україна.

2. Солнцев, А.Н., Попов, А.И., Иванов, А.М., Гаевский, В.В., Осипов, В.И. & Клюкин, П.Н. (2012). *Основные конструкции современного автомобиля*. Москва: Машиностроение.

3. *Салон автомобіля* [Електронний ресурс]. Взято з https://ru.wikipedia.org

4. *Стенд випробування підголівника* [Електронний ресурс]. Взято з http://www.google.ch/patents/US5373749

5. Башта, Т.М., Руднев, С.С. & Некрасов, Б.Б. (1982). *Гидравлика, гидромашины и гидроприводы*. Москва: Машиностроение.

6. Светлов, М.В. & Светлова, И.А. (2015). *Техническое обслуживание и ремонт автомобильного транспорта.* Москва: КНОРУС.

7. Гришкевич, А.И. (1986). *Автомобили. Теория*. Москва: Машиностроение.

8. Павлище, В.Г. (1993). *Основи конструювання та розрахунок деталей машин*. Київ: Вища школа.

9. Башта, Т.М. (1972). *Гидропривод и гидропневмоавтоматика*. Москва: Машиностроение.

10. Башта, Т.М. (1971). *Машиностроительная гидравлика.* Москва: Машиностроение.

11. *Складальний манекен та спосіб випробування сидіння* [Електронний ресурс]. Взято з https://www.google.ch/patents/US6386054

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Streamlining influence on the long-haul trucks with an installed movable roof fairing performance properties teoretical studies

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Influence streamlined elements located on the roof of the tractor main train such as the efficiency of the vehicle with installed no bonnet on it an advanced roof rack, which has the ability to adjust to any trailed warehouse. It has been assumed that it provides high levels of accuracy with respect to analogues due to the possibility of choosing the appropriate geometric parameters, provides ease of use and the ability to adjust the cushion during vehicle movement, prevents probable malfunctions during strong winds or hurricanes due to the strength of the drive management and modern electronic systems mechanical design. On the basis of theoretical studies of the main motor trains various tractors flow, the dependences of power and fuel consumption on the air traffic vehicle coefficient have been established at its movement speeds. It has been proven that it is advisable to use an advanced racing hub on precision drum trucks with small and medium-height cabs.

Key words: main motorway, streamlined, aerodynamic properties, roof fenders, streamlining coefficient.

Теоретичне дослідження впливу обтічності на експлуатаційні властивості магістральних автопоїздів із встановленим рухомим даховим обтічником

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Досліджено вплив обтічного елементу, розташованого на даху тягача магістрального автопоїзда, а саме ефективність роботи безкапотного автомобіля з встановленим на нього удосконаленим даховим обтічником, який має можливість налаштування під будь-який причіпний склад. Було припущено, що відносно аналогів він забезпечує високі показники обтічності за рахунок можливості вибору відповідних геометричних параметрів, забезпечує зручність у використанні та можливість налаштування обтічника під час руху автомобіля, запобігає можливості ймовірних несправностей під час сильних поривів вітру чи буревіїв за рахунок міцності конструкції механічної частини приводу керування та сучасних електронних систем. На основі проведених теоретичних досліджень обтікання різних тягачів магістральних автопоїздів були встановлені залежності потужності та витрати палива від коефіцієнту обтічності автомобіля при прийнятих швидкостях його руху. Було доведено, що удосконалений обтічник доцільно використовувати саме на безкапотних тягачах із малою та середньою висотою кабіни. Теоретичні дослідження проводилися за допомогою прикладного програмного забезпечення Microsoft Exel та SolidWorks за допомогою електронних обчислювальних машин.

Ключові слова: магістральний автопоїзд, обтічність, аеродинамічні властивості, даховий обтічник, коефіцієнт обтічності

Introduction. The truck market is full of proposals for the purchase of tractors - both new and with mileage. In order to choose among all possible options, the buyer needs to evaluate each of them by the key technical, functional and operational qualities, not forgetting about saving.

One of the main factors when choosing a tractor is fuel consumption. In the conditions of operation, the road quality, the vehicle load and the movement speed have a great influence. Therefore, the automobile engine has to work on different load and speed modes.

Aerodynamic experts argue that half the fuel, or perhaps a large part of it, which burns a car at high speed, is spent on overcoming air resistance. Therefore, if it is possible to reduce the dynamic resistance, then the cost of goods delivery can be significantly reduced. Reducing the dynamic air resistance is the shortest way to generate additional profits [1, 2, 3].

Analysis of recent sources of research and publications. Lorries and trains are among the poorly streamlined vehicles. At the same time, if the low traffic velocity of a truck due to its low speed has relatively little effect on its technical and operational performance, then in relation to high-speed main tracks, their influence becomes decisive in the struggle for fuel economy, safety, dynamism, ergonomics and environmental friendliness [4 – 6].

The nature and level of traffic jams of the main train is determined by its shape, structural features and parameters of the air environment. The rectangular shape of the transverse and longitudinal section of modern highway trains bodies, in combination with flat walls, provides the most useful space for placing the cargo in them, but is unsatisfactory fconsidering aerodynamics. At the same time, in the case of on-board trucks, the main component of their frontal projection is cab frontal area, then in the main road trains with high bodies, approximately the same size area above the body front wall cabin is added [4].

As the constructive analysis shows, the typical for highway trains is the presence of a significant one, reaching 1 m or more, exceeding the body over the cabin, a large (1-2 m) clearance between them (between the bodies), in combination with an uncircumcised or rounded small radius the front edge of the cabin and the body [5]. In addition, there is a considerable distance from the front bumper to the road surface, which, depending on the type and degree of the automobile train loading, ranges from 0.5 to 0.7 m. The influence of the above factors significantly reduces the trains air travel level, as there are large areas of high and low pressure, and because of the boundary layer breakdown on the cabin and the body front edges there are energy-intensive tearing currents that have a pronounced vortex structure. As a result, the trains aerodynamic characteristics significantly deteriorate, its movement resistance is substantially increased, and stability and handling indicators are reduced.

In [4] in detail the complex mechanism of highway trains flow of different layout scheme both oncoming and lateral wind. It negatively affects the trains aerodynamic characteristics. Due to increased pressure resistance and tearing currents, aerodynamic resistance increases and the lifting force acting on the carriageway, which adversely affects trains aerodynamic stability and handling, worsens its course stability. In this case, the aerodynamic resistance intensively increases with an increase in the angle of incidence.

Selection of previously unsettled parts of the general problem. Most authors [7, 8, 9, 10] who are considering improving the aerodynamic properties of road trains, offer theoretical research and modeling with the help of 3D modeling software simply by installing individual rails in different places, both the tractor cab and the trailer link. It is good when designing new equipment, but for the already existing one the question is not solved, although there are enough domestic and foreign production cars used on roads that have not exhausted their resources, but need further modernization for their own competitiveness in the main cargo transportation services market.

Among all the existing inventions, the simplest and most effective remains is the usual fenders, located on the roof and tractor side parts. Its effectiveness is scientifically proven, but the application of one and the same ramp to different semitrailers, trailers, tanks causes a violation of the tractor aerodynamics. Such use is inappropriate and ineffective.

Setting objectives. The article purpose is to conduct a theoretical study of the main trailer aerodynamic properties with the installation of a roof railing on it with the possibility of adjusting to any trailer structure using the 3D modeling software.

Basic material and results. As described in a number of papers [4, 5], the air resistance P_{wis} determined by the friction of the air layers adjacent to the car surface; compressing air with a moving machine; dilution by car; swirling around the car air layers.

When driving the car, the air is located in front, compressed and pushed to the place where the pressure is less, that is, up, down and in the sides. By car there is a relative dilution. This low pressure area is filled with air that goes away. Since the movement of air mass during car movement is associated with a change in air flow direction, there is turbulence formation.

The larger the cross-sectional area, that is, the machine projection area on a plane perpendicular to the longitudinal axis, the greater the amount of air forced to bypass the car. The car largest cross-sectional area is called the front axle. The constituent strength of the air resistance, depending on this area, is called the frontal resistance. It is the main component of air resistance total strength, its share reaches 60% of the total. This resistance is also called resistance form, because its value depends on the body shape. Different shapes bodies investigation[6] on the frontal resistance showed such a dependence on the form, that is, on the edges.

Other components of the air resistance forces:

- internal resistance, created by air flows passing inside the car for body ventilation and heating, for cooling the engine. The share of this component is approximately 10%;
- resistance to surface friction (10%);
- induced resistance (5%) - is caused by the interaction of forces acting in the direction of car longitudinal axis (lifting) and perpendicular to it (lateral);
- additional resistance (15%), created by different protruding parts (headlights, indicators of turning, door handles, license plates).

Given the influence of air resistance all components, this force is determined by the dependence

$$P_w = k_в\, S\, V^2 \tag{1}$$

where $k_в$ – the air resistance coefficient, which considers factors that are not dependent on car shape.

$$k_в = 0.5\, \beta\, C_x\, \rho \tag{2}$$

where β – coefficient which considers additional supports.

The approximate values of air resistance coefficients for different cars are as, $Нс^2м^{-4}$, follows:

sports – 0.13 ... 0.15; cars – 0.15 ... 0.35;
buses – 0.25 ... 0.40; trucks – 0.50 ... 0.70;
trains – 0.55 ... 0.95.

The overall height of the sidecar is determined by the trailer couplings height. Their main resistance is created by air, which flows through the cabin and runs into the front wall of the semitrailer. In addition, in the intervals between the automobile train links powerful vortices are formed, which seem to increase the frontal area. Therefore, in various ways to reduce the aerodynamic resistance, rails various designs [6] are used reducing the air resistance.

Area of the front support

$$S = \alpha\, B_h\, H_h \tag{3}$$

where α – the factor of filling the area (for cars 0.78-0.8, for freight 0.75-0.9, with more value for heavier cars);

$B_h\, H_h$ – the overall width and height respectively, m.

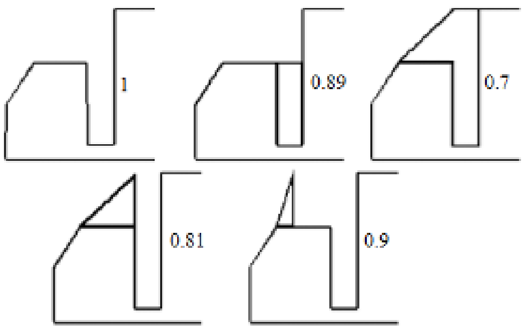

Figure 1 – Ways to improve the vibration of the car by installing the roof rails

It is calculated the roof rails modes and parameters with the ability to adjust for any trailer.

The speed of the car on the horizontal section of the road is determined, as V with its maximum power is equal P. The coefficient of car frontal resistance, and the largest area of intersection in the direction perpendicular to the speed S. The car undergone a reconstruction, the largest area of intersection in the direction, perpendicular to the speed did not change, but the coefficient of frontal resistance decreased to magnitude C'. It is considered the friction force on the surface of the road unchanged, the air density ρ.

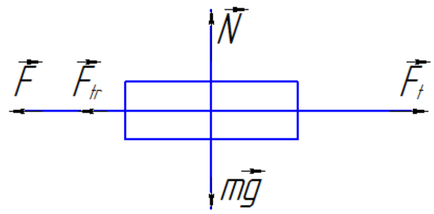

Figure 2 – Conditional application of force to the vehicle

Power car Expressions

$$P = F_t\, V \tag{4}$$

where F_t – thrust car N.

If the vehicle set on a road flat stretch is set at a constant speed is moved, then according to Newton's second law it is

$$\overline{F_t} + m\overline{g} + \overline{N} + \overline{F_{tr}} + \overline{F} = 0 \tag{5}$$

Formed on a projection the axis of OH it is obtained

$$F_t - F_{tr} - F = 0 \rightarrow F_t = F_{tr} + F \tag{6}$$

The strength of the resistance experienced by the car moving in the air stream is

$$F = \frac{C\rho V^2 S}{2}. \tag{7}$$

Then the power of the car P, kW, can be written

$$P = (\frac{C\rho V^2 S}{2} + F_{tr}) \cdot V \,, \tag{8}$$

Hence, it is expressed the force of friction F_{tr}, H, the car on the road

$$F_{tr} = \frac{P}{V} - \frac{C\rho V^2 S}{2}. \tag{9}$$

The power expression, but for the changed parameters, is

$$P' = (\frac{C'\rho V^2 S}{2} + F_{tr}) \cdot V \,, \tag{10}$$

Considering that the force of car friction on the road has not changed

$$P' = (\frac{C'\rho V^2 S}{2} + \frac{P}{V} - \frac{C\rho V^2 S}{2}) \cdot V. \tag{11}$$

Fuel consumption in liters per 100 km of mileage is expressed in the form of the following dependence

$$Q_s = \frac{g_e \cdot P^{/}}{100 \cdot V \cdot \gamma_T}, \qquad (12)$$

where g_e -- specific fuel consumption, g / kWh $^{-}$;
γ_T – fuel density, for diesel, γ_T = 0,84 kg / l;
V – car speed, km / h.

To determine and construct a mathematical model, several tractors were given in Table 1. The results of the parameters are given in Table 2.

Due to the many tests, it was determined that the size of the propulsion coefficient strongly depends on the vehicle shape e and the attachment. The tests results are in Table 3.

Further calculations are presented in Table 4 and presented in the form of charts in Figures 3 and 4.

Table 1 – Constructive features a sidecar tractor constructions in terms of overall dimensions

Car make	Cabin	Height, mm	Width, mm	Lenght, mm	Wheel base, mm	Power, kW
MAN	average	3600	2500	6100	3600	265
MAN	low	2900	2500	5800	3600	300
Renault	average	3600	2400	5900	3700	320
Renault	low	3100	2500	6100	3600	315
DAF	average	3500	2500	5900	3700	335
DAF	low	2600	2400	6800	3600	300
VOLVO	average	3600	2500	6800	3800	280
VOLVO	low	2800	2400	5900	3600	280
SCANIA	average	3400	2500	6500	3600	310
SCANIA	low	3100	2500	5900	3600	300

Table 2 – Average tractor parameters are given

№	Height of the cab	Length, mm	Width, mm	Height, mm	Engine power, kW	Fuel consumption, l / 100km
1	Average	6200	2500	3500	300	19 - 21
2	Low	6200	2500	2900	300	20 - 22

Table 3 – Change of the frontal load resistance from the car and ramp parameters

№	Height of the cab	Flying	Location	Frontal resistance coefficient C_x
1	Average	No	-	1,08
		Ordinary	Behind	0,77
		Improved		0,52
2	Low	Missing	-	1.09
		Normal	Behind	0.85
		Improved		0.57
		Improved	AheadWe	0.87

$$P^{/} = (\frac{0.52 \cdot 1.2041 \cdot 25^2 \cdot 10}{2} + \frac{300000}{25} - \frac{1 \cdot 1.2041 \cdot 25^2 \cdot 10}{2}) \cdot 25 = 254.85$$

$$Q_s = \frac{510 \cdot 254.85}{100 \cdot 90 \cdot 0.84} = 17.19$$

Table 4 – Estimated power and fuel consumption, depending on the change in the strain coefficient

№	Cabin height	Roof Fairing	Position	C_x	P_{90}	Q_{90}	P_{60}	Q_{60}	P_{40}	Q_{40}
1	Middle	Absent	–	1 – 1.09	300 - 308.8	20.2 – 20.8	162.5 - 167.8	16.7 – 17	99.7 - 100.4	15.13 - 15.24
2		Ordinary	–	0.77 – 0.86	278.36 - 286.8	18.8 – 19.3	158.8 - 161.3	16.06 - 16.3	97.8 – 98.5	14.84 - 14.95
3		Improved	Backward	0.52 – 0.61	259 – 268	17.5 – 18.1	151.8 - 154.3	15.36 - 15.6	95.7 - 96.45	14.5 - 14.64
4	Low	Absent	–	1.05 – 1.14	304.7 - 313.2	20.55 - 21.1	166.86 - 17.1	16.86 - 17.1	100.1 - 100.8	15.2 – 15.3
5		Ordinary	–	0.85 – 0.94	285.9 - 294.35	19.3 – 19.9	16.3 – 16.5	16.3 - 16.5	98.4 – 99.2	14.94 - 15.05
6		Improved	Backward	0.57 – 0.66	254.8 - 263.3	17.2 – 17.8	15.5 – 15.8	15.5 - 15.8	96.1 – 96.9	14.6 – 14.7
7		Improved	In front	0.87 – 0.96	287.7 - 296.2	19.4 – 20	16.35 - 16.6	16.35 - 16.6	98.6 – 99.3	14.97 - 15.07

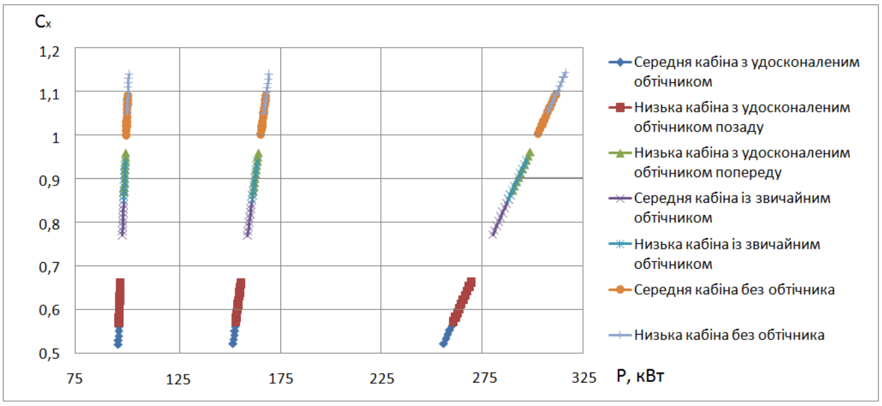

Figure 3 - Dependence of power consumption on the air flow coefficient at speeds 40 - 90 km / h

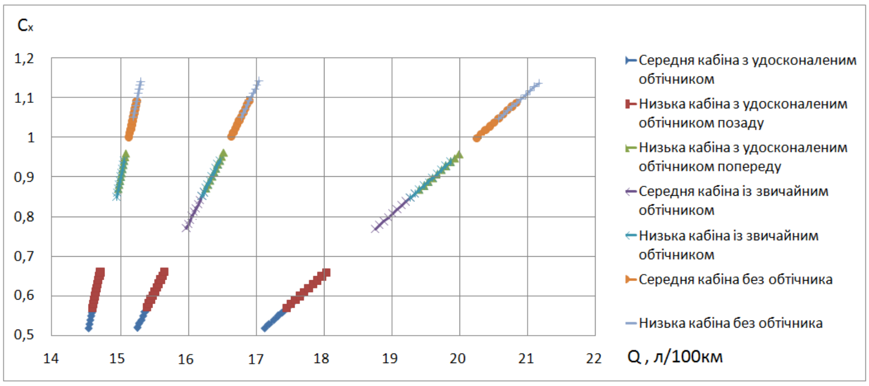

Figure 4 - Dependence of fuel consumption on the coefficient of airworthiness at speeds 40 - 90 km / h The

Graph (Figure 3) represents the ratio of power to the vehicle speed. The larger the coefficient of frontal resistance C_x is, the worse the car overcomes the air masses. All sharp edges, protruding surfaces, and streamlined elements affect the car airiness. The graph shows various trains with and without their improvements. The sloping lines correspond to the car speeds 40, 60 and 90 km/h, respectively. From the graph, it is evident that at speed of 40 km/h, a carriage train with or without a racing gun spends almost the same power on overcoming a given speed. At speed of 60 km/h, the difference between good and bad traffic trains starts to appear. At a speed of 90 km/h, there is a strain significant influence on power car. It is due to the fact that when speed increases, the required power increases to overcome the resistance forces that can stop the auto-train.

The graph (fig. 4) shows the change in the amount of fuel consumed by the trains traction. On the graph, sloping lines correspond to speeds of 40, 60, 90 km/h, respectively. As in Figure 3, there is a tendency to increase the speed ratio to the amount of fuel consumed. With the increase in the speed of different tractors, the expired power increases, and therefore, the fuel amount consumed per 100 km of road increases.

The relative economy of a tractor with a low cabin and an improved ramp is 16.3% of the tractor without an outboard and 10.9% of the fitted with a conventional racing gun. For the average cab with improved racing, the figures are 13.4% and 7.1% respectively.

5-10 shows the increase in the amount of fuel consumed from the increase in car power at speeds of 40, 60 and 90 km / hr, respectively, for trains with different improvements and different corduroy. With low speed and even motion, the car spends the smallest amount of fuel and power. With an increase in speed up to 60 km/h, the car spends approximately 2 times more power in support of a given speed. With increase in speed up to 90 km/h, the power increases almost 3 times. It is due to the fact that the motionvelocity has a quadratic function and a parabolic character.

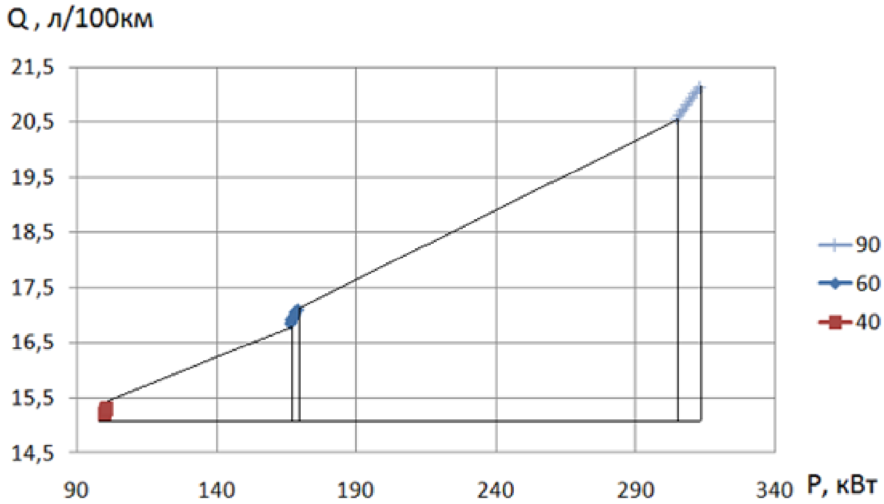

Figure 5 – Power dependence on the amount of fuel consumed by the low cab without a rudder

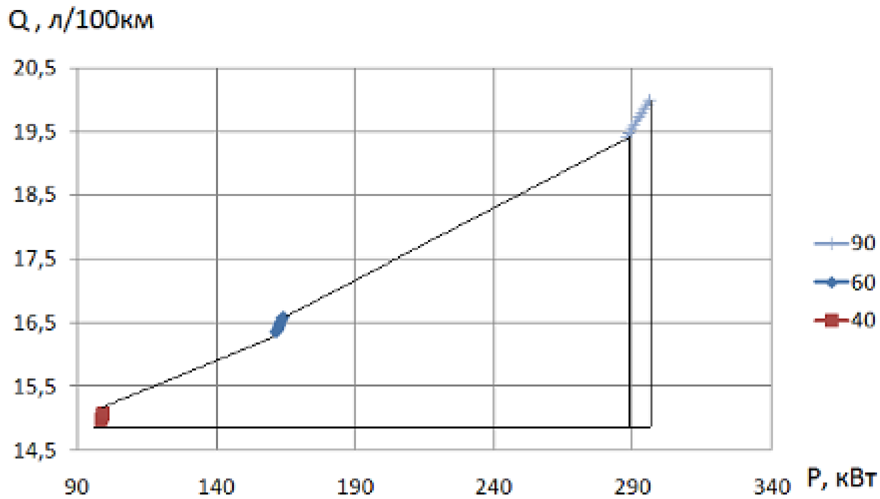

Figure 6 – Power dependence on the amount of fuel consumed by the lower cabin with improved ramp ahead

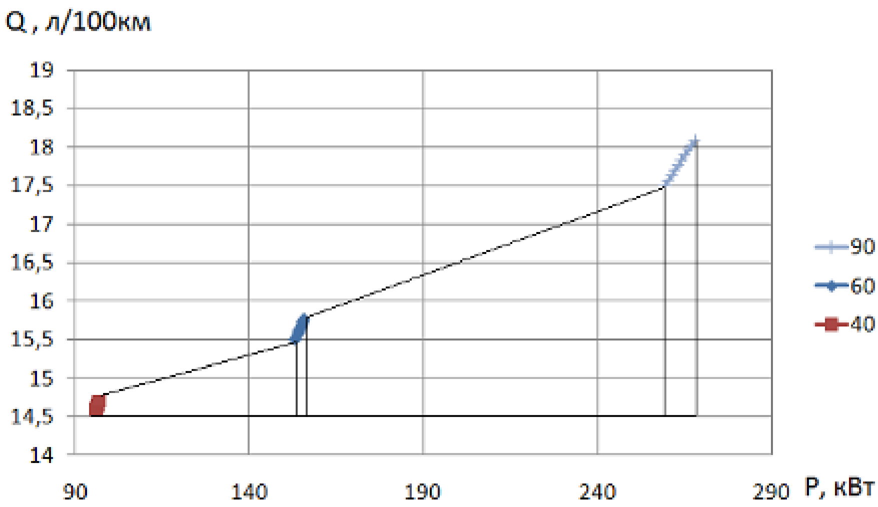

Figure 7 – Power dependence on the amount of fuel consumed by the low cabin with improved ramp behind

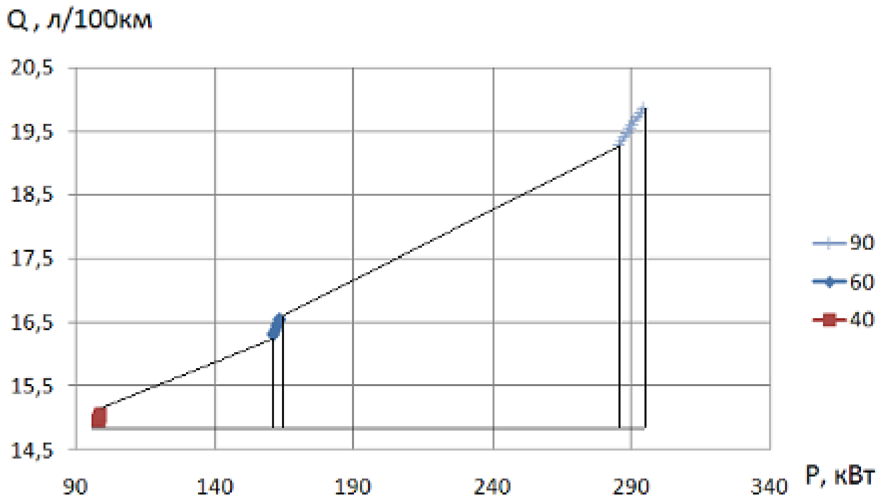

Figure 8 – Power dependence on the quantity consumed fuel of a low cabin with a normal riviera

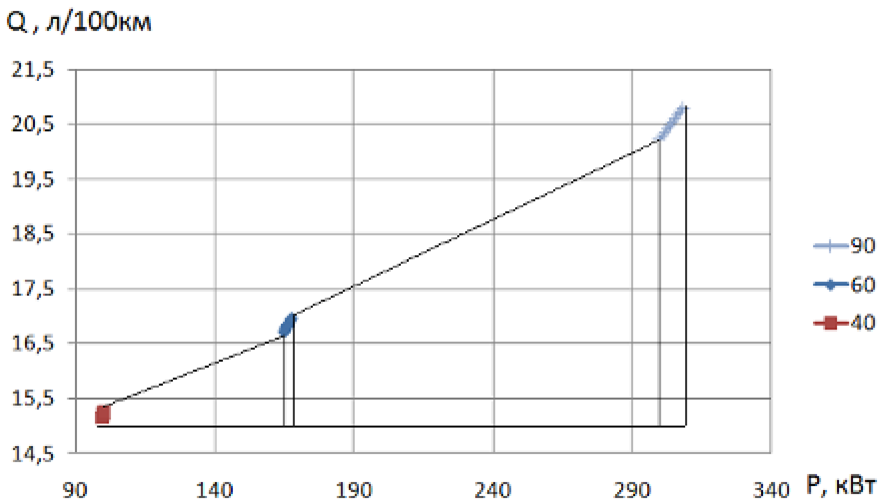

Figure 9 – Power dependence on the amount of fuel consumed in the average cabin without a ramp

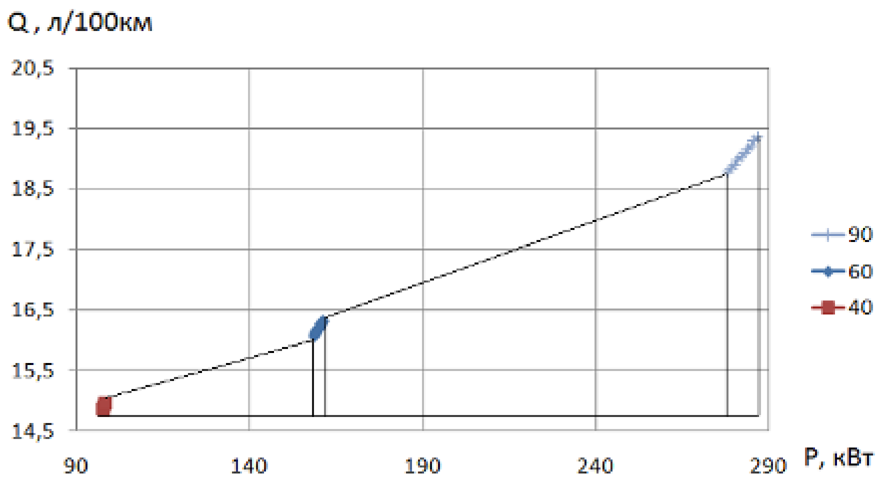

Figure 10 – Dependence of power relative to the amount of fuel consumed by the average cabin with a conventional rampramp

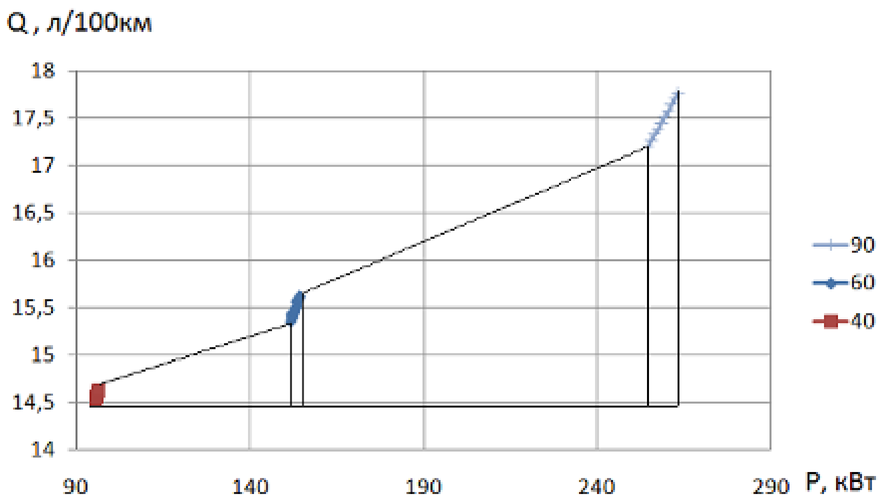

Figure 11 – Power dependence on the amount of fuel consumed in the average cabin with an improved figure

Conclusions The theoretical study of the strain influence on the main motor tracks performance enables to achieve the following results:

– after analysis the existing flow elements installed on the main train trailer and the ramps revised large number installed directly on its roof, it was substantiated the expediency of installing a movable rope on the tractor roof, which can change the angle of airflow trailing link with the combination of two movements - vertical and horizontal;

– the method of calculation and the calculations of power and quantity of fuel consumed for various types of booths and their possible improvements by the proposed moving ramp are proposed;

– an imaginary main motorway is divided into the air flow zone with its flow in the road projection and determine the air velocity in these zones.

The next step of the research is the development of a schematic diagram and the roof ramp design, a hydraulic system for changing the parameters of the roof rack with the possibility of adjusting for any trailer structure, as well as possible options for connecting and adjusting its parameters.

References

1. Карабцев, В.С. & Валеев, Д.Х. (2011). Аэродинамика плохобтекаемых тел и возможности ее применения при проектировании грузовых автомобилей. *Механика машин, механизмов и материалов,* 4, 97-102.

2. Cooper, K.R. (2004). Commercial Vehicle Aerodynamic Drag Reduction: Historical Perspective as a Guide. *The Aerodynamics of Heavy Vehicles: Trucks, Buses, and Trains.* (pp.9-28). Berlin, Heidelberg: Springer,. doi:10.1007/978-3-540-44419-0_2

3. Pevitt, C., Chowdury, H., Moriaand, H. & Alam, F. (2012). *A Computational Simulation of Aerodynamic Drag Reductions for Heavy Commercial Vehicles*. 18th Australasian Fluid Mechanics Conference. Launceston, Australia. Retrieved from https://people.eng.unimelb.edu.au.

4. Евграфов, А.Н. (2010). *Аэродинамика автомобиля*. Москва: МГИУ.

5. Гухо, В.Г. (1987). *Аэродинамика автомобиля.* Москва: Машиностроение.

6. Гришкевич, А.И. (1968). *Автомобили: теория*. Минск: Высшая школа.

7. Пилипенко, О.М., Батраченко, В.О. & Литовченко, І.М. (2017). Моделювання аеродинаміки сідельного автопотягу. *Вісник Хмельницького національного університету*, 2, 27-33.

8. Khosravi, M., Mosaddeghi, F. & Oveisi, M. (2015). Aerodynamic drag reduction of heavy vehicles using append devices by CFD analysis. *Journal of Central South University,* 22, 4645-4652. doi:10.1007/s11771-015-3015-7.

9. McCallen, R., Flowers, D., Dunn, T., Owens, J. et al. (2000). Aerodynamic Drag of Heavy Vehicles (Class 7-8): Simulation and Benchmarking. *SAE Technical Paper* 2000-01-2209. doi:10.4271/2000-01-2209.

10 Khaled, M., Elhage, H., Harambat, F. & Peerhossaini, H. (2012). Some innovative concepts for car drag reduction: A parametric analysis of aerodynamic forces on a simplified body. *Journal of Wind Engineering and Industrial Aerodynamics,* 107/108, 36-47.
doi:10.1016/j.jweia.2012.03.019

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Specificity of strength calculation for glued-in steel rods in LVL with unidirectional veneer

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Design of LVL elements with glued-in steel rods and metal connectors joints is considered as semi-rigid connection and requires considering the compliance. The beams with a metal connector and glued-in steel rods and solid beams test results comparative analysis has been made in the paper. Design method of glued-in rods in LVL is proposed and failure mode is considered. It enables reducing the distance between the rods axes and the distance from the rod axis to the edges in the beam cross section and increasing the joint strength.

Keywords: glued-in steel rods, semi-rigid connection, laminated veneer lumber Ultralam – R), axes and edge distance, model of rupture, two-side fixing.

Особливості розрахунку міцності вклеєних стержнів у ЛВЛ брусі з однонаправленим шпоном

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При проектуванні вузлових з’єднань ЛВЛ брусу на вклеєних стержнях з металевими вставками є напівжорсткими та потребують врахування податливості. У статті виконано порівняльний аналіз результатів випробувань балок з металевим вузлом на вклеєних стержнях та цільних балок. Розроблена методика розрахунку міцності вклеєних стержнів на висмикування з урахуванням характеру руйнування зразків, яка дозволяє знизити відстані між осями стержнів та від осі стержня до граней у поперечному перерізі балки та збільшити міцність з’єднання. Овалізація деревини навколо вклеєного стержня при руйнуванні з’єднання обумовлена різницею міцності ЛВЛ брусу при сколюванні вздовж волокон по пласті та по грані, що насамперед є результатом шпонової структури цього брусу та враховується при розрахунках руйнівного зусилля згідно до запропонованої методики. Проведений аналіз рекомендованих відстаней між вклеєними стержнями, які регламентовані стандартами різних країн, дозволив сформулювати рекомендації щодо конструктивних вимог при проектуванні з’єднань на вклеєних стержнях у ЛВЛ брусі. Результати розрахунків балок з металевою вставкою, отриманих у аналітичному програмному комплексі ANSYS, показали незначну розбіжність відносно результатів, отриманих при експериментальних натурних випробуваннях балок, яка складала біля 13%. Руйнування усіх балок відбувалось через висмикування вклеєних стержнів у нижній розтягнутій зоні балок. Для збільшення несучої здатності з’єднання, зменшення розтріскування деревини та забезпечення в’язкого руйнування рекомендується встановлення гвинтів у напрямі поперечному до напряму вклеєних стержнів. Також для зменшення деформативності напівжорстких з’єднань з металевою вставкою можливо встановлювати гайки з двох сторін для включення стержнів у роботу на продавлювання у стиснутій зоні. Досліджений вузол с металевою вставкою на вклеєних стержнях може використовуватись при створенні сітчастих оболонок складної просторової геометричної форми.

Ключові слова: вклеєні стержні, напівжорстке з’єднання, шпоновий брус ЛВЛ (Ultralam – R), відстань між стержнями, модель руйнування, двостороння фіксація вузла.

Introduction. The problem of strength and deformability of node joints on glued-in rods and a metal connector using an LVL beam is extremely relevant and has been considered when designing complex geometric shape atrium project (Fig. 1), where a nodal solution has been proposed that requires testing. Its results are presented in this publication. The lack of recommendations regarding the calculation and design of the joints on the glued-in rods in the structural elements of LVL beam increased the relevance and interest in this type of connections. The geometry of the atrium computational model has been created in the Tekla Structures program, and then exported to the SCAD Soft program to make the calculation.

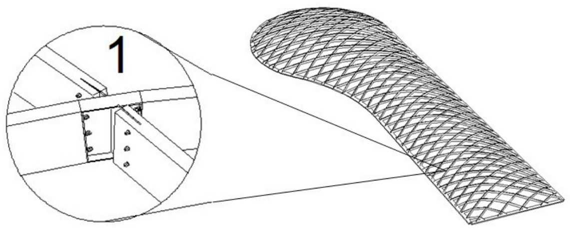


Figure 1 – General view of the atrium and nodal joint

Review of the latest research sources and publications. The peculiarities of the LVL beam with the help of glued-in rods also arouse great research interest of colleagues from different countries, including M. Stepinac [13], R. Steiger, E. Gehri, A. Buchanan, E. Serrano, N. Meyer, and others. In the CIS countries, glued-in rods as a rigid joint of timber structures are an joints integral engineering solution when designing frameworks for large-span buildings. In practice of the CIS countries, the glued-in rods use is cheaper than screw joints, which are popular in the European countries and are an alternative to glued-in rods. However, the elements of the well-known construction "Metropol Parasol" in Seville (Spain), which is a landmark of this city, are made with the help of LVL beams on glued-in rods. Glued-in rods in timber structures joints are used in two versions, forming rigid and semi-rigid joints: joining timber-to-timber or with welding glued-in rods to the metal element (Fig. 2-a) and joining timber to the metal connector on the bolts (Fig. 2-b).

The first variant of the joint is usually prefabricated, which is not demountable. The second variant of the joints or semi-rigid joints is increasingly used in combined frames or complex systems of structures, where metal-free solutions are impossible.

a) b)

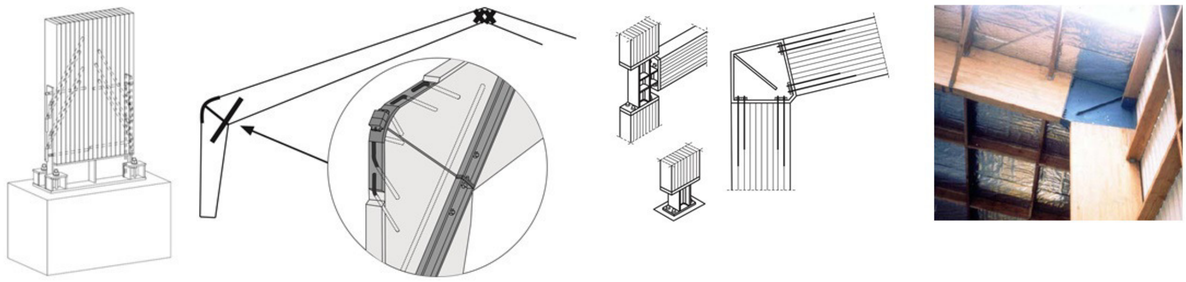


Figure 2 – Rigid and semi-rigid joints with glued-in steel rods:
a – rigid heel joint in a foundation column and a moment joint in a three-hinged frame;
b – semi-rigid moment joint and heel joints

Definition of unsolved aspects of the problem. The bolted joints compliance of timber structures on glued-in rods semi-rigid joints is not only joints deformations, but also glued-in rods ductility, which is correspondingly less than the first one. For inclined glued-in rods according to the norms [1], the compliance is 0.001 mm/kN. There are known the methods for compliance accounting of dowel joints in timber structures such as trusses, due to the large number of the perforating structure elements nodal joints, which have a significant effect on the magnitude of the total structural deformation or deflection. The deflection determination for trusses with lower height is especially important. The manual of SNiP on the calculation of timber structures [10] proposed to determine the movement of truss nodes, considering the joints compliance according to the rules of structural mechanics with the introduction of the reduced modulus of elasticity (Section 6.29). In the tutorial of professor Serov E.N. [8] it is indicated that if there are specific normative values for the joints compliance and an arbitrary level of their bearing capacity use, the deflection caused by the joints compliance definition, considering the forces acting in them, should be determined by the formula:

$$f_c = f_{tj} + f_{aj} = \sum\nolimits_1^{mk} \Delta_{Hi} \frac{N_{ci}}{N_{H.ci}} + \sum\nolimits_1^{m} \Delta_{Hi} \frac{\sigma_{смi}}{R^{\alpha}{}_{смi}} \qquad (1)$$

where: f_{tj} – deformation of compliance in tension dowel joints;

f_{aj} – deformation of compliance in the angle joints and buffer stops;

k – number of dowel joints;

m – number of joints on the angle joints and buffer stops;

Δ_{Hi} – the normative value of compliance of the i-th joint at its full load-carrying capacity;

N_{ci} – force in the i-th joint;

$N_{H.ci}$ – load-carrying capacity of the i-th dowel joint;

$\sigma_{смi}$ – crumple stress in the i-th joint;

$R^{\alpha}{}_{смi}$ – design resistance to crumble in the i-th joint.

Problem statement. High strength and cost, slightly exceeding the laminated timber cost, ensured high popularity in the market of timber construction for veneer lumber or LVL based on softwood produced by Ultralam and Kerto. The veneered structure of the LVL beam requires additional tests of timber structure joints classical types, which are often used for laminated timber elements. The behavior of LVL and its destruction during tests of joints structural elements differ from laminated timber and require the additional recommendations regarding the rules for the nodal joints design and their calculation.

Basic material and results. Under the guidance of professor Fursov V.V. in 2016, a nodal joint on glued-in rods with a metal connector installed in the middle of an LVL beam with a unidirectional arrangement of Ultralam veneer was tested (type R). Fig. 3 shows the beams loading with static load and the installation location of measuring devices for determining deformations. Tests of the nodal connection were the key and the LVL beam research final logical stage [7, 9], since for the node complete analysis, tests of strength and elastic characteristics under compression from different angles, specimens for chipping and specimens for pulling single glued-in rods were previously performed.

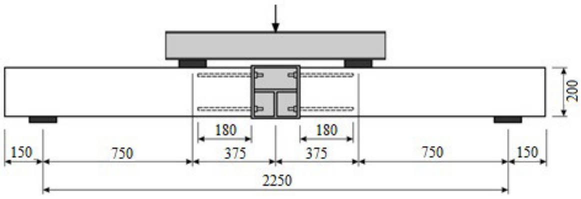


Figure 3 – The loading condition of beams with a metal connector

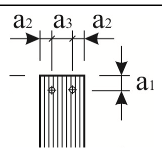
The first type of the beams were tested with a cross section of 75x200 mm, a span of 2.25 m with a node located in the middle of the beam, where M16 studs were installed in upper and lower parts. The second type of the beams differed from the first one that in the upper part of the cross section M10 studs were glued-in, and in the lower part M16 studs were glued-in. Solid beams without a nodal connection were also tested. Strength grade of steel studs is 5.8. Adhesive compound is based on resin ED-20. The insertion depth was 180 mm and the diameter of all holes was made 2 mm larger than the diameter of the glued-in rod. The distances between the axes of the rods in all sections were taken to be the same (Fig. 4). The tests of the beams were carried out by static loading them with a 25-ton jack through a traverse, distributing the load to 2 points in the thirds of the span. The model was detached from the plane on the supports and in the places of load application.

Before the test, each beam was loaded for 1 ton and was completely unloaded; then all the nuts in the joint were tightened. Load step was 1 t on the traverse, an exposure load at each step was 2 min. All beams were brought to destruction. The displacements were measured on supports and in the span using measurement devices with a scale value of 0.001 mm.

Figure 4 – The location of rods and the joint after destruction

Results and discussions. The pattern of specimen destruction has a typical brittle character in the form of pulling out rods covered with timber layer or wedge-shaped chipping of timber around rods. When drilling holes, the center distance and the distance to the edges were taken to be less than recommended by the standards of various European countries and European technical conclusions [1-5], as shown in Table 1, where d is the diameter of the glued-in rod. Considering the tests results of single glued-in rods under axial loading in specimens of LVL beam with unidirectional veneer, it was noted that the fracture pattern has a characteristic splitting and a small area of get out timber, as detailed in [7]. In addition to single glued-in rods, specimens with two glued-in rods with a center distance of less than recommended were also tested. For example, according to the norms of Russia on the design of timber structures SP 64.13330.2017 [1] the minimum distance from the package sides to the rod axis is taken to be at least 2d and the distance between the axes of the rods should be at least 3d.

Table 1 – Distances between the glued-in steel rods, according to the standards of different countries

			
	a_1	a_2	a_3
СП 64.13330.2017 (Russia) [1]	2d	2d	3d
DIN 1052 DIN EN 1995-1-1 (Germany) [4]	2.5d	2.5d	5d
ÖN B 1995-1-1 (Austria) [5]	2.5d	2.5d	5d
New Zealand [11]	2.5d	2.5d	5d
R. Steiger (Switzerland) [12]	2.3d	2.3d	4d
Z-9.1-791 [2]	1.75d	1.75d	3.5d
Z-9.1-778 [3]	1.875d	1.875d	3.75d
Tests	1.16d (18.5 mm	1.16d (18.5 mm	2.37d (38 mm)

The reduced axial spacing of the glued-in rods and the distances to the edges did not violate the proposed pattern of destruction, since the area of the get out timber did not reach the edges or outer edges, as shown in fig. 5. An analysis of the rods destruction behavior enables to note that the area of the sheared timber is ovalized around the glued-in rod, see fig. 6. The largest part of the timber is cleaved along the layers of veneer in the cross section of an LVL beam, where the area of the timber does not exceed 4 layers of veneer with a total width of 13-15mm.

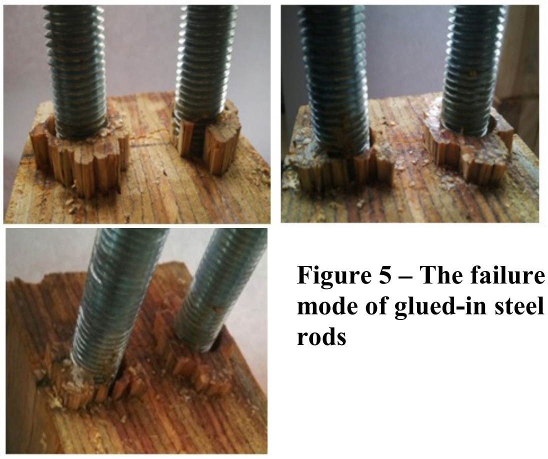


Figure 5 – The failure mode of glued-in steel rods

Beams were also tested with a modification of the joint, by installing studs with a smaller diameter in the upper cross-sectional area, which as a result had little effect on the breaking load and the beam deformability in the middle of the span, see fig. 7. The upper rods were not included in the extrusion, but only fixed a metal connector, which crushed the upper part of the beam cross-section. Also, the deformation in the beam upper zone was increased by crumpling the rods thread in the upper zone. The level of the tightened nuts was not monitored.

a)

b)

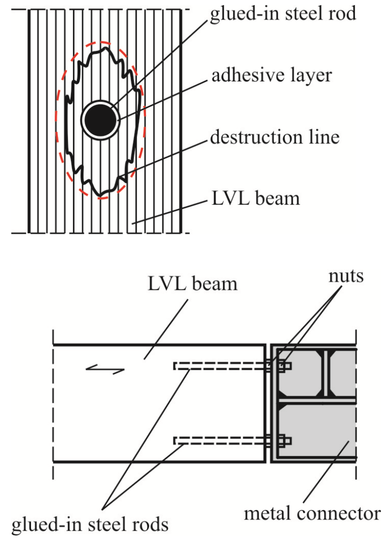
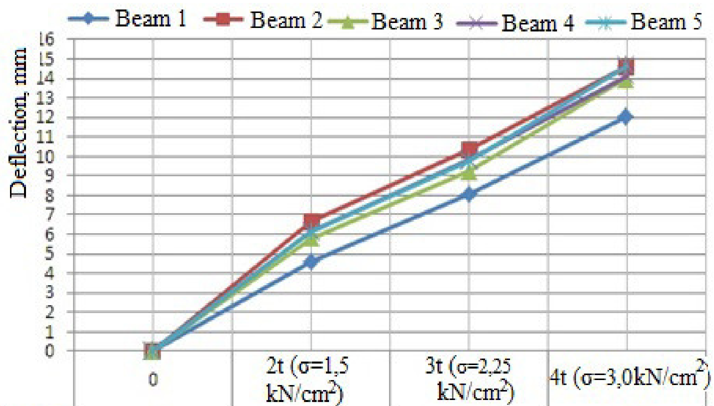


Figure 5 – The failure mode of the glued-in steel rods which were placed parallel to the grain in a LVL beam:
a – ovalization of the timber shear area;
b – two-side fixing

When testing 5 beams with a metal connector, the values of the beams deflections at three points were analyzed separately: in the middle of the span and in the thirds of the beam span, where concentrated forces were applied. The results of beam mid-span deformations at various loading levels are shown in Fig.7, where a strong coincidence of the beams deformation curves can be observed 2-5. Breaking load for beams was observed in the range from 48 kN to 55 kN. A slight divergence of the beams curves deformations in the middle of the span increased with an increase of the load close to destructive.



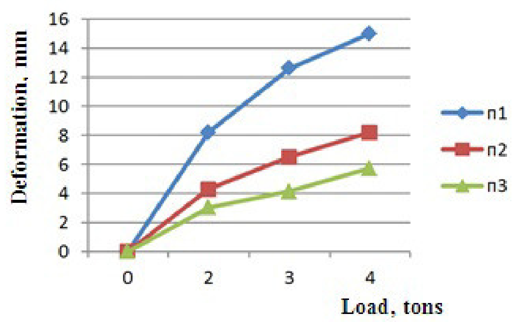
	0	2t (σ=1,5 kN/cm²)	3t (σ=2,25 kN/cm²)	4t (σ=3,0kN/cm²)
Beam 1	0	4,586	8,102	12,046
Beam 2	0	6,668	10,384	14,604
Beam 3	0	5,8	9,26	14,024
Beam 4	0	6,13	9,85	14,13
Beam 5	0	6,126	9,782	14,648

Figure 7 – Beam deflection in the middle of the span with a metal connector

For a comparative assessment of the beam deformability with a connector, tests of solid beams without a node were performed. Curve n1 in fig. 8 shows the deformations in the beam mid-span, and the curves n2 and n3 show the beam deformation in thirds of the span. Fig. 8-a) shows the deformations of the solid beam with a characteristic significant excess of the deformation or deflection in the middle of the span. The beam with the connector at the initial stages of loading had a deformation in the span slightly higher than in the thirds of the span, since the beam stiffness in the node is higher due to the metallic elements of the connection and the lack of the glued-in rods pliability.

It should be considered that the beam geometric model in the calculation is idealized and free of a number of inaccuracies associated with the connection on the glued-in rods implementation, the contact surfaces quality, etc., which constitute loose deformations that are not considered in the design software packages.

a)



b)

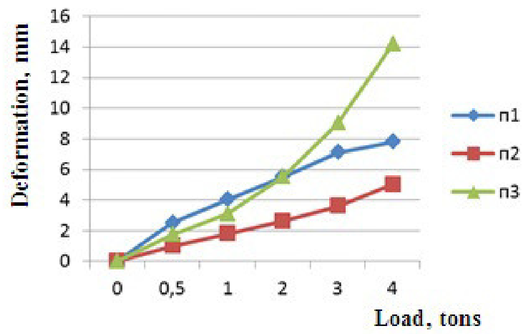


Figure 8 – Deflection curves of a solid beam (a) and a beam with a connector (b)

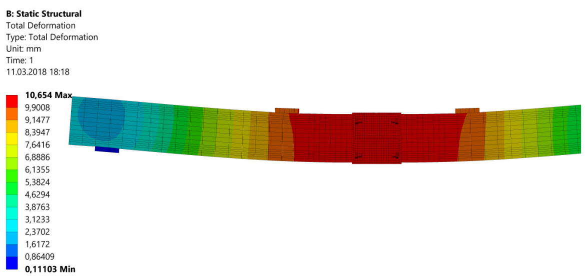


Figure 9 – Beam deformation according to calculation

The proposed method for calculating the strength of glued-in rods includes a number of prerequisites in the form of parameters borrowed from various formulas for calculating glued-in rods in solid or glued timber. The need to develop a modified method of calculation is primarily due to the large difference between the experimental strength values and the various analytical data on the joint strength. Also, the developed formula (2) for the calculation of rods in an LVL beam includes not only the main design parameters (see Fig. 10), but also the features observed during testing, such as the ovalization of sheared timber around the rod, and therefore the strength value when splitting LVL timber along the grain on the edge and on the face.

$$R_{ax,k} = \pi \cdot d_h \cdot l \cdot (f_{v,k,ed} \cdot f_{v,k,fl})^{0,5} \cdot k_c \tag{2}$$

where: $R_{ax,k}$ – the characteristic pull-out capacity of GIR in LVL with unidirectional veneers, in N;

$f_{v,k,ed}$, $f_{v,k,fl}$ – the characteristic shear strength flatwise and edgewise for LVL with unidirectional veneers (Ultralam type R) in N/mm^2;

l – anchorage length;

d_h – drilled hole diameter;

k_c – coefficient that considers the uneven shear stress distribution depending on the rod anchorage length:

$$k_c = 1{,}2\text{-}0{,}02 \cdot l/d \tag{3}$$

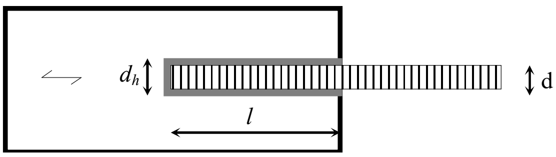


Figure 10 – The main parameters of the design model

To ensure the joint design strength on the glued-in rods, it is necessary to follow the rules of the glued-in rods placement along the grain in the beam cross-section, namely the minimum distances, as shown in Fig.11. It should be noted that the distance between the rods along the veneer layers is greater than in the perpendicular direction relative to the veneer due to the ovalization of the sheared timber around the rod formed by the difference in shear strength along the LVL grain on the face and on the edge.

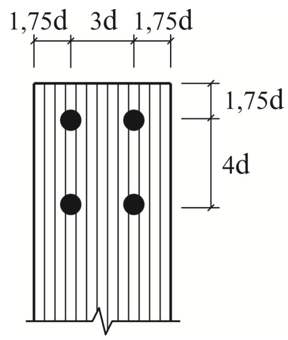


Figure 11 – Recommended distances between the glued-in rods and to the edges

The specified minimum distances between the rods and to the LVL element cross section edges can be used if the rods are accurately glued in (without distortions of the holes and the rods when gluing). To reduce the beam with a metal connector on the glued-in rods deformability, it is possible to use double-sided installation of nuts on the rods located in the upper part of the beam cross section, or in a compressed zone. It is also possible to perform a controlled tightening of the nuts not exceeding 70% of the timber compressive strength calculated value along the grain. Created prestress in the node reduces loose initial deformations by 32-40%. The complexity of tightening the nuts to a predetermined value should be considered.

The proposed calculation method and design rules are of advisory nature and require further studies to clarify some data. Fig. 12 shows the glued-in rods strength diagram obtained during the tests ("tests" curve) and the expected values, according to various norms, as well as according to the proposed method (2), the "proposing" line. Symbols of straight lines plotted on the diagram are as follows: "tests" - test results, "din" – the value expected by the German standards, "ru" – according to the Russian standards, "ru-vr" – according to the Russian standards using the temporal resistance value when splitting LVL beam, "en" – the value expected by the pan-European methodology.

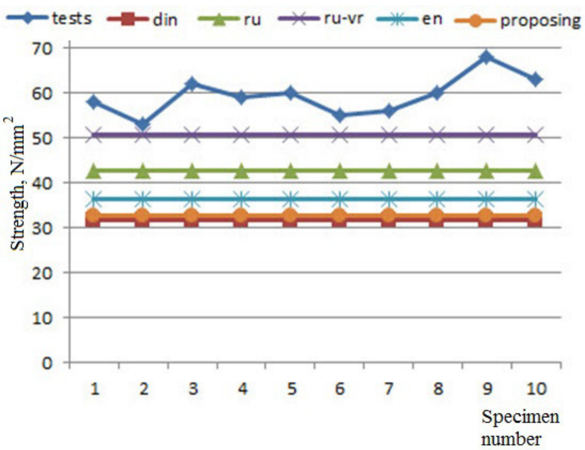


Figure 12 – The strength of the glued-in rods obtained during testing and the expected values, according to various standards, as well as according to the proposed method of calculation

Conclusions. The tests of the nodal joint on the glued-in rods with a metal connector and the analysis of the experimental data confirm the possibility of reducing the center distance between the glued-in rods to 2.4d and the distance to the faces 1.2d, while not allowing the block scheme of destruction of a group of glued-in rods. To calculate the strength of glued-in rods in a LVL beam with an unidirectional veneer installed along the grain, it is possible to use the calculation formula (2), which takes into account the deviation of the shear strength along the grain in the LVL beam. The fragile nature of the fracture with cracking along and across the veneer requires the installation of screws or the use of an LVL beam with cross veneer layers. A comparative analysis of the deformability of the beams on glued-in rods with a metal node confirms the need to take into account the compliance of this semi-rigid connection.

Also, an obvious and necessary addition to the considered type of connection with a metal connector should be performed with two-sided installation of nuts relative to the metal connector, that ensures that the glued-in rods located in the compressed zone of bending element are included to the work, potentially with a smaller diameter than the rods installed in the lower zone.

References

1. СП 64.13330.2017 (2017). Свод правил. Деревянные конструкции. Москва: Минрегион РФ.

2. Allgemeine bauaufsichtliche Zulassung Nr. Z-9.1-791 vom 17. (Januar 2012). *Verbindungen mit faserparallel in Brettschichtholz eingeklebten Stahlstäben.* Deutsches Institut für Bautechnik. Berlin.

3. Allgemeine bauaufsichtliche Zulassung Nr. Z-9.1-778 vom 31. (Oktober 2012). *2K-EPKlebstoff GSA-Harz und GSA-Härter für das Einklebten von Stahlstäben in Holzbaustoffe.* Deutsches Institut für Bautechnik. Berlin.

4. DIN EN 1995-1-1. (2013). *Eurocode 5: Bemessung und Konstruktion von Holzbauten - Teil 1-1mit NA: Allgemeines – Allgemeine Regeln für den Hochbau.* DIN-Deutsches Institut für Normung e.V.

5. ÖN B 1995-1-1:2015. (2015). *Eurocode 5: Bemessung und Konstruktion von Holzbauten - Teil 1-1: Allgemeines - Allgemeine Regeln und Regeln für den Hochbau*. Austria.

6. Standard SIA 265:2012. (2012). *Timber Structures. SIA Swiss Society of Engineers and Architects*. Zurich, Switzerland.

7. Фурсов, В.В., Бидаков, А.Н. & Распопов, Е.А. (2016). Прочность вклееных стержней на выдергивание при осевом нагружении установленных в LVL элементы с однонаправленным расположением шпона. *Збірник наукових праць Українського інституту сталевих конструкцій імені В.М. Шимановського*, 18, 24-32.

8. Серов, Е.Н., Санников, Ю.Д. & Серов А.Е. (2011). *Проектирование деревянных конструкций*. Москва: Изд-во АСВ.

9. Бидаков А.Н. & Распопов Е.А. (2017). Прочностные и упругие характеристики шпонового бруса LVL при сжатии под различными углами к наклону волокон. *Містобудування та територіальне планування: наук.-техн. збірник*, 65, 91-99.

10. *Пособие по проектированию деревянных конструкций*. (1986). ЦНИИСК им. Кучеренко. Москва: Стройиздат.

11. NZW14085 SC. (2007). New Zealand Design Guide. Timber Industry Federation.

12. Steiger, R., Gehri, E. & Widmann, R. (2007). Pull-out strength of axially loaded steel rods bonded in glulam parallel to the grain. *Materials and Structures*, 40(1), 827-838. doi:10.1617/s11527-007-9251-z

13. Steiger, R., Serrano, E., Stepinac, M., Rajcic, V., O'Neill, C., McPolin, D., & Widmann, R. (2015). Strengthening of timber structures with glued-in rods. *Construction and Building Materials*, *97*, 90-105. doi:10.1016/j.conbuildmat.2015.03.097

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