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Poltava National Technical Yuri Kondratyuk University

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У збірнику представлені результати наукових і науково-технічних розробок у галузі машинобудування, автомобільного транспорту та механізації будівельних робіт; із проектування, зведення, експлуатації та реконструкції будівельних конструкцій, будівель і споруд; їх основ та фундаментів; будівельної фізики та енергоефективності будівель і споруд.

Призначений для наукових й інженерно-технічних працівників, аспірантів і магістрів.

Адреса видавця та редакції – Полтавський національний технічний університет імені Юрія Кондратюка
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COOPERATION OF UKRAINIAN SOCIETY FOR SOIL MECHANICS, GEOTECHNICS AND FOUNDATION ENGINEERING WITH INTERNATIONAL SOCIETY ISSMGE

Information about cooperation of Ukrainian Society for Soil Mechanics, Geotechnics and Foundation Engineering with International Society for Soil Mechanics and Geotechnical Engineering, the results of Ukrainian society activity and prospects of its development are presented in paper. It is considered participation of Ukrainian specialists in International and regional conferences; results of the ninth All-Ukrainian scientific and technical conference «Soil mechanics, geotechnics and foundation engineering: problems, innovations and implementation of Eurocodes in Ukraine» and prospects of Ukrainian society development.

***Keywords:** International society for soil mechanics and geotechnical engineering, Ukrainian society for soil mechanics, geotechnics and foundation engineering.*

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СПІВПРАЦЯ УКРАЇНСЬКОГО ТОВАРИСТВА МЕХАНІКИ ҐРУНТІВ, ГЕОТЕХНІКИ І ФУНДАМЕНТОБУДУВАННЯ З МІЖНАРОДНИМ ТОВАРИСТВОМ ISSMGE

Наведено інформацію про співпрацю Українського товариства механіки ґрунтів, геотехніки і фундаментобудування з Міжнародним товариством механіки ґрунтів та геотехніки, зокрема про результати діяльності Українського товариства і перспективи його розвитку. Розглянуто участь українських фахівців у міжнародних та регіональних конференціях; результати дев'ятої Всеукраїнської науково-технічної конференції «Механіка ґрунтів, геотехніка і фундаметобудування: проблеми інновації та імплементація Єврокодів в Україні».

***Ключові слова:** Українське товариство механіки ґрунтів, геотехніки і фундаментобудування, Міжнародне товариство механіки ґрунтів та геотехніки.*

Introduction. The All-Ukrainian Public Organization «Ukrainian society for soil mechanics, geotechnics and foundation engineering» (UkrSSMGE) as juridical person has been established in 2001 on the constituent congress of the delegates from all Ukraine regions. Kyiv's national University of construction and architecture, Odesa state Academy of Iconstruction and architecture and State enterprise «Research Institute of building constructions» have been the initiators of its establishment.

In 2003 UkrSSMGE was admitted to membership of International society for soil mechanics and foundation engineering (ISSMGE).

International Society for Soil Mechanics and Geotechnical Engineering

Société Internationale de Mécanique des Sols et de la Géotechnique

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30th November 2004

CERTIFICATE

All-Ukrainian Public Organization "Ukrainian Society for Soil Mechanics, Geotechnics and Foundation Engineering" has been a full member of the International Society for Soil Mechanics and Geotechnical Engineering since August 2003.

PROFESSOR R. N. TAYLOR
SECRETARY GENERAL
30 NOVEMBER 2004

ISSMGE SIMSGE

Figure 1 – Certificate issued for Ukrainian society

Analysis of recent sources of research and publications. Cooperation between UkrSSMGFE andISSMGE is leading by two directions [1 – 4]:

1) Organization of the Ukrainian specialists participation in International and regional conferences with reports deal with national achievements in area of geotechnics, particularly in decision of the complex issues in interaction system «soil base – foundation – structure» in different engineer-geological, natural and anthropogenic conditions. Ukraine has support from ISSMGEas the increased quotas for papers quantity for these conferences. Young Ukrainian geotechnics get the ISSMGE grants on free of charge participation in youth conferences under auspices of ISSMGE.

2) Receiving of significant number of scientific documents deal with world achievements in geotechnics field, including materials deal with complex problems of interaction of the geotechnical and structural decisions and seismicity.

Identification of general problem parts unsolved before. Today, the information about cooperation of Ukrainian Society for Soil Mechanics, Geotechnics and Foundation Engineering with International Society for Soil Mechanics and Geotechnical Engineering and the results of Ukrainian society activity and prospects of development its is necessary to analyses.

Purpose of this paper is to inform the Ukrainian geotechnical engineers about cooperation results with International society for soil mechanics and foundation engineering.

Basic material and results.

1 Participation of the Ukrainian specialists in International and regional conferences

The most developed side of cooperation between UkrSSMGFE and ISSMGE is participation of the Ukrainian geotechnical engineers in International, regional and thematic conferences. Particularly, the 18-th International conference on soil mechanics and geotechnics (September 2013, Paris, France) under slogan «Challengers & innovations in geotechnics» was devoted to innovation solutions in geotechnics [1]. Significance of geotechnics for environment protection, behavior of system «soil-foundation-structure» out the seismic collapse boundary, enhancement of pressiometric tests usage, geotechnics place in maintenance of historical monuments etc. was considered on plenary session. Ukrainian geotechnical society has been presented 5 reposts which were included in conference proceedings.

The specialized exhibition was organized in conference frames. More than 70 firms from all over the world took participation and 20 major firms have been the sponsors for this measure.

On September 2015 the 16-th European geotechnical conference under slogan «Geotechnics for infrastructure and development» hold in Edinburg (Great Britain) [2]. More than one thousands delegates participated in that measure and approximately 400 reports were presented there. Ukraine presented 6 reports which were published in conference proceedings.

On September 2017 2015both the 19-th International conference on soil mechanics and geotechnical engineering 6-th young geotechnical engineers' conference hold in Seoul (Korea). On the first from them, Ukraine presented 2 reports:

– Innovative projects in difficult soil conditions using artificial foundation and base, arranged without soil excavation(authors from Kyiv, Poltava, Zaporog'e and Dnipro);

– Innovative design and technological solutions and test method for pile supports with increased bearing capacity (authors from Odessa and Kharkiv).

UkrSSMGFE assisted to society member O. Samorodov to get the grant from conference organizers (Korean geotechnical society) for free of charge participation in conference.

Ukraine has presented too two reports on the young geotechnics conference:

– Design of foundations for low-rise buildings in dense urban environment(author M.Korzachenko, Chernigiv);

– Method for determining bored piles load-bearing capacity, taking into accountthe direction of vertical loading in water-saturated soils(author S.Tabachnikov, Kharkiv).

UkrSSMGFE assisted to S.Tabachnikov to get grant from ISSMGE for free of charge participation in conference.

By the way, the next geotechnical conferences would be held such way:

– 16-th European Danube geotechnical conference (www.decge2018.mk) – in 2018 in Skopje, Macedonia;

– 17-th European geotechnical conference under slogan «Geotechnical engineering – foundation for the future»– in 2019 in Reykjavik, Island.

2 Analysis of the International conferences materials

The following problems, which are actual ones for nowadays for construction activity in the world, may be mentioned wholly according to reports:

– physical and math modelling and laboratory researches;

– peculiarities of seismic impact to structures and facilities when different soil behavior;

– soil and structures interaction; – foundation engineering in complex soil conditions;

– offshore geotechnical problems; geotechnics in safety system and objects operational suitability;

– underground construction; shallow foundations;

– sustainable geotechnics; thermal geotechnics;

– uncertaintyes in modern geotechnical codes.

All abovementioned is showing the great potential of the scientific and technical decisions in international practices. This potential should be studied, analyzed and used for development of national science and co9nstruction practices.

Resume of the international conferences shows the reasonability for participation activation of relevant institutes, Universities and industrial organizations participation in the international organizations activity with purpose the more operative survey and using of the world scientific and technical achievements. It concerns to participation of Ukrainian specialists in work of Technical commissions and Work groups and active presentation of the national geotechnical achievements on the international level.

Modern state of information change in Ukraine may be assessed as satisfactory. The thematic conferences and seminars are organizing yearly on the wide spectrum of scientific problems of concrete, reinforced concrete and geotechnics. The specialized journals are publishing by Universities and trade Institutes. The same measures are organizing in other CIS countries. But analysis of advanced international experience should be better.

Ukraine has significant construction problems. We have the some complex engineer-geological conditions for construction; it is territories with sedimentary rocks, above mine working, karsts, underground facilities and catacombs in cities, dangerous landslide in mountains and on sea coasts; up to 20% of state territory is situated in seismic dangerous zones [3]. That fact significantly influences not only on the new construction but on suitability support of the existing old construction objects with significantly out-of-repair resource of the building structures.

In last decades the hydrogeological conditions were changed significantly because of influence of human factor. This fact increases the risks for the objects integrity because of soil conditions change.

Studying and generalizing of the international experience can ensure for us (without significant discharges on structures development, research and test) to reach wide potential for solution of the geotechnical problems existing now in Ukraine.

3 The ninth All-Ukrainian scientific and technical conference «Soil mechanics, geotechnics and foundation engineering: problems, innovations and implementation of Eurocodes in Ukraine»

Conference was organized and conferenced by State enterprise «State research Institute of building constructions», SHEE «Prydnsprovs'ka State Academy of Civil Engineering and Architecture», All-Ukrainian Public Organization «Ukrainian society for soil mechanics, geotechnics and foundation engineering» and journal «Geotechnics world» to meet the plan of Ministry of regional development, construction and communal services of Ukraine [4].

Scientists from Ukraine, Azerbaijan, Byelorussia, Iraq, Kazakhstan, Mauritius, Germany, Poland, France and Czech Republic have participated in conference. General number of the papers, which is included in Interdepartmental scientific and technical collection «Building structures», is 134 ones.

The general and specific construction problems in complex Ukraine engineer-geological conditions, the theoretical and practical aspects of designing in accordance with Eurocodes, the state and prospects for development of Ukrainian normative geotechnical base, the increasing of quality and effectiveness of engineering searching for construction, the progressive structures and modern technologies for foundation engineering and other were considered on conference.

ISSMGEPresident Prof. Roger Frank (Paris, France) and former ISSMGE vice-president Prof. Ivan Vanichek (Prague, Czech Republic) have presented the key reports during plenary session.

Four sessions worked during conference:

– session 1. Geotechnical innovations, implementation of the Eurocodes in Ukraine;

– session 2. Modern problems of soil mechanics, soil science and engineer geology. Classification and methodology for its design parameters determination;

– session 3. Foundations and structures interaction. Design systems «base – foundation – structure» in conditions of natural and artificial bases;

– session4. Construction in complex engineer-geological conditions. Modern technologies for bases and foundations arrangement.

Besides that, the «round table» on the issues of implementation of the European Codes in practices of buildings and facilities designing, construction and operation has been organized with foreign specialist's participation. The ways for elaboration of the practical manuals deal with using of DSTU-EN and for teaching of the bachelors and masters on specialty «Construction and civil engineering» were planned.

Conference has stated the following.

Implementation of the geotechnical researches in the Ukrainian Universities, scientific and design organizations is activated in spite of prolonged recession in construction engineering. It is characterizing as spreading of themes well as using of the new research methods and novelty and practical value of the gotten scientific results. It is perspective for improvement of the structural decisions for both underground and upper part of different structures; for search of the economical schemes, methods of bases preparation and methodology for foundations and buildings of civil, hydro-technical, road, military and other purpose installation, for engineering protection of the existing and new objects, for its retrofitting. There are given proposals for development of the normative base with using of Eurocodes principles. Such position of the conference participants is explaining their social responsibility to state.

At the same time, some reporters were concerned about low level of state bodies care about construction science development and engineer training in accordance with requirements of European educational space.

Conference has noted following moments:

– high actuality of considered issues;

– reasonability of Ukrainian society involving in development of normative base deals with geotechnical design problems on base of international and national experience;

– implementation of Eurocodes in geotechnical design permits to use European experience and advanced technologies. It permits to save the expenditures during buildings construction up to 20-40 % and to increase the foundation durability on weak soils.

On conference opinion the following activity directions deal with scientific problems of geotechnics, soil mechanics and foundation engineering are prospects:

– protection of the buildings and facilities against abnormal deformation of soil bases and complex geological influences, including joint interaction of the bases-foundations-structures;

– development and implementation of the effective methods for soil consolidation and quality soil improvement;

– protensity of the researches and technical decisions development for the buildings and facilities on underworked territories;

– considering trends dealing with territory under-flooding and soil water saturation;

– development of the methods for strengthening of the existing buildings and facilities foundations (including historical and architectural monuments), їits monitoring and abnormal tilts liquidation;

– studying and using of the advanced Ukrainian researches and international experience in geotechnics engineering;

– equipping of the research organizations and industrial enterprises with modern test equipment for ensuring of normative base development.

4 Prospects of UkrSSMGFE development

New conception for Ukrainian society development has been adopted on conference to listen to reports and elect new officials. It has been held on March 2017. There are its basic theses.

1) Activation of the scientific and technical assistance to production:

– conduct of the conferences, seminars and «round tables» with participation of the industrial organizations (reports about complex problems, technical achievements, normative base and implementation of progressive technical decisions);

– participation widening in international measures, including presentation of the national technical achievements;

– generalization and processing of the advanced experience for its using in production activity, normative base and education.

2) Improvement of the organization work in society:

– rejuvenation of society, specification of regional units structure, renovation of Presidium structure;

– widening of the thematic sections in society and ensuring of the horizontal relations between regional units;

– participation on society members in work of ISSMGE technical committees;

– actualization of database on the advanced scientific-technical, innovation and calculation – design decisions on base of the problems arising in manufacturers and designers;

– complex work in society, including the thematic sections and work groups on the contiguous directions of activity (structures, seismic etc.) considering the cooperation with international organizations on these directions;

– holding of the young geotechnical conferences;

– wide information activity (normative base, publications in the international editions and other).

The prospective and annual plans are elaborated by society Presidium for this conception implementation. The regional units are informed about these plans.

Conclusion. The following tasks are actual ones for society and its Presidium.

1. Generalization of the international and national conferences experience and implementation of that experience.

2. Activation of relevant institutes, Universities and industrial organizations participation in the international organizations activity with purpose to monitoring more operatively and use the world scientific and technical achievements.

3. To stimulate the society members to maximal using of advanced experience when creation and renovation of the national normative base in construction engineering.

4. To use that experience in educational sphere.

5. It is reasonably to organize the society members participation in the other international societies – International federation for structural concrete (FIB), International and European associations for earthquake engineering (IAEE andi EAEE) and other ones, where the modern geotechnical problems are under consideration.

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THE HISTORY OF CREATION AND THE ESSENCE OF THE COMBINED SOILS TESTING OF PENETRATION AND ROTATIONAL CUT METHOD

In the early 1960s, the USSR geotechnicians were given the assignment to develop the non-rock soils strength characteristics determining method in order to enable the operative tracked and wheeled vehicles passage estimation through the cross country. As a result of theoretical justification alongside the approbation in lab and field conditions by the group of professionals, there was developed a new method of soils strength investigation by the way of joint penetration and rotational cut testing. A new method of estimating soils strength undergone the overall investigation in different soils environments of the European part of USSR, particularly in Karelia – on stripped clays, center and the south of Ukraine – on sands and loess loams, Moscow region – on heavy clay soils. The prototype equipment allowed conducting the research in the cross country, shallow water areas, on the river beds and bottoms of the lakes. In 1970 the equipment and methodology of joint penetration and rotational cut testing were used during the Moon surface investigation by Lunokhod-16 apparatus.

***Keywords:** soils strength investigation method by the way of joint penetration and rotational cut testing, soils strength, rutting.*

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ІСТОРІЯ СТВОРЕННЯ ТА СУТЬ МЕТОДУ СУМІСНИХ ВИПРОБУВАНЬ ҐРУНТІВ НА ПЕНЕТРАЦІЮ Й ОБЕРТАЛЬНИЙ ЗРІЗ

Розкрито історію створення методу визначення характеристик міцності нескельних ґрунтів на початку 60-х років минулого століття геотехніками СРСР. Унаслідок теоретичного обґрунтування з апробацією в лабораторних та польових умовах групою фахівців було розроблено новий метод дослідження міцності ґрунтів шляхом сумісних випробовувань на пенетрацію й обертальний зріз. Описано процес усебічного дослідження методу оцінювання міцності ґрунтів у різних ґрунтових умовах Європейської частини СРСР, а саме в Карелії – на стрічкових глинах, у центрі та на півдні України – на пісках і лесованих ґрунтах, у Московській області – на важких глинистих ґрунтах тощо. Розглянуто дослідні зразки відповідної техніки, що дозволяли проводити дослідження в умовах бездоріжжя, мілководних акваторій, на дні річок та озер. Доведено, що у 1970 році обладнання і методика сумісних випробовувань на пенетрацію та обертальний зріз були використані при дослідженні поверхні Місяця апаратом Луноxід-16.

***Ключові слова:** сумісні випробовування, ґрунт, пенетрація, обертальний зріз, міцність, колієутворення.*

Introduction. In the 1960s there was a necessity of developing the express method of determining soils strength characteristics in order to estimate the passage possibility of tracked and wheeled vehicles through the cross country. A contractor was the Ministry of Defense of USSR, the task was entrusted by the All-Union Scientific and Research Institute of Hydrogeology and Engineering Geology (AHEG) in Moscow. A Senior Scientific Associate of the AHEG Vadym Fedorovych Razorenov was appointed as a division head for the task.

In 1962 at Poltava engineering construction institute (PECI) there was a post-graduate school opened for the 05.23.02 «Bases and Foundations» specialty. Professor Evgenii Volodymyrovych Platonov was appointed as its supervisor. Platonov and Razorenov once worked together on the ferry crossing construction of Crimea-Caucasia and shared the same scientific interests ever since. According to Razorenov's suggestion, the post-graduates of Poltava ECI were envolved into the development of the problem as crewmen of a geological expedition of AHEG. A fieldwork on the subject started in 1963. The crew consisted of V.F. Razorenov, V.D. Shytov and the chief of geological expedition – T.A. Demcnuk from the AHEG, from Poltava ECI the crew was joined by the post-graduates: V.H. Khilobol, H.V. Zhornik, V.H. Zabara, M.L. Zotsenko, I.N. Skryl. Structural works performed by Alexander Mozhaysky Military Space Academy employees under command of colonel P.I. Eizler [3, 4].

Analysis of recent sources of research and publications. It is known that the passage possibility of tracked and wheeled vehicles through the cross country estimated by the depth of the rut left by vehicles moving along the same route. When the rut depth reaches the vehicle clearance, its movement is terminated due to friction of the vehicle bottom against the soil. Tracks and wheels keep rotating which just deepens the rut [11, 12]. A process of rutting is caused by soils destruction, which is estimated in soil mechanics by the loose of soil strength. According to the Coulomb law, the characteristics of soil strength are internal friction angle φ and specific cohesion c. Thus, for rutting prediction, it is necessary to determine these characteristics for soils on the traffic intervals to the depth of about 1 m [5, 7].

Identification of general problem parts unsolved before. According to state standards of that time, strength characteristics of soils estimated in laboratories after planar shear tests of soil samples. For military purposes, it was necessary to develop a new, field method that would allow prompt soil testing during the scout vehicle motion. That would allow prediction of the rutting process for estimation of passage possibility of tracked and wheeled vehicles through the cross country. In the early 1960s in the USSR and abroad the field methods of soils characteristics investigation via penetration, dynamic and static probing, rotational cut, were actively developing [13, 14, 19, 20]. Using penetration or probing there was determined the average characteristic of soil – penetration resistance (probing resistance) which was a function of internal friction angle and cohesion [15, 16].

The **goal** was, using the rotational cut for clay soils, to determine a specific cohesion only [8, 9].

Basic material and results. Basic material and results. In 1962 V.F. Razorenov [5] suggested the idea of combining the static penetration (probing) and rotational cut in a single device. Using the data obtained from it, it would be possible to determine separately the characteristics of soils strength for conducting the geotechnical calculation including the rutting.

Penetration is a method of soils characteristics investigation by the way of estimating the soil resistance to submerging the bits of different shapes and sizes. There are distinguished the penetration such as when bit penetration depth does not exceed its height and probing – when the depth is greater than a size of a bit. Penetration is used in lab and field environment for shallow soils testing. Probing is used in a process of field soils testing along the depth of its occurrence [17 – 19].

The rotational cut is a method of soil properties investigation by the determination of soil resistance to rotation of winged bit with the four orthodox paddles (Fig. 1, a). It is considered that when the winged paddle submerges on its own height only, the shear surface of soil consists of cylindrical and bottom circular, created by wing rotation.

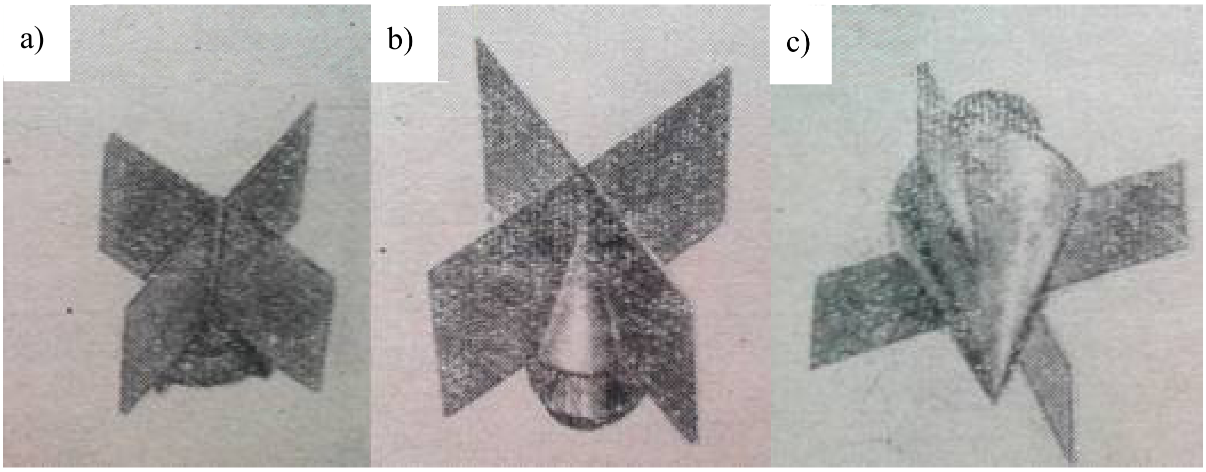

Figure 1 – Combined bits for conducting the joint penetration and rotational cut testing:
a – winged bit; b – bit with the wing along the whole its height;
c – bit with awing in the top part of the cone

If the winged bit is submerged in soil to the depth greater than its height, the top circular surface will contribute to the total shear surface. The limit resistance to rotational cut τ is estimated by the formula:

$$\tau = \frac{M_{max}}{k_\tau}, \text{ kPa}, \tag{1}$$

where M_{max} – maximal external moment, applied to a winged bit; it is determined by the correlation diagram of winged bit rotation angle β and rotational cut resistance τ;

k_τ – static moment of rotational cut surface resistance with a single circular surface is determined by:

$$k_\tau = \frac{\pi D^2}{2}\left(\frac{D}{6} + h\right), \text{ cm}^3, \tag{2}$$

– with two circular surfaces, is determined by the formula

$$k_\tau = \frac{\pi D^2}{2}\left(\frac{D}{3} + h\right), \text{ cm}^3, \tag{3}$$

As a result of numerous experimental studies for combined penetration tests and rotational cut, combined bit of P.I. Eizler design was adopted. It had cone shape with an opening angle of 30^0 with a wing at the top part of it (fig. 1, c) [9 – 10]. The lower part of the conical tip is used for penetration. Then the tip plunged into the ground at full height and a rotational cut was performed. It was this version of combined tests for penetration and rotational cut that was adopted to solve the problems of rutting and later, for lunar soil studies.

For a combined tip with blades in the upper part, the following formula is used

$$k_\tau = \frac{\pi D^2}{2}\left[\frac{1}{6}\left(D - \frac{d_{con}^2}{D^2} d_{con}\right) + h_w\right], \text{ cm}^3, \tag{4}$$

where d_{con} – diameter of cone base, cm;

D, h_w – diameter and height of wing, cm.

When testing clay soils by a rotational cut from the surface, the soils own weight can be neglected, in this case, a specific cohesion c equals to limit rotational cut resistance τ [5]. Then, according to the results of the rotational cut, the specific cohesion of soil c is directly determined. Professor V.G. Berezantsev, based on the theory of limit equilibrium of cohesive soils, [1] established functional dependence of the type

$$c = k_{\varphi} R \, , \tag{5}$$

where k_{φ} – coefficient of proportionality, depending only on the angle of internal friction of the soil φ. V.G. Berezantsev represented this dependence by the graph in Fig. 2.

Thus, determination of the specific adhesion c and the angle of internal friction φ of clay soils in the range of their natural strength is carried out using a combined tip with blades at the base of the cone of the P.I. Eizler's design. in the following order:

– initially, the conical part of the tip is immersed in the ground, while penetration is performed with the determination of penetration specific resistance R;

– then the upper part of the combined tip with the wing is pressed into the ground and a rotational cut is made to determine the specific cohesion c of the soil;

– according to the received data, the coefficient k_{φ} is established, according to which using the graph in Fig. 2, the value of the internal friction angle φ is determined

$$k_{\varphi} = \frac{c}{R} \, , \tag{6}$$

In laboratory conditions, the combined tests of clay soils on penetration and rotational cut method has been introduced into the educational process as a lab practical in studying the course of Soil Mechanics. The device for carrying out this lab practically is shown in Fig. 3.

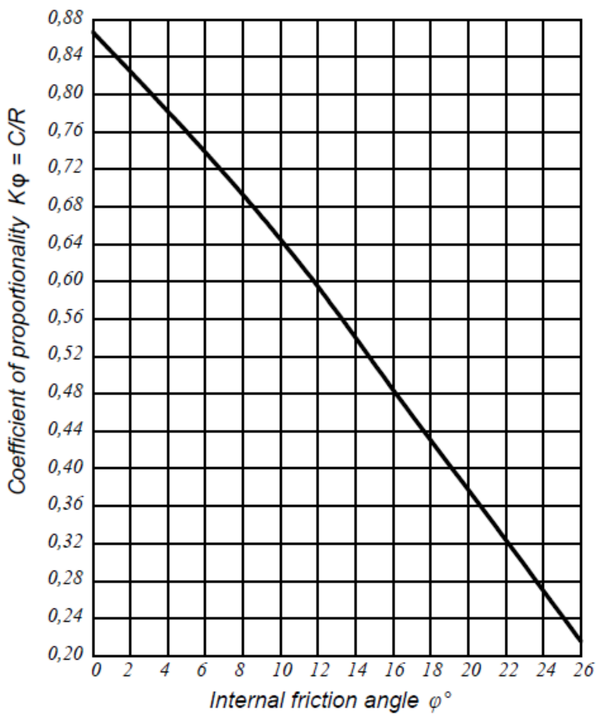

Figure 2 – Graph of the dependence of the coefficient k_{φ} on the angle of internal friction of the soil φ

Figure 3 – «Camomile» device

It is a standard laboratory consistometer equipped with rotary section «Chamomile» attachment of P.I. Eisler's design. A sample of soil is taken into a metal ring with a diameter and height of 70 cm. On one side of the sample, a penetration test is performed with a conical tip with an opening angle α = 30^0 to determine the specific resistance of penetration R. After that, the rod with the cone is removed, the ring with the ground is rotated on the reverse side to perform a rotational cut with a wing-shaped tip (Fig. 1, a).

For this purpose, the ground ring is rigidly attached to the attachment disc. The bar with the wing is rigidly fixed to the bracket to prevent it from turning. The rotational moment on the wing is transmitted through the ground by rotating the disc under the weight of a stepped application of the weights onto the suspension. Experience is considered to be completed when wing makes a turn in the ground. The result wing rotation angle dependence in the ground on the magnitude of the rotational cut torque. The limit resistance to the rotational cut τ is determined by the formula (1).

With soils studied surface, the ultimate resistance of the rotational cut is equal to the specific cohesion. The angle of soil internal friction is determined from the graph in Fig. 2. It should be noted that the combined test method for penetration and a rotational cut is not used for engineering and geological surveys, it is not available in state regulatory documents. Experimental studies of Zhornik G.V. [10]. convincingly proved that with an equal degree of objectivity in establishing the characteristics of soils strength, conducting costs are much lower than for planar shear.

To test the technique of combined tests for penetration and rotational cuts in various soil conditions, mechanical penetrometers were installed on high-throughput machines, three of which are shown in Fig. 4.

a) b) c)

Figure 4 – Machines of various patency
with mechanical penetrometer designed by P.I. Eizler:
a) floating swamp-boat for work in the water area; b) light artillery tractor; c) car GAZ-69

With the help of these machines in the period from 1963 till 1966, the technique of penetration and rotational cut was worked out in various soil conditions of the USSR European part from Karelia to the southern Ukraine. Experimental samples of the appropriate technology allowed to conduct research in the conditions of impassability, shallow water areas, at the bottom of rivers and lakes. Based on the results of these studies, the report was compiled, which was accepted by the customer with a high rating [11, 12].

In 1970, Lunokhod-1 was sent to the moon, a mechanized penetrometer was installed on board, it was equipped with a combined tip with blades at the base of the cone of Eizler P.I. design. To assess the characteristics of lunar soil strength, the technique of combined penetration and rotational shear tests was used. In Fig. 5 Lunokhod-1 scheme with the equipment specified is shown. According to our data, the described method of soil investigation, in the modern modification, is now used on space vehicles of different countries that visit the nearest planets.

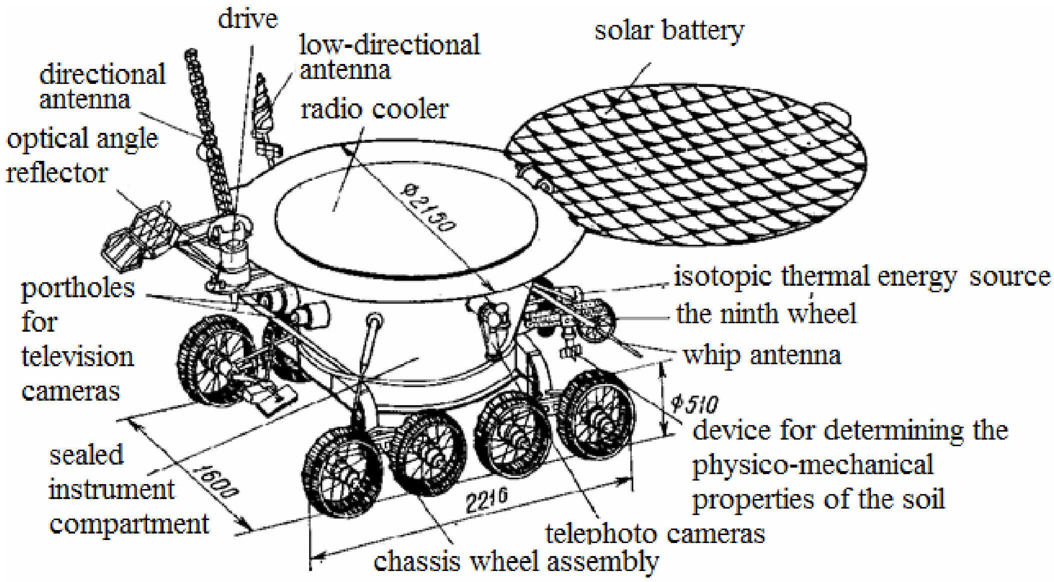

Figure 5 – Operational scheme of Lunokhod 1

Conclusions. The existing experience of using the combined test method for penetration and rotational cut in the process of engineering and geological survey shows its efficiency, economy and sufficient accuracy in determining soils strength characteristics. For obvious reasons, it has not been standardized for a long time and therefore has not received wide circulation.

Today, when reviewing the State Building Standards of Ukraine, namely DSTU B.V.2.1-3-96 (GOST 30416-96) «Soils. Laboratory tests. Terms» it is considered the soils combined tests method should be included in the regulatory framework of the state as an alternative to the planar shear method.

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THE DEVELOPMENT OF THE NATIONAL LEGAL FRAMEWORK ON THE BASES AND FOUNDATIONS FOR BUILDINGS AND STRUCTURES DESIGN

The main document of the regulatory framework for the bases and foundations of buildings and structures design is DBN V.2.1-10-2009 (with Amendments No. 1 and No. 2). For the replacement of existing DBN it is proposed to develop a system of regulatory documents that will include DBN V.2.1-10:201X «Bases and foundations of buildings and structures. Main provisions» and standards for its development. The project provides the principles (general provisions) and requirements regarding the design, construction and reconstruction of bases and foundations for the buildings and structures of all types and classes of consequences (responsibility). The attention is focused on the peculiarities of foundations calculations according to design features and interactions with the base and various depth foundation design.

Keywords: *bases, foundations, buildings, structures, class of consequences (responsibility), design, construction.*

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РОЗВИТОК НАЦІОНАЛЬНОЇ НОРМАТИВНОЇ БАЗИ З ПРОЕКТУВАННЯ ОСНОВ І ФУНДАМЕНТІВ БУДІВЕЛЬ ТА СПОРУД

Основним документом нормативної бази стосовно проектування основ і фундаментів будівель та споруд є ДБН В.2.1-10-2009 (зі Змінами № 1 і № 2). На заміну існуючого ДБН запропоновано розробити систему нормативних документів, що включає ДБН В.2.1-10:201Х «й ряд стандартів у його розвиток. У проекті наведено принципи (загальні положення) і вимоги до проектування, будівництва й реконструкції основ і фундаментів будівель та споруд усіх видів і класів наслідків (відповідальності). Акцентовано увагу на особливості розрахунків фундаментів за конструктивними особливостями й умовами взаємодії з основою, проектування фундаментів різної глибини закладання.

Ключові слова: *основи, фундаменти, будівлі, споруди, клас наслідків (відповідальності), проектування, будівництво.*

Introduction. At present, most of the territories without any significant ground surface gradients, with usual geological conditions and without dangerous geological processes have been built up. It led to reduction of vacant territories with favorable conditions for construction and the need to design and erect new construction sites on areas with difficult engineering and geological conditions and in zones influenced by dangerous geological processes.

These areas are characterized by the significant gradients of ground surface elevations, the possibility of landslide processes activation, the high levels of groundwater standing, the difficult engineering and geological conditions, the presence of soils with specific properties etc.

The National regulatory framework of Ukraine concerning the bases and foundations design for buildings and structures includes a set of codes and standards on the bases and foundations design for buildings and structures in ordinary and difficult geological conditions and for zones with dangerous geological processes influences.

At present, the document DBN V.2.1-10:2009 with Amendments No. 1 and No. 2 is the main component of the Ukrainian codes and standards system concerning the bases and foundations design for buildings and structures in various engineering and geological conditions.

The bases and foundations of buildings and structures shall be designed considering the implementation of Technical Regulation on Construction Products, Buildings and Structures requirements [1].

The Minregion of Ukraine entrusted the Research Institute of Building Constructions to develop the new building standards for this field, which should meet the modern requirements to the development of construction standards and comply with the science development level and technology basis for the construction industry.

The first draft DBN V.2.1-10-201X «Bases and foundations for buildings and structures. Main provisions» was developed to supersede the document DBN V.2.1-10:2009 and its Amendments No. 1 and No. 2 [2 - 4]. For the new standards draft elaboration, a working group was organized. It included leading experts from design, production, research institutions and universities of Ukraine in the field of geotechnics.

Analysis of recent sources and publications. The issues of the bases and foundations design for buildings and structures for the usual and difficult geological conditions, as well as for the zones influenced by dangerous geological processes, the methods of bases strengthening and the constructive solutions for the buildings and structures foundations have been addressed in a number of codes and standards, including DBN V.2.1-10 with its Amendments No. 1 and No. 2, DBN V.1.1-45, DSTU-N B V.1.1-39, DSTU-N B V.1.1-40, DSTU-N B V.1.1-41, DSTU-N B V.1.1-42 and DSTU-N B V.1.1-44 [2-10].

Identification of unsolved issues in the problem under consideration. The existing complex of regulations and standards on designing the bases and foundations of buildings and structures does not provide the full possibilities for designing at the present-day level.

The document DBN V.2.1-10-2009 and its Amendments No. 1 and No. 2 were adopted rather long ago, so it is necessary to revise some of their provisions, which are outmoded and do not comply with the modern practice of bases and foundations designing for buildings and structures.

DBN V.2.1-10-2009 Amendment No. 1 contains the following improvements to the regulatory act text: Subsection 8.5 «Piles and pile foundations» was added; Annex Б was supplemented with terms and Annex Н «Calculation determination of piles bearing capacity based on the ground base properties» and Annex П «Calculation of piles and pile foundations by bases deformations» were added. But the additions given in Amendment No. 1 require refining the terms and calculation requirements and should be transformed into an appropriate standard DSTU-N «Guidelines on the design of piles and pile foundations». The principles of the piles bearing capacity determination based on the static tests data should be revised as well.

DBN V.2.1-10-2009 Amendment No. 2 contains the following improvements to the regulatory act text: items 4.20-4.22 were added to Section 4, items 5.13-5.18 were added to Section 5, Subsection 7.3 item 7.3.3 was set out in a new wording, items 8.3.5-8.3.7 were added to Subsection 8.3, and Annex P «The features of designing a base strengthened with soil cement piles» and Annex C «Stages and sequence of works on the structures erection for the buried and deep foundations and basement-foundation parts» were added. Besides, the text was subject to improvements as follows: 22 changes were made in various sections; 21 changes were introduced into Annex A; 2 changes were introduced into Annex B; 4 changes were introduced into Annex Д and one change was made in Annex Ж. But the additions presented in Amendment No. 2 relate to some improvements to the sections of DBN V.2.1-10 and references replacement in Annex A «List of regulatory documents cited in the Norms». Amendment No. 2 to DBN V.2.1-10-2009 should be converted into two new standards, namely, DSTU-N «Guidelines for shallow foundations design» and DSTU-N «Guidelines for deep foundations design» that should single out barrettes as individual elements of deep foundation structures with an increased bearing capacity, which are arranged using the technology of «ground walls» according to DBN V.1.1-46:2016.

The implementation of new DBN V.2.1-10:201X «Bases and foundations of buildings and structures. Main principles» and standards for its further development will allow updating the complex of regulatory documents on the design of the bases and foundations for civil engineering and production works, including buildings and facilities with basement floors, as well as for underground and buried buildings and structures designed for undeveloped territories and densely built-up areas, normal and complicated engineering geological conditions or the zones subject to hazardous geological processes influences.

Research objectives. The DBN V.2.1-10:201X code is developed to significantly improve the existing national construction standards [2-4] and bring them into conformity with the current needs and state of the legal framework of Ukraine and with international codes in the field of designing the bases and foundations of buildings and structures, as well as to ensure the norms continuity in respect of modern principles of such items designing. The code will be the main component of the system of codes and standards that establish mandatory requirements to the design of bases and foundations of buildings and structures and are intended for the use at all stages of the life cycle of construction objects [5 - 10].

The development of the DBN V.2.1-10:201X «Bases and foundations of buildings and structures. Main principles» first draft is carried out according to the DBN A.1.1-2-93 requirements.

Main material and results. The DBN V.2.1-10:201X first draft provides the general provisions and requirements for the design, construction and reconstruction of the bases and foundations of the buildings and structures of all types and classes of consequences (responsibility) and contains the basic requirements for bases engineering preparation design, the engineering surveys composition, environmental requirements to the buildings and structures bases and foundations design.

This code will be applied for designing the bases and foundations for new objects, as well as in the reconstruction and enhancement of objects in service.

The choice of the foundation types and strengthening means for the bases of buildings and structures should be based on technical and economic comparison of options and on engineering calculations, should consider the urban planning requirements and the requirements as to the environment protection and the rational use of land resources, and should ensure the territories stability and reliable trouble-free work during the estimated service life of the designed objects.

Calculations of bases, as well as of buildings and structures should be performed for the first (strength) and second (deformation) groups of limit states at the time of construction and operation of objects.

The DBN V.2.1-10:201X code was developed with the use of a parametric method, which is based on the goals and objectives hierarchy and widespread in the European Union countries and many others. The parameters include goals, bases and foundations functional requirements and criteria that the buildings and structures should meet. Parametric norms define the normative act purpose, and declare in the general form the basic principles (provisions) for achieving this purpose. They do not regulate the specific calculation methods, technologies, materials or products; therefore the various aspects of their application are not described.

The parametric method used in DBN V.2.1-10:201X envisages the determination of the bases and foundations parameters that ensure the safety, functionality and reliability of buildings and structures during operation. The national standards provide the basic rules ensuring the implementation of the basic principles (provisions) specified in the State Building Regulations.

The structure of the system of codes and standards on the design of bases and foundations for buildings and structures for the ordinary geological conditions of Ukraine is shown in the Fig. 1.

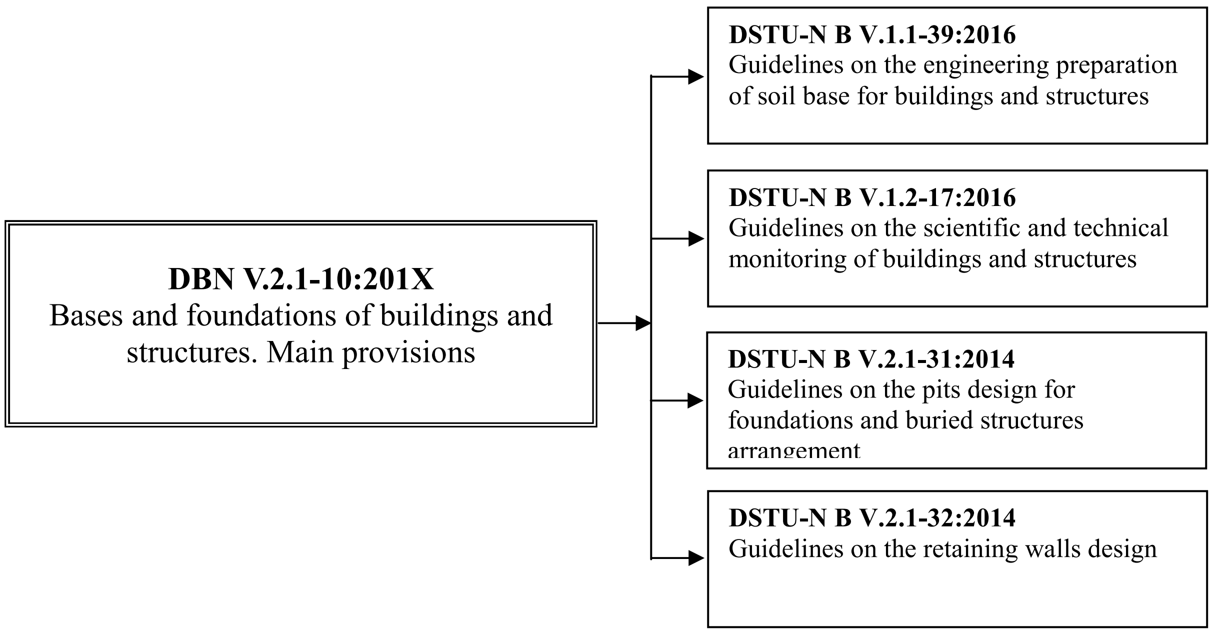

Figure 1 – The structure of codes and standards system of buildings and facilities bases and foundations design in the ordinary geological conditions

The structure of the system of codes and standards on the bases and foundations design in difficult geological conditions is shown in Fig. 2.

In Ukraine the areas of territories subject to dangerous geological processes and quantities of landslides and transitions of previously stable slopes (declivities) into the category of landslide hazard slopes permanently increase, resulting in the increase of geological risks of those territories development and emergency situations occurrences.

The structure of codes and standards system on the bases and foundations design for buildings and structures in zones influenced by dangerous geological processes is shown in Fig. 3.

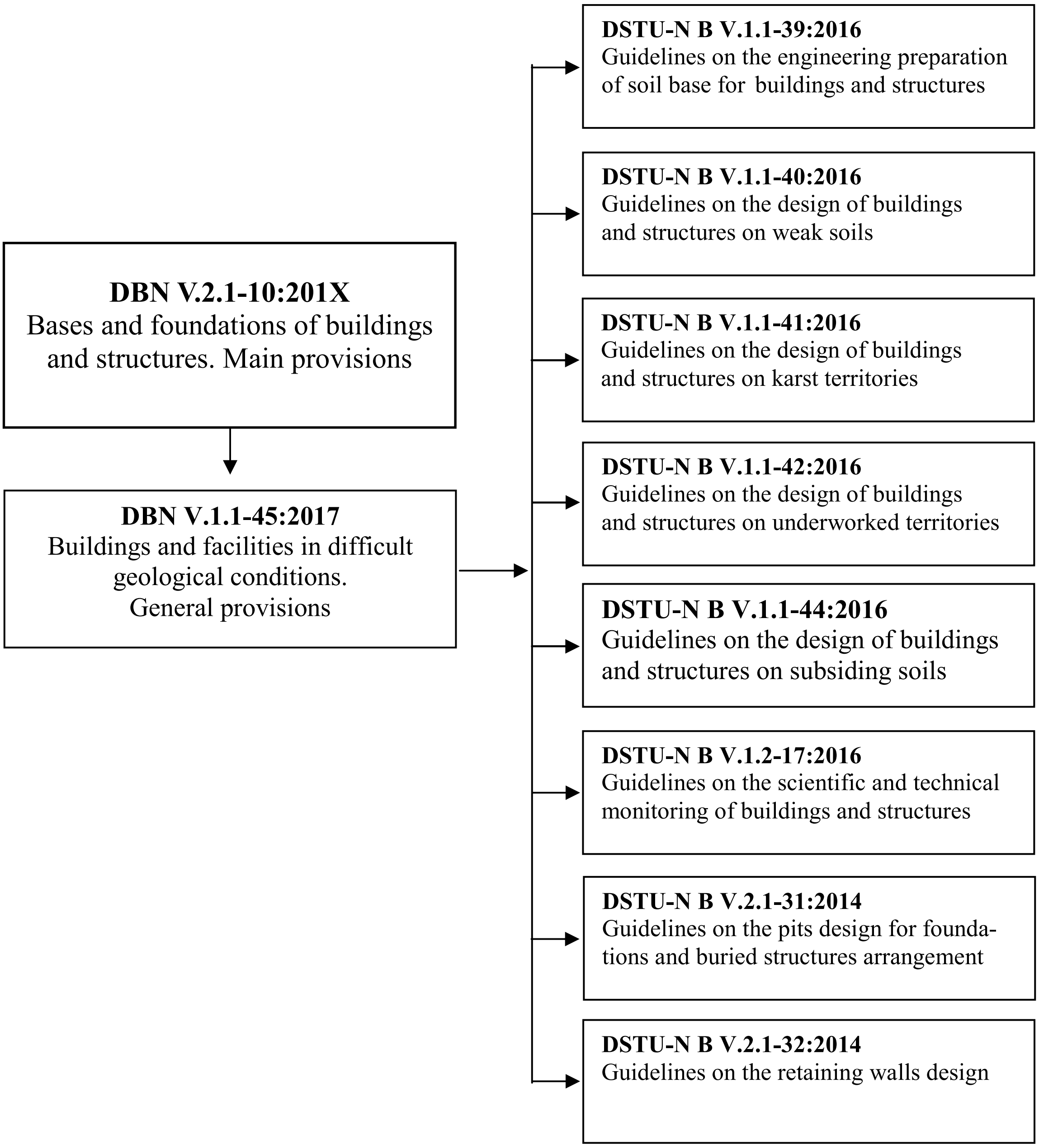

Figure 2 – The structure of codes and standards system on the design of bases and foundations for buildings and structures in the difficult geological conditions

In Fig. 4 the proposals for the DBN V.2.1-10:201X provisions development by means of the elaboration of additional standards on the foundations design for the buildings and structures of various purposes are shown.

In the DBN V.2.1-10:201X draft it is stated that the bases and foundations design for buildings and structures, and the choice of the foundation type/design and the method for bases preparation (if necessary) should be carried out taking into account the following data:

– the results of engineering surveys for construction in compliance with DBN A.2.1-1-2008. For structures with considerable consequences (CC3) the presence of geopathic zones should be taken into account;

– the data characterizing the purpose, as well as the structural and technological features of a building, loads acting on the foundation and the conditions of their operation;

– the feasibility study of alternate technical solutions for the basement-foundation section.

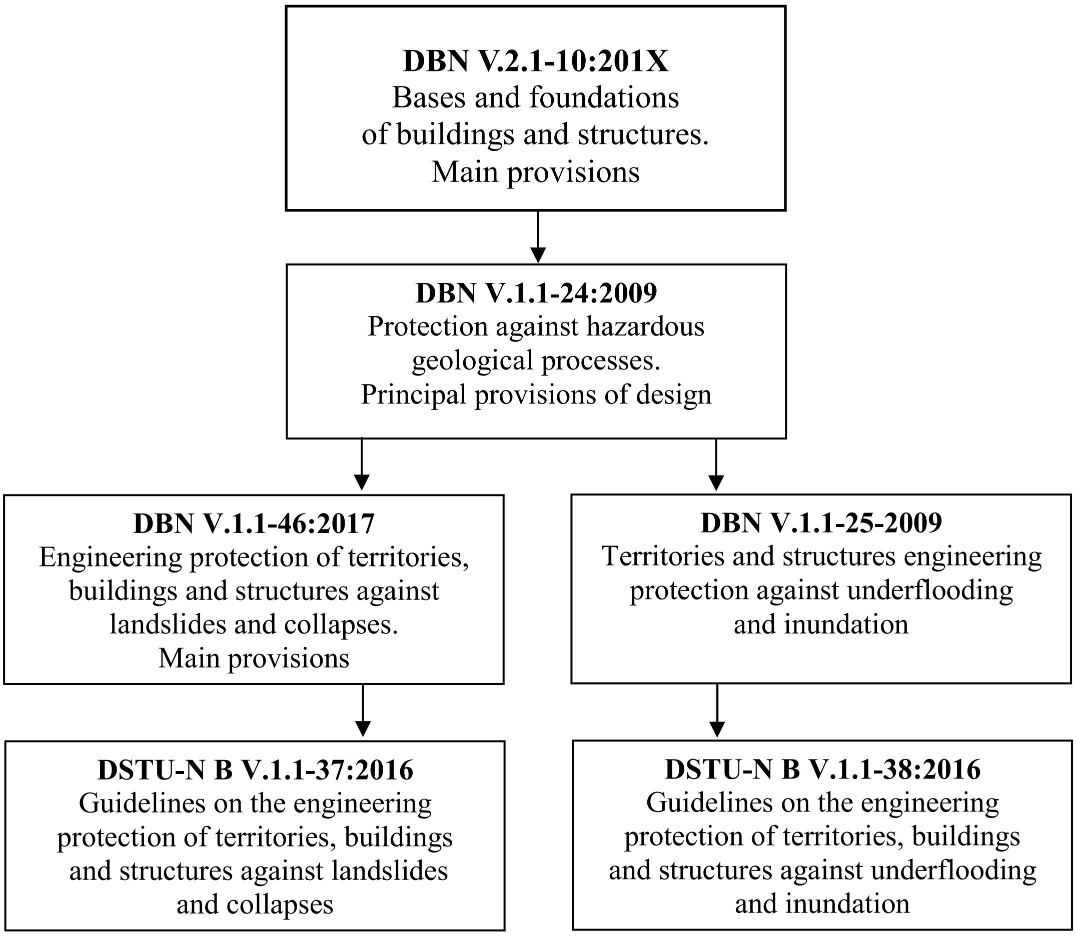

Figure 3 – The structure of codes and standards system on the bases and foundations design for buildings and structures in zones with dangerous geological processes influences

The design should ensure the most comprehensive use of the bearing capacities and deformability of base soils, as well as of the physical and mechanical properties of foundations and substructures materials.

All types of bases and foundations should meet the following requirements concerning:

– safety;

– serviceability;

– life duration (excepting the special cases specified for temporary buildings); and

– some additional requirements defined by the terms of reference on the buildings and structures design.

The bases and foundations should be designed with ensuring the implementation of the above mentioned requirements by the assignment of the following aspects: characteristics of the foundation materials and base soils; coefficients of reliability; loads and actions types; calculation schemes corresponding to the actual behavior of bases and foundations at the various stages of building construction and operation; structural, technological and operational requirements; and values of limit deformations (deflections, maximum and non-uniform settlements and rolls).

If the base design deformation under difficult engineering geological conditions exceeds the limit values or if the base bearing capacity is insufficient, the measures should be foreseen for the deformations negative influence mitigation.

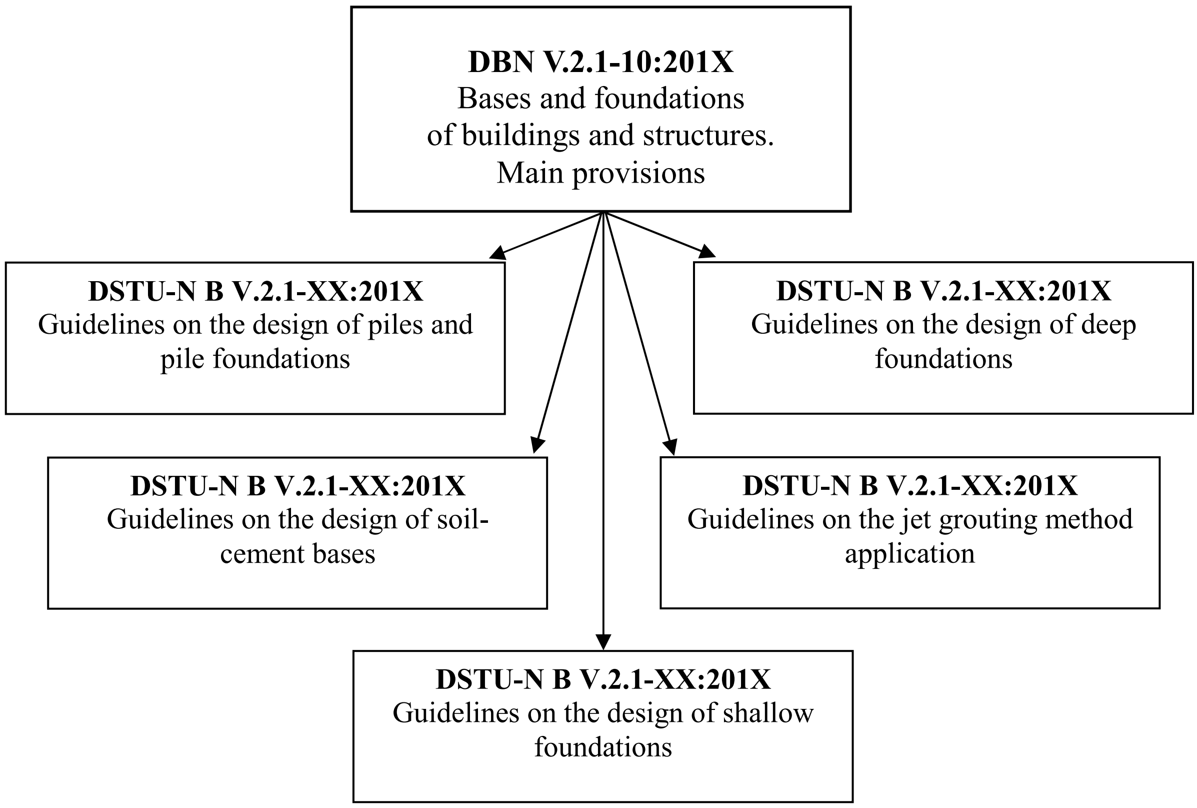

Figure 4 – The DBN V.2.1-10:201X provisions development proposals concerning the foundations design for the various purpose buildings and structures

When designing bases and foundations, it is necessary to envisage the cutting of fertile soil layer with the later using it for the regeneration (reclaiming) of disrupted or inefficient agricultural lands or for the building area planting with trees.

The bases and foundations should be designed based on the initial data necessary for the selection of foundations type, structure, depth and dimensions; the natural engineering preparation of a base or the arrangement of an artificial base; forecasting of buildings and structures bases and foundation deformations over time, making the decisions as to the environment preservation (protection); and the development of engineering measures for territory protection against hazardous geological processes.

The bases and foundations designing should include the computationally substantiated selection of the following data with taking into account the complexity category of geological conditions (DBN A.2.1-1-2008) and buildings and structures consequences class (DBN V.1.2-14-2009):

– the type of base, structures, materials and depths of foundations;

– engineering measures for the reduction of bases deformations influence on buildings operational qualities and the environmental protection ensuring.

The foundations and basement-foundation parts should be calculated as a building part according to the properties of base soil (natural or artificial) and its structures materials. The foundations analysis by the base soil properties should be carried out for two groups of limit states as follows:

a) the first group concerns the bearing capacity (strength and stability);

b) the second group concerns such deformations as settlements, rolls and horizontal displacements, and here the parameters of contact surface deformations in cases of their predicted occurrence in the difficult engineering geological conditions should be taken into account.

The foundations analyses for the first group of limit states are carried out in the cases, when the vertical and/or horizontal loads, including seismic or dynamic ones, act on the building; a building is located near a declivity or on a slope; a base is composed of rock or poor-bearing soils with specific properties or steeply dipping layers; a foundation works for pull-out, as well as in all circumstances, when calculations by base deformations are performed during a non-linear stage.

The foundations analyses for the second group of limit states are carried out in all cases based on the condition of the building or facility combined action with a base. The fundamental principles of bases and foundations calculations having a long history of their evolution and numerous practical applications are preserved in the new standards.

The calculations of foundations according to their structures materials should be carried out with respect to the actions of static and (or) dynamic loads from the structures resting on them; influences of the base non-uniform deformations; in the cases of their predicted occurrence in the difficult geological conditions and under the dynamic or seismic actions transmitted by the base, for the limit states of:

a) the first group, when the foundation materials strength is considered according to the requirements of concrete, reinforced concrete and masonry structures designing; and

b) the second group, when the deformations non-uniformity and cracks initiation or opening in reinforced concrete foundations are considered according to the reinforced concrete structures design requirements.

The buildings and structures calculations by base deformations should be carried out depending on the condition of their combined action. Calculations by bases deformations are carried out from the following condition

$$S \leq S_u , \tag{1}$$

where S is a common deformation of the base and building determined by the calculation, mm;

S_u is a limit value of base and building common deformation, mm.

The foundations calculation by the bases bearing capacity is carried out to ensure their bases strength and durability, as well as to prevent displacements along a bottom or foundation overturning. The base failure scheme to be accepted at attaining the base limit state should be admissible in static and kinematic aspects for this influence and this design of a foundation or building.

The foundations calculation by the bases bearing capacity is carried out based on the following conditions

$$\sigma \leq \sigma_u \text{ (in general) or } F \leq \gamma_c F_u / \gamma_n , \tag{2}$$

where $\sigma = F/bl$, Pa;

σ_u is a stress corresponding to the base bearing capacity limit, Pa;

F is a design load on the base, N;

F_u is a force of base limit strength; the vertical component of the force of the base limit strength, N_u, N;

γ_c is a service conditions factor, which is accepted as follows for: sands, excepting dust sands, as $\gamma_c = 1.0$; dust sands and clayey soils in a stabilized state as $\gamma_c = 0.9$; clayey soils in a non-stabilized state as $\gamma_c = 0.85$; and rocky soils, including unweathered rocks and low-weathered rocks, as $\gamma_c = 1.0$; weathered rocks as $\gamma_c = 0.9$ and strongly weathered rocks as $\gamma_c = 0.8$;

γ_n is a safety factor by responsibility (importance coefficient) determined depending on the consequences class of an object in compliance with ДБН В.1.2-14-2009;

b and l are the dimensions of foundation faces in a plan view (width and length), m.

The limit strength force of a base composed of soft soils in a stabilized state should be determined in view of the requirement that the relation between normal and tangential stresses along all sliding surfaces, which corresponds to the base limit state, obeys the following dependence:

$$\tau = \sigma\, tg\, \varphi_{\mathrm{I}} + c_{\mathrm{I}}, \tag{3}$$

where φ_{I} and c_{I} are the design values of an angle of internal friction and soil unit cohesion, respectively.

It is allowed, when appropriate, to calculate the bases stability using the graphical analytic methods (with circular cylindrical or broken sliding surfaces), if:

a) base is irregular in depth and area;

b) surcharging of the base from the different sides of the foundation is different and the intensity of the larger of them exceeds $0.5R$;

c) building is located on a slope or near a declivity;

d) non-stabilized state of the base soils can occur.

In all cases, when the foundation is affected by horizontal loads, and the base is composed of soils in a non-stabilized state, the foundation should be calculated for shear along a bottom.

The foundation calculation for shear along the bottom is determined by the following condition

$$\Sigma F_{s,a} \le (\gamma_c \Sigma F_{s,r})/\gamma_n, \tag{4}$$

where $\Sigma F_{s,a}$ and $\Sigma F_{s,r}$ are sums of the projections on the sliding plane of design forces that displace and hold, respectively, which are determined considering active and passive pressures of the soil on the lateral faces of the foundation.

If bearing capacity of natural soil bases is insufficient, it is necessary to perform their engineering preparation by improving their properties at the place of their occurrence to the required level or by their strengthening with arranged in them bearing or draining structural members of soil and other materials in compliance with DSTU-N B V.1.1-39 [5].

To improve the properties of soils in place of their occurrence it is possible to apply their mechanical (surface and deep) or physical compaction, as well as mechanical or chemical stabilization.

To strengthen the soil bases it is possible to apply the following engineering measures:

– replacement of poor-bearing layers with soils having higher mechanical characteristics;

– draining of water-saturated soils by means of drains from natural and/or artificial materials;

– bases squeezing by temporary embankments, including those with the arranged drains;

– mixing of poor-bearing soils with cement or other binder solutions;

– reinforcement of soil bodies with (stiff and/or having constrained stiffness) structural members;

– reduction of the bases lateral expansion under loading by means of bases enclosing with permanent sheet-pile or pile walls.

For the conservation of buildings durability and the elimination of the accelerated wear of reinforced concrete structures in watered environment the water protection designing for bases, underground engineering structures, buried structures, underground-foundation parts and foundations is performed. Requirements for water protection should be developed taking into account the following influences of water:

– the temporary ones due to atmospheric precipitation infiltration, underflooding by high waters or accidents at water conduits, and

– the permanent ones due to the presence of soil moisture or ground waters.

To prevent the penetration of ground water in buildings, structures and construction sites protected from flooding, it is possible to use the grout curtains arranged by the injection method or the method of trench walls. They are the most efficient when they reach watertight soils or soils of low permeability with the factor of permeability not exceeding 2x10-2 m/day.

The complex of activities on the protection of the bases and foundations of cultural heritage monuments should be executed within the framework of a special program. The geotechnical research program may be an independent document or a part of a comprehensive program of scientific and restoration research of the monument, which is elaborated for the development of monument conservation projects and the implementation of urgent works on its preservation. The program of the integrated investigations of the technical condition, as well as a plan of measures for the protection of the bases and foundations of landmarked buildings should be agreed upon by the executive authority responsible for the cultural heritage protection.

During designing the bases, foundations and underground or buried structures the engineering surveys should be carried out in accordance with DBN A.2.1-1-2008 in order to assess the engineering and environmental conditions on the territory (site) of construction (reconstruction) and forecast their possible changes.

The composition and scope of engineering and environmental surveys should be sufficient for obtaining information necessary for drawing the conclusion about the territory environmental safety during the construction. The deterioration of the environmental situation, which must be taken into account during the design, may be caused by the changes in the conditions of building development, hydrogeological processes and the technical solutions of construction objects.

The design must contain the engineering solutions necessary for the conservation, protection or improvement of the ecological situation on the site of construction and adjoining territory.

The design should envisage the measures for the prevention or protection of the construction zone and object against the negative impacts of:

– contaminated soil layers - clearing or removing and transfer to agreed burial sites;

– toxic gases (radon) - creation of delayer barriers (screens) and arrangement of ventilated crawl spaces for removing gas and preventing its transfer to living premises,

– contaminated soil and surface waters - construction of dams, grout curtains, water protection walls, sediment basins etc.

If necessary, the following measures should be envisaged: karst control, landslide control, water proofing, protection against dynamic actions and toxic substances, compliance with environmental safety during the construction at waste disposal sites and on man-made waste; and the solution of issues of contaminated soil dumps and preservation of the fertile layer and green plantations.

Scientific and technical support is necessary for complex objects of construction or re-construction (with basements exceeding one floor, located in conditions of high density areas, having a specific structural scheme, high-rise ones, potentially dangerous, unique, religious and monuments); for specific geological, hydro-geological, ecological conditions and complex terrain; and for structures in the zones of new construction (reconstruction) influences (risks) or in areas where dangerous geological processes are possible.

Monitoring is a component of scientific and technical support. It is carried out at the stage of designing and construction, as well as during reconstruction and conservation of the buildings of significant consequences (CC3) – in all cases and of the buildings of CC2 – in difficult geological conditions, on densely built-up areas and in zones influenced by new construction or rebuilding.

Monitoring at the stage of construction and operation according to the functional profile of a building should include the visual-instrumental field observations and surveys (including geodetic control) of structures, bases, territories, as well as the hydrogeological and ecological observing system, and analysis of the results.

Conclusions. Thus, the elaboration and subsequent use of DBN V.2.1-10:201X «Bases and foundations of buildings and structures. Main provisions» will allow more reasonable carrying out the design of bases and foundations for buildings and facilities, increasing the reliability and safety of objects due to the application of new standards, and bringing the practice of designing the bases and foundations of buildings and structures in accordance with modern requirements.

The developers of the new national building codes invite Ukrainian specialists in geotechnics to take part in the further improvement of the DBN draft and will be grateful for careful considerations and proposals.

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USING OF DYNAMIC AND STATIC LOAD PILING TESTS IN ASTANA, KAZAKHSTAN

In this paper the analysis results of precast piles different tests are presented. Extreme soil conditions of Astana (Kazakhstan) involve realizing the work precast piles in various soil ground and interaction soil ground and piles. There were carried out dynamic and static load tests of piles in extreme soil ground conditions in Astana. Based on data results of pile foundations the piles bearing capacity was determined. According to the results of DLT with PDA of driving piles (30.0 cm) the bearing capacity of the piles is 911 kN. The bearing capacity of the driven piles according to the results of SLT amounted to be 878 kN. Soils physic-mechanical properties in extreme conditions of Astana along with graphs of dependence are between settlement and load. The precise analysis of climatic and geological factors of the construction sites is shown. Investigations method for precast concrete piles testing is presented. Dynamic load test methodology in Astana for concrete piles testing is shown. These investigations are important for of Pile-Soil interaction on problematical soil ground.

***Keywords:** pile foundations, load test, bearing capacity, conditions.*

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ВИКОРИСТАННЯ ДАНИХ ВИПРОБУВАНЬ ПАЛЬ СТАТИЧНИМИ ТА ДИНАМІЧНИМИ НАВАНТАЖЕННЯМИ В АСТАНІ, КАЗАХСТАН

Наведено результати аналізу різних випробувань збірних паль. Оцінено роботу збірних паль у надзвичайних ґрунтових умовах Астани (Казахстан) та їх взаємодію з оточуючим ґрунтом. Представлено дані статичних та динамічних випробувань паль у надзвичайно складних ґрунтових умовах. Визначено несучу здатність паль з використанням отриманих результатів. Відповідно до результатів випробувань паль (30,0 см), несуча здатність склала 911 кН, несуча здатність рухомих паль залежно від результатів ТЗ становила 878 кН. Представлене дослідження має важливе значення для розуміння взаємодії системи паля – ґрунт з проблематичним ґрунтом.

***Ключові слова:** пальовий фундамент, випробування навантаженням, несуча здатність, геологічні умови.*

Introduction. Unique buildings and structures are built in the new capital of Astana. There are Trade & Entertaining Centre «Mega Plaza Astana» and scientific and educational complex «Nazarbayev University». The design of foundations requires careful analysis of natural climatic, geological factors: air temperature, winter duration, snow cover thickness, soils thermal properties.

Recent sources of research and publications analysis. A. Zh. Zhussupbekov, M. K. Syrlybaev, R. E. Lukpanov and A. R. Omarov [3 – 5] have investigated dynamic and static piling tests applications of Astana, A. Zh. Zhussupbekov, R. Zh. Lukpanov, A. L. Omarov have investigated the Results of Dynamic (Pile Driving Analysis) and Traditional Static Piling Tests in Capital of Kazakhstan.

Identification of general problem parts unsolved before. The design of foundations requires a careful analysis of natural climatic, geological factors such as air temperature, winter duration, snow cover thickness, thermal soils properties not analyzed before in this region.

The **goal** of the research is to analyze the interaction of extreme soil ground conditions with precast piles under various loads.

Basic material and results. Engineering-geological conditions of the site are: construction site of the Trade & Entertaining Centre «Mega Plaza Astana»; the site of researches for construction is on the Northeast side of Kazakhstan capital Astana, on the left side of the Esil river between two streets of Kabanbai batyr avenue and Ryskulov street in Astana (Kazakhstan).

Figure 1 – Construction sites in Astana (Kazakhstan)

Pile tests were performed in accordance with the requirements of GOST 5686-94 «Soils. Methods of field testing with piles».

Results of soil construction site physic-mechanical properties are resulted from Table 1.

Table 1 – Physic-mechanical properties of site soil

Type of soil	E, MPa	C, kPa	φ, deg	ρ g/cm^3
Poured soils	-	-	-	1,87
Muddy loams	4,0	14	10	1,78
Loams	7,0	12	20	2,04
Sands of average size	17,0	1	35	1,92
Semi-gravel sands	21,0	-	38	1,92
Gravel soils	21 – 23,0	-	-	2,00
Clayey soils	12,0	-	28	1,95
Gruss - rock debris soils	23 – 29,0	-	-	2,12-2,20
Sands stones	8,0	-	-	2,57

Construction site of Scientific & Educational complex «Nazarbayev University». Astana, Kazakhstan: the construction site is on the Northeast side of Astana, on the left side of the Esil river (Table 2).

Table 2 – Physic-mechanical properties of site soil

Type of soil	E, MPa	C, kPa	φ, deg	ρ g/cm^3
Poured soil	-	-	-	1,87
Loams	13	21	31	1,97
Sands of average size	17	2	35	1,47
Gravel sands	21	1	38	1,61
Gravel soils	23	-	-	2,00
Loams	16	35	33	2,00
Silty medium gravel	29	-	-	2,20
Sands stones	32	2	-	2,40

Precast concrete pile: static tests were carried out on precast concrete pile with a total length of 6.2 m. with cross-section 30×30 cm (Fig. 2). Pile is applied by bituminous (corrosion protection) material and marked every 0.1 m. The purpose of testing is estimation of the bearing capacity and comparison of the results pile foundations. For driving piles used driving rig Junttan PM-20 with hydraulic hammer NNK-5A part of blow weight of 5.0 tons. Field works are made in accordance with the requirements [1, 2].

Investigation method for precast concrete pile: before driving test piles have been marked every of 10 cm on all length (L = 6.2 m). During the first stage of dynamic tests carrying out the loads were made by C10-30 (28 piles). Piles immersed in the soil to the absolute mark of the pile point 337.24 m.

Figure 2 – Precast concrete pile on the site

In the first stage, piles were driving according to preliminary criteria for a stop: on 911 kN working load, pile refusal should be equal 0.83 cm; on 620 kN working loading, pile refusal should be equal 0.56 cm. The results of dynamic tests is given in Tab. 3.

Table 3 – Results of dynamic tests

Number of pile, across	Embedded depth, m	Design load, kN	Refusal of pile at driving, cm
6C10-30, 30×30	8.2	620	0.56
9C10-30, 30×30	9.1	878	0.59
2C10-30, 30×30	8.4	911	0.83

Dynamic load test (DLT): in Kazakhstan experimental piles with dynamic loads production was carried out on October 27, 2016, using a Junttan PM-20 pile-rig with an NNK-5A hydraulic hammer, with a 5,000 kg impactor mass and a weight of 835 kg. Re-driving of the test piles was carried out by three and five hammer blows. To determine the bearing capacity, the largest average failures were obtained when piles were pierced after their «rest». The strain gauges with the length of 10 cm were attached on piles top before starting re-driving. Strain gauges had been fixed on 60 cm from the pile head. Allowable piles bearing capacity with safety factor (FS = 1.4) equal to 620 kN [2-5]. The piling of the tested piles was carried out on October 20 and 21, 2016, temperature -5ºC, using a Junttan PM-20 pile-driving machine with a hydraulic hammer, NNK-5A, with a shock weight of 5000 kg and a headgear weighing 835 kg. When driving the piles, wooden pads, 10 cm thick, were installed on the metal cushion for damping hammer impact forces on the reinforced concrete pile head. Before the start of piling the test pile on its surface, preliminary, the paint was divided over the entire pile length 1 m, and on the last meter, the proposed depth of immersion, after every 0.1 m. In the process of pile driving, counting the number of hammer blows for each pile dive meter was counted and on the last meter for every 10 cm of driving. At the same time, the impact hammer heigh was recorded. The results of piles re-drive is given in Tab. 4.

Table 4 – Results of piles re-drive

Number of pile	Refusal of pile at re-driving, cm	Particular value of the limit resistance of piles, kN
6C10-30, 30×30	0,57	465
9C10-30, 30×30	0,3	658
2C10-30, 30×30	0,28	685

Static tests carrying out: driving test piles was carried out in 6.2 m. The technology of precast concrete piles static loading test was done in according with the requirements of GOST 5686-94, i.e. according to the GOST requirements precast pile is tested after 6 days «rest». Testing platform presented system from steel, which consisted of metallic beam. For platforms applied concrete blocks, total weight was 185 tons. The results of the first stage static tests is given in Tab. 5.

Table 5 – The first stage static tests results

Number of steps	Design load, kN	Settlement, Mm	Steps duting stability
Load			
1	106	0,26	1
2	212	0,97	1,5
3	318	1,93	2
4	398	2,74	2
5	478	3,66	2
6	558	4,59	2
7	638	5,61	2
8	718	6,70	2
9	798	7,73	2
10	878	8,95	3
Unload			
1	718	8,88	15 min
2	558	8,31	15 min
3	398	7,55	15 min
4	212	6,04	15 min
5	212		1

The vertical load was created with hydraulic jack CLS 2006. The load was recorded with pressure-measuring manometer H4049L from 0 to 1000 bar. Measurements of each pile movements were made by two deflection gauge 6PAO, which have been marked every of 0.01 mm. Safety factor (SF=1.2) [2]. The graph of dependence of Settlement S from Load P is shawn in Tab. 3.

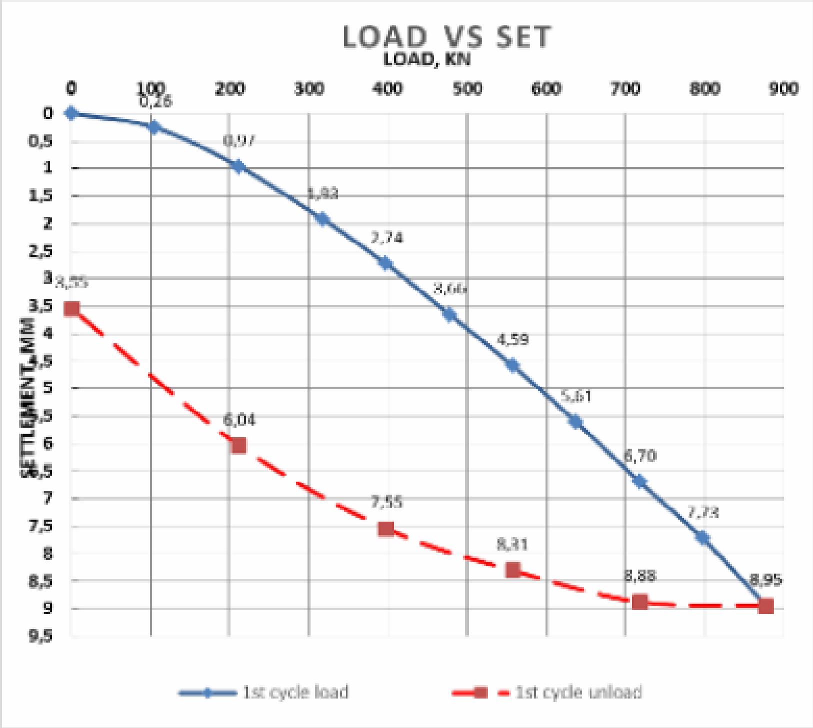

Figure 3 – Graph of dependence of Settlement S from Load P (by GOST 5686-94)

Conclusions. The testing aim is to determinate pile foundations bearing capacity in extreme ground soils of Astana, Kazakhstan. According to the results of DLT with driving piles PDA (30.0 cm), the piles bearing capacity is 911 kN. The driven piles bearing capacity according to the results of SLT is 878 kN. These investigations are important for Pile-Soil interaction comprehension on problematical soil ground.

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MODEL TESTING OF PILED CLUSTERS AND LARGE MONO-PILES OF IMPROVED DESIGN

When constructing piled clusters and structures supported by large mono-piles, piles designed are used to take up significant lateral and pressing-in loads. New effective and less resource-demanding design of piled cluster was considered before. At this paper some results of its model testing in laboratory conditions are analyzed and discussed. To increase energy-absorbing capacity of mooring/fender dolphins it was worked out and researched a new design of combined tubular mono-pile structure, incorporating internal flexible pile and damping element placed at the zone of pile head. This design has been tested by laboratory experiments using small scale model. Obtained results confirm its effectiveness and practicability.

Keywords: *piled cluster, tubular mono-pile, bearing capacity, energy-absorbing capacity.*

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МОДЕЛЬНІ ВИПРОБУВАННЯ ПАЛЬОВИХ КУЩІВ ТА ВЕЛИКИХ МОНОПАЛЬ УДОСКОНАЛЕНОЇ КОНСТРУКЦІЇ

Доведено, що при будівництві пальових кущів та споруд, що оперті на великі монопалі, використовують палі, запроектовані на великі бокові та вдавлюючі навантаження. Презентовано аналіз модельних випробувань у лабораторних умовах нової ефективної та економічної конструкції пальового куща. Для збільшення енергоабсорбційної здатності швартовно-відбійних палів розроблено та досліджено нову конструкцію комбінованої трубчастої монопалі, яка містить внутрішню гнучку палю й амортизатор, розміщений у зоні голови палі. Це рішення було перевірено лабораторними експериментами на моделі малого масштабу. Отриманими результатами підтверджено його ефективність та практичність.

Ключові слова: *пальовий кущ, трубчата монопаля, несуча здатність, енерго-абсорбируюча здатність.*

Introduction. When constructing piled clusters and structures supported by large mono-piles, piles are used to take up significant lateral and pressing-in loads [1].

Analysis of recent researches and publications. In particular it relates to deep-water sea structures piled foundations which need long piled supports of high bearing capacity [2 – 5]. It corresponds to one meets high level of stresses and significant deformations in such constructions. Some improved structures and technologies have been worked out to optimize stress-strain state of piled clusters and mooring/fender dolphin [6].

Identification of general problem parts unsolved before. To study peculiarities of these two innovative design solutions model testing in laboratory conditions have been produced in Odessa National Maritime University (Department «Sea, River Ports and Waterways»). Physical modeling was provided at two stages. The first one (preliminary experimental research) was simplified by testing of the model without soil media (instead of piles embedded into the soil piles' tips were fixed by special clamps (console scheme).

The aim of this first (simplified) stage was to determine the most appropriate calculation model and related software to be applied later at the second stage of experiment. The second stage was planned to test the same models in the soil box without artificial fixing of piles tips (piles were embedded into sandy soil).

In this paper we consider and discuss results of the first testing stage.

Basic material and results. 1) Model testing of pile cluster with shelled element. Developed is effective and less resource-demanding design when connection of all piles with large diameter casing provides their joint work and favorable distribution of stresses and deformations in pile cluster (Figure 1). Large diameter casing is installed both above and below sea bottom level relieving piles and then in decreasing stresses. Connection between casing and piles may be provided with similar sheet piles interlocks.

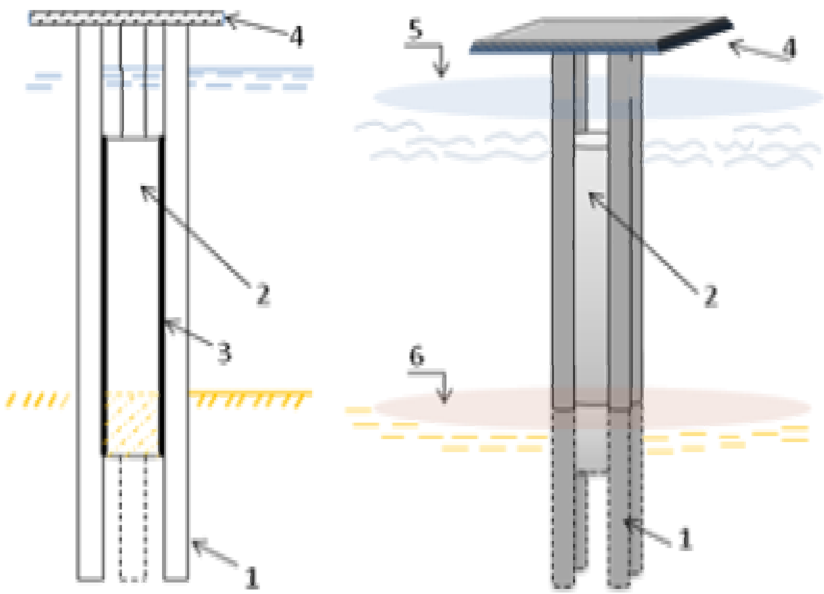
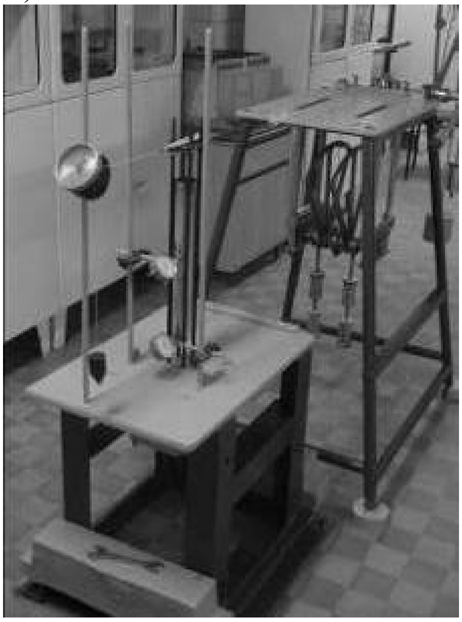


Figure 1 – Piled cluster of innovative design: a – cross-section; b – 3D view: 1 – bearing piles; 2 – steel cylindrical casing; 3 – interlock connections; 4 – superstructure; 5 – water level; 6 – bottom level

Such structure has been tested on 3-D physical model (scale appr. 1:100). Pile supports are fixed by special clamps to exclude their tips' displacements. Total length of the piles was preliminary determined according to the known recommendations of actual design codes (Figure 2).

a)



b)

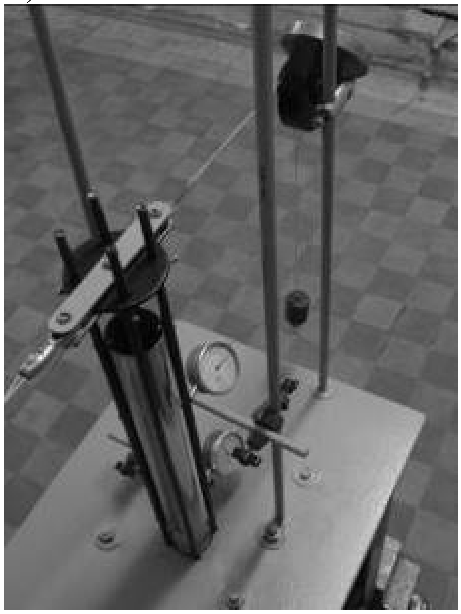


Figure 2 – Model testing of piled cluster model in laboratory
a – general view of the model; b – measuring and loading systems

The same model was analyzed by 3-D numerical simulation (FEM) using different programs in order to determine the most proper calculation model and software regarding peculiarities of the model and interaction between its elements. Three programs were applied to determine stress-strain state of the model: AxisVM, Lira-SAPR and Midas-Gen (calculation scheme is presented on the Figure 3).

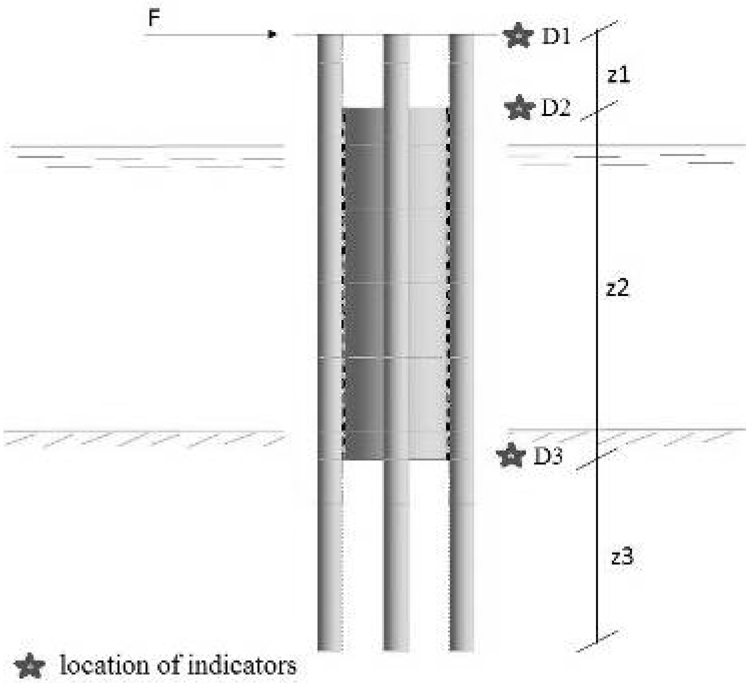


Figure 3 – Calculation scheme
(piles' tips are fixed, D1, D2 and D3 – locations of displacement indicators;
F – applied lateral force)

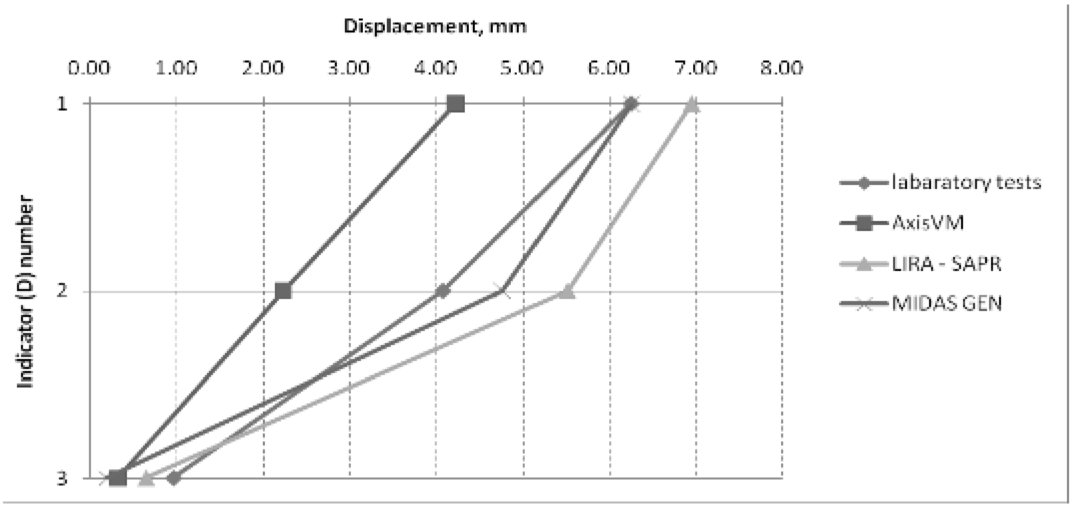


Figure 4 – Displacement diagrams obtained in experiment and calculated using different software (external lateral force F=350 N)

Program Midas-Gen demonstrates the most close results to experiment. So it will be applied further to describe more complex system «structure-soil».

2) Model testing of combined mooring/fender dolphin. To increase energy-absorbing capacity of mooring/fender dolphins it is worked out and researched a new design of combined tubular mono-pile structure. It incorporates internal flexible pile and damping element (cushion) placed at the zone of pile head (Figure 5). Work of damping element is correlated with bending strain parameters of combined mono-pile structure.

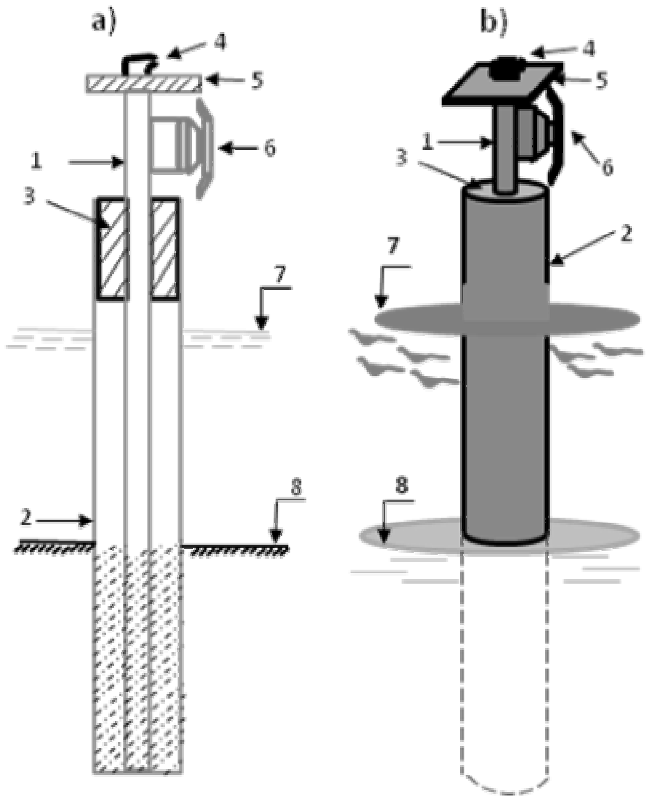


Figure 5 – Mooring/fender dolphin: a – cross-section; b – 3D view:
1 – internal pile; 2 – external tubular pile; 3 – damping element (cushion);
4 – bollard; 5 – superstructure; 6 – fender; 7 – water level; 8 – bottom level

External force provoked by ship mooring (either via bollard of via fender) initially is taken by internal pile. While bending the internal pile presses the damping element and through this cushion transfers the decreased force to the external tubular pile. Thus due to joint work of three elements (internal and external piles and damping cushion between them) the dolphin may take essential ship load. It makes needless use of large diameter (3-4 m) heavy-walled (40-60 mm) tubular pile of ruggedness strength (and, correspondingly, of high cost). In such a case combined dolphin made of two comparatively small diameter piles (of about 1 m for external pile and 0.5 m for core one) may be profitably applied to withstand large operational force.

This structure has been tested on physical model in laboratory (Figure 6). Pile supports (both internal and external) are fixed by special clamps to exclude their tips' displacements. Total length of the piles was preliminary determined according to the recommendations of actual design codes.

a)

b)

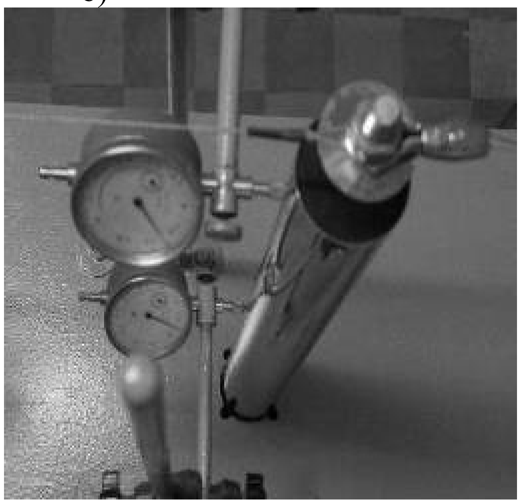


Figure 6 – Model testing of mooring/fender dolphin model in laboratory
a – general view of the system; b – measuring and loading facilities

The same model was analyzed by 3-D numerical simulation (FEM) using different programs (as above) in order to determine the most proper calculation model and software regarding peculiarities of the model and interaction among its elements. Calculation scheme is presented on the Figure 7, where points d1, d2, d3 and d4 correspond to the location of displacement indicators (to simplify external force application both tubes in this test were fixed horizontally).

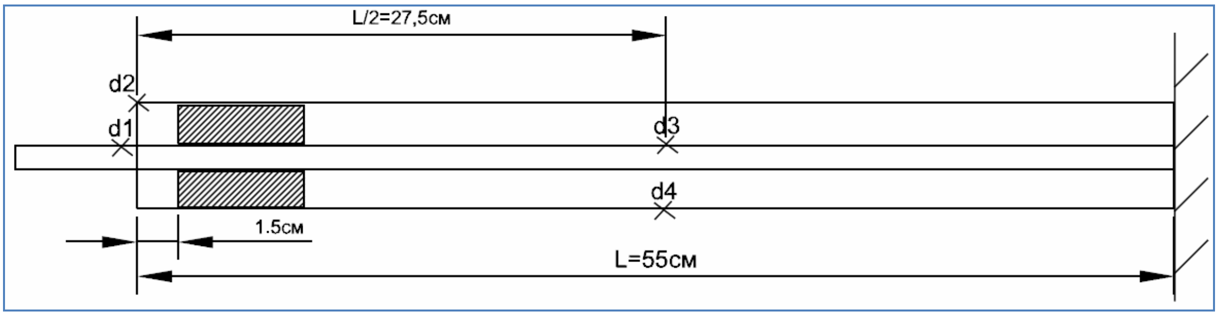


Figure 7 – Simplified scheme of the dolphin's model

The most appropriate results were obtained by use of program Midas-Gen when for description of damping element work elastic-plastic model of applied Druker-Prager (Figure 8 and Table 1).

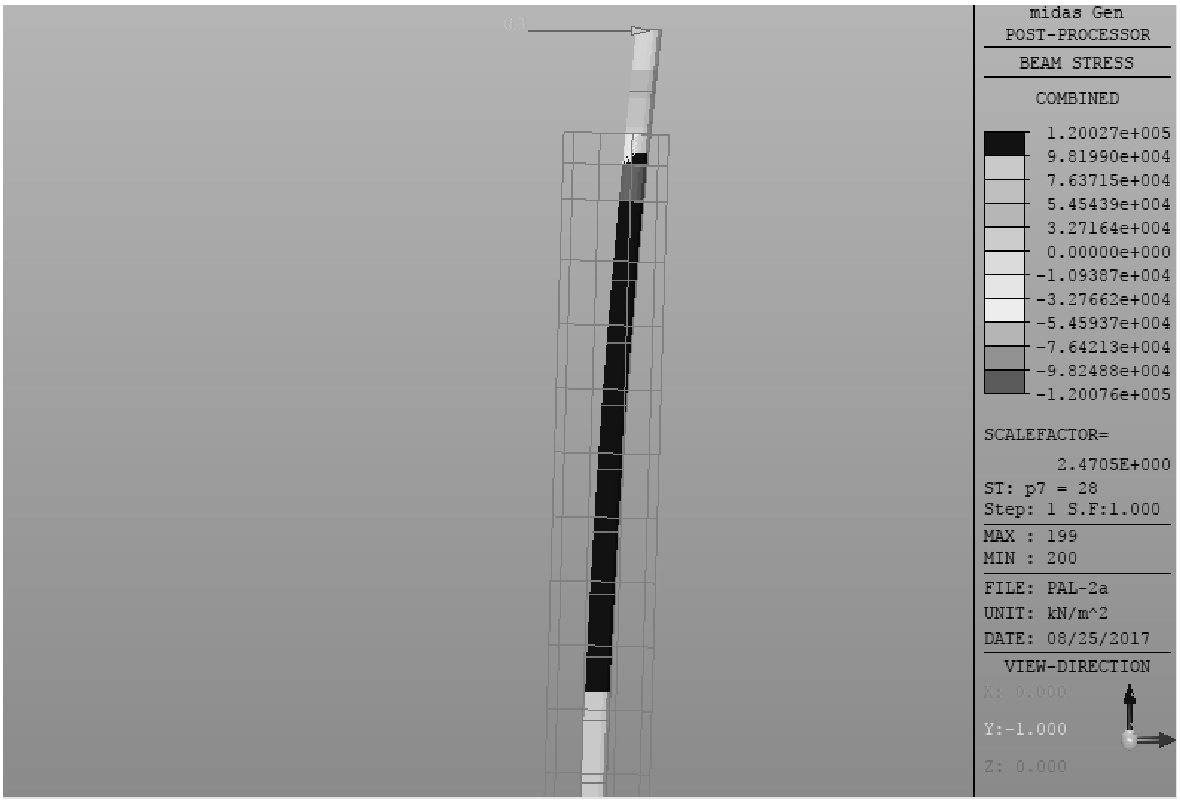


Figure 8 – Deformations of the dolphin under lateral force application (Midas-Gen)

Table 1 – Calculated and measured horizontal displacements of internal and external piles at points d1 and d2

Force, N	Indicator d1, mm			Indicator d2, mm		
	test	calculation	difference	test	calculation	difference
4	1,525	1,443	0,082	0,745	0,837	-0,092
8	2.310	2,880	-0,570	1,545	1,673	-0,128
12	3,200	4,320	-1,120	2,415	2,508	--0,093
16	5,050	5,760	-0,710	3,315	3,343	-0,028
20	7,050	7,190	-,0140	4,265	4,177	0,088
24	9,110	8,610	0,500	6,015	5,010	1,005
28	10,910	10,030	0,880	7,045	5,844	1,201
32	12,600	11,440	1,160	7,960	6,677	1,283

So discrepancy of test and calculations reaches up to 17 %. Therefore program Midas-Gen and above mentioned model may be applied for considered structure design for further more complicated stage of the system «dolphin – soil». Besides in general, results of tests demonstrated expected contribution of both piles (internal and external) and damping element either qualitatively or quantitatively.

Conclusions. Fulfilled experiments have given possibility to check effectiveness of innovative structural and technological solutions effectiveness of piled cluster and mooring / fender dolphin on combined mono-pile.

The first stage of the planned tests (experiments without soil with fixed piles' tips) occurred to be useful for study of proposed structural peculiarities of both new designs as well as for appropriate calculation program determination (by comparison of measured and calculated data).

Obtained results may be used as sufficient background for the second stage of planned experiments when both innovative structures will be tested in the sand box for detailed study of structure-soil interaction.

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DRILL INJECTED TECHNOLOGY APPLICATION EXPERIENCE FOR THE PURPOSE OF GROUND REINFORCEMENT AND PILE ERECTION DURING RECONSTRUCTION

The drill injected technology for pile erection and shallow foundations strengthening implementation in practice is described in the article. The reconstruction of the building, the topography and geotechnical conditions of the building footprint analysis are presented. The article reveals the design solutions for reconstruction of existing building for industrial purposes, and describes the basic technological processes. It is considered design scheme of the building, the results of the spatial frame calculation, the substantiation of bases slab base strengthening necessity provided design solutions for building new above-ground building structures. The article describes the case of pile foundations for the elevator shaft building, the description of the technical solutions in piling, the results of piled-raft foundation settlement observations, which was loaded by the loads from the walls and ceilings of mine and lifting equipment. The paper discloses the design of technical solution for existing slab base of special technology foundation strengthening, the technology of amplification, as well as the results of experimental and control sensing.

Keywords*: injection, settlement, foundation, pile, test, observation, the raft.*

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ДОСВІД ЗАСТОСУВАННЯ БУРОІН’ЄКЦІЙНОЇ ТЕХНОЛОГІЇ ДЛЯ ПОСИЛЕННЯ ОСНОВ ТА ВЛАШТУВАННЯ ПАЛЬ ПРИ РЕКОНСТРУКЦІЇ

Досліджено питання впровадження в практику будівництва буроін’єкційної технології для влаштування паль і посилення основ фундаментів мілкого закладення. Наведено опис будівлі, що реконструюється, аналіз рельєфу й інженерно-геологічних умов площі забудови. Розкрито проектні рішення з реконструкції існуючої будівлі для виробничих потреб, описано основні технологічні процеси. Розглянуто розрахункові схеми будівлі, результати об’ємного просторового розрахунку каркаса, наведено обґрунтування необхідності посилення основи плитних фундаментів, проектні рішення щодо вбудування нових надземних будівельних конструкцій. Обґрунтовано влаштування пальових фундаментів для вбудованої ліфтової шахти, дано опис технічних рішень з улаштування паль, наведено результати спостережень за осіданням пальово-плитного фундаменту, що завантажений стінами і перекриттям шахти та ліфтового обладнання. Наведено проектні технічні рішення з підсилення основ існуючих плитних фундаментів буроін’єкційною технологією, технологією посилення, а також результати контрольного зондування.

Ключові слова*: ін’єкція, осідання, основа, паля, спостереження, ростверк.*

Introduction. There is a large number of unfinished construction of buildings and structures, industrial and residential purpose in the Republic of Belarus. These buildings are not being exploited, physically and morally obsolete nowadays. The new investors that come on the market in our country, repurchase such buildings, perform their reconstruction and introduce them into service. In one of these renovation projects it is involved
the arrangement of a new floor, the device of the elevator shaft was used for the application of special technologies.

Building footprint topography and geological analysis. The building was built on the planned site, the area is confined to the fluvioglacial plain within Minsk hills. The primary relief of the building footprint of the building was changed during the construction of adjoining buildings, roads, laying of city communications, landscaping. Adjacent to the building territory is not landscaped, grass is missing partially.

As a result of the 2015 survey, it was identified that the geological structure to a depth of 15.0 m from the level of planning involved the following deposits (top – down):

A) modern anthropogenic (artificial) deposits of the Holocene horizon (thIV), represented by different grain size of sand heaps, from clay sand with inclusions of 10-15% of the gravel, pebbles and construction waste (broken bricks, concrete rubble, glass, wire). Opened the maximum capacity of 3.9 m Prescription fills for over 15 years. Bulk soil lies above the depth of the foundations, is characterized by heterogeneity of composition, addition and compressibility.

B) deposits Ajskogo horizon (g II sz)

– fluvioglacial nadmiernie (fIIsžs) deposits, represented by Sands of medium, coarse and gravelly yellow, brownish-yellow, malovlazhnyh, buried under loose soil with the opening capacity of 4.4 and 6.4 m.

– moraine (gIIsž) deposits are represented by sandy loam red-brown solid, with the inclusion of gravel and pebbles up to 15%, with frequent layers of sand up to 10 cm, overlain by fluvioglacial Sands with depths of 7.6 and 10.0 m.

Soil characteristics are given in table 1.

Table 1 – Soil characteristics

	Name of soil	Humidity	Density, кН/m^3			Cohesion, кPa			Angle of friction, degree			Deformation modulus. MPa	p_d MPa
		w	γ_H	γ_{II}	γ_I	c_H	c_{II}	c_I	φ_H	φ_{II}	φ_I	E	
	Anthropogenic (artificial) deposits of the Holocene horizon (thIV)												
1	Made ground	12,4	18,7	18,3	18,0								2,0
	Fluvioglacial nadmiernie (fIIsžs) deposits												
2	Sand medium strength	4,3	17,4	17,4	17,4	1,4	1,4	9	36	36	33	29	7,6
3	Sand large and gravelly	6,2	15,6	15,6	15,6	-	-	-	34	34	31	12	1,5
4	Sand large and gravelly medium strength (2,8 ≤ Pd, МПа ≤ 6,0)	5,7	16,8	16,8	16,8	0,3	0,3	0,2	37	37	34	20	4,2
5	Sand large and gravelly (6,0 <Pd, МПа ≤ 14,0)	5,1	17,7	17,7	17,7	0,7	0,7	0,5	39	39	35	35	8,6
	Moraine (gIIsž) deposits												
6	Sandy loam	8,7	21,8	21,7	21,7	36	36	24	28	28	24	25	6,0

The area is characterized by the absence of groundwater in the hydrogeological respect.

Overview of existing object. The frame of the building was built in 1990, in the 1990 to 2014, no construction work in the building was conducted. Building is multi-storied, rectangular in plan, with dimensions for the outer axes 111,325×21,245 m, the level of responsibility – II (normal), the complexity class of the building according to STB 2331-2014 K3. Engineering networks of water supply, sanitation, electricity to the building before the renovation was summed up.

The building consists of two main blocks structurally:

– multi-storey block (in the axes 1-11) was made with slab foundations on a natural basis in full frame with load-bearing structures of reinforced concrete prefabricated elements with self-supporting enclosing brick walls of the stairwell and wall claydite-concrete panels.

– 2nd administrative block B, located in the axes 12-20 – frameless, four-storey, including basement, done with the bearing longitudinal reinforced concrete walls in the basement and brick walls on 1-3-rd floor; ceilings and floors of precast concrete slabs.

The roof of the building is combined, roll with organized internal drain.

The results of the survey, based on [2,] showed that building structures have common critical defects.

Design solutions. The new owner desicion was to design project documentation for the reconstruction of the existing building into a multifunctional centre for social and industrial purposes.

The reconstruction is provided by stages. At the 1st stage the project envisages reconstruction of block A (in the axes 1-11) for the production designation aim, and the bringing in of external engineering networks; vertical planning of the territory, construction of new transformer substations, landscaping of the production site. At the 2nd phase of reconstruction of housing social purpose the Customer has decided to make reconstruction of the production part of the building to contain the printing company (according to [3]) with the capacity of 5000000 liters/month (for labelling products, cartons, booklets, catalogues, magazines, calendars of various kinds, promotional products, posters, books, brochures) at this stage.

The printing plant consists of the main parts for production purposes (typography in axes 3-11/B-E) and the administrative part in axes 1-3/B. Reconstruction of the production, made according to [6], unit involves the division of the 2nd floor in two. New recessed floors are devoted for administrative office needs. Design value of uniformly distributed regulatory burden on the 1st floor in industrial premises and 20 kPa, on the 2nd floor – 13 kPa, on the 3rd – 2,0 kPa.

Design solutions implementation. The following major works were completed while building:

1) a brick elevator shaft for a 3.2 tons capacity elevator was built in the axes 2-3/D-E. The Elevator shaft has a pit and machine room, protruding above the roof. The foundations of the lift shaft – pile [made according to 5, 8], CFA piles united on top by a monolithic reinforced concrete grillage. The walls of the Elevator shaft is designed and made of solid ceramic bricks. The ceiling of the engine room and covering – reinforced concrete castorbridge on steel beams;

2) the existing concrete slabs strengthening was made at elevation +6,000 for design regulatory load 13 kPa;

3) the new monolithic reinforced concrete slab on steel beams in the axes 5-11/B-E at around 10,400 and in axes 1-2/b-E at around +3,000 were erected.

4) the slab foundation basement was strengthened by the injection molding.

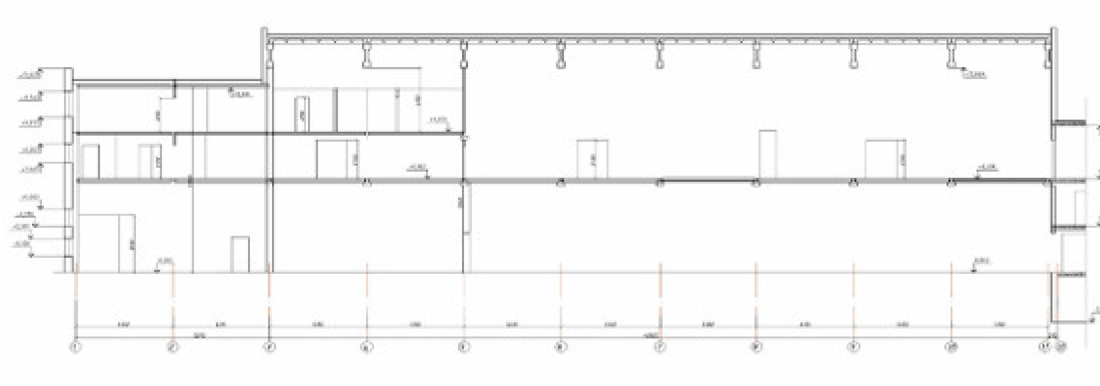
Volumetric building frame calculation. To determine the correct force arising from existing building structures it was determine the required elements and types of amplification, was determined deformations and precipitation was performed volume calculation of the whole building block.

Static calculation of a building frame in two groups of limit states was performed on the software package SOFiSTiK. The estimated complex implements finite element modeling of static and dynamic design diagrams, checking of stability, the choice of disadvantageous combination of efforts, selection of reinforcement for reinforced concrete structures, verification of the bearing capacity of steel structures in accordance with the applicable rules of structural engineering of the Republic of Belarus.

The calculation considered the spatial work of building structures. The estimated model includes only load-bearing elements of the building: columns, tie, wheels and floor coverings. Columns, braces and joists present the core elements of the General type. Plates of overlappings and coverings were presented by the bending flat final elements that have node six standard degrees of freedom (3 linear and 3 rotational) and are able to withstand longitudinal and lateral forces, bending moments.

It was considered all permanent and short-term loads and impacts that correspond to its functional purpose and constructive solutions when performing the calculations.

a)



b)

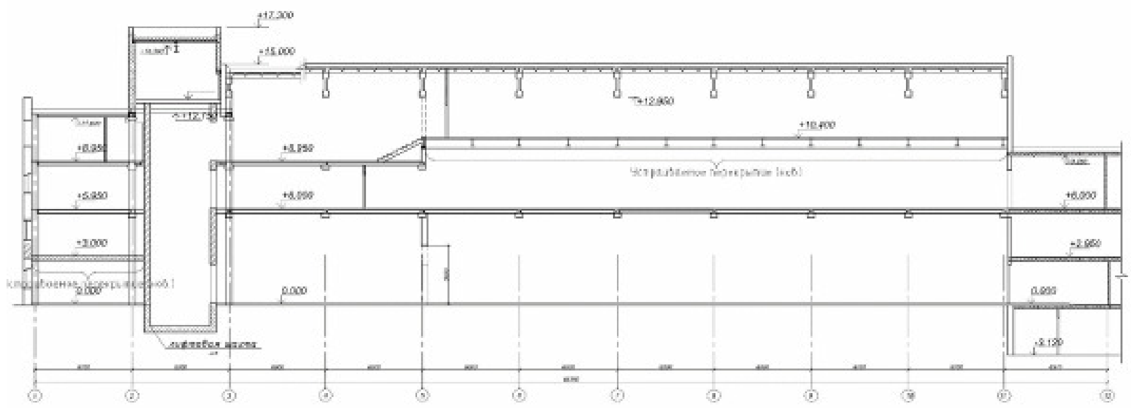


Figure 1 – Building scheme
a – existing building, b – design solutions

The calculation results. The spatial frame of the building performed calculation proofed the need of the existing building structures and grounds reinforcement: the maximum relative difference between the sediment foundations without reinforcement made up 27/6000 = 0,0045, and exceed the maximum normalized value ΔS/L = 0,002, in the design scheme have been adjusted with respect to the strengthening of injection foundation methods, which reduced the value of the maximum precipitation is the foundation of the middle column with 54 mm to 34 mm, and thus the maximum relative difference between residue of the foundations amounted to 8/6000 = 0,0001.

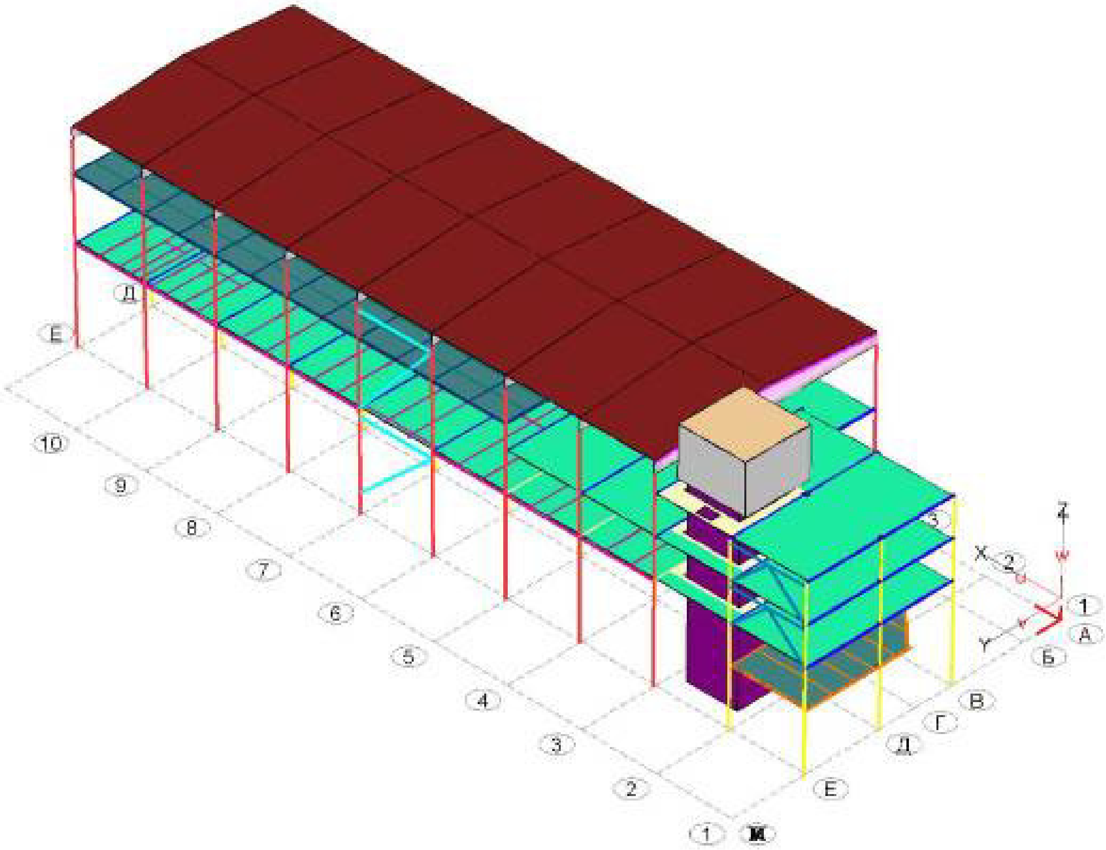


Figure 2 – Building calculation model

The elevator shaft device. The connection between the 1st and 2nd floors is provided be the 3.2 tons capacity elevator, which is situated in the axles 2-3/ B-D. The building frame column foundations are located near the mine being built – slab, monolithic with the size of the midsole in terms of 2.28...2,40×1,80 m and a depth from the floor level – a...3,540 3,710. Due to the presence of a sufficiently large capacity of bulk soils and also near located framework columns shallow foundations, in was made a decision to erect the device pile foundation under the elevator shaft [4, 5, 10].

Due to the cramped conditions of construction, his tight schedule, and testing research on influence of injection molding on the bearing capacity and deformability of foundations pile foundations it was decided to perform the loading experienced piles loads from building constructions erected an Elevator shaft with the observation of rainfall system, «Bush pile - raft Foundation». After pouring of pile caps it was found only 7 brands which according to [1] and was determined by precipitation (Fig. 3). Precipitation amount was measured in a specially designed program with step size of 0.1 N, where N is the design force on the raft. The latest measurement of the sediment was performed after all construction works and commissioning of elevators with its test.

Figure 3c shows that the measured value of the residue does not exceed the maximum values imposed [1] to the piles experienced strain.

The results of the experimental investigations established that wall of the concrete sump is a raft perceiving impact of the Elevator shaft building and transmitting it to the pile. However, precipitation is experienced marks located on the pit walls that are more important than sediment in the Central part (Fiig. 3). This is because at the center of the raft bottom directly above the Central piles brand No. 7 were installed and it can be concluded that the rigidity of a raft monolithic slab foundation are not completely evenly distributes impact loads.

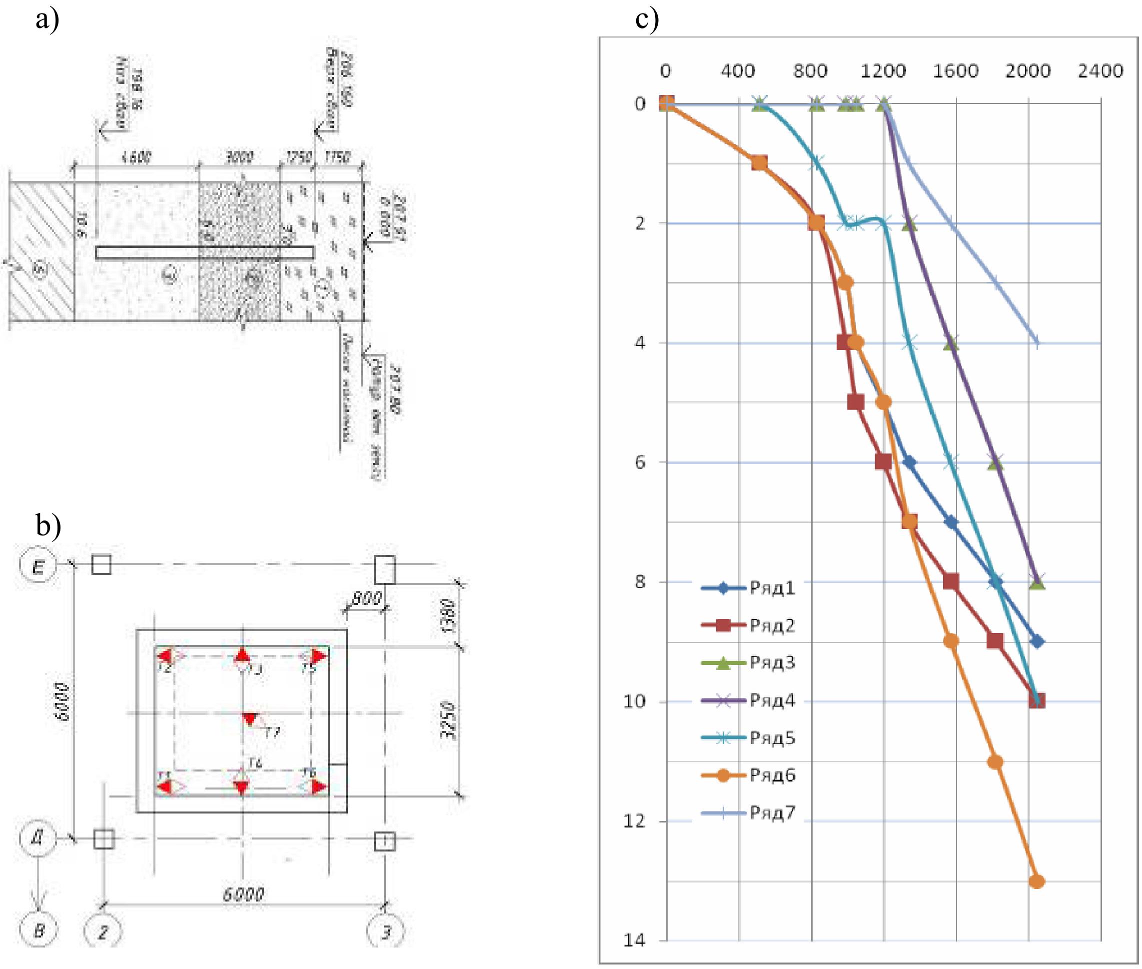


Figure 3 – layout of experienced brands
a – geological column with applied pile; b – layout of an experienced brands (1-7);
c – experienced brands settlements

Strengthening foundations for the perception of new loads. The foundations of interior columns along the axis B/4-10 – slab, monolithic with the size of the midsole in terms of 1,7...1,8×1,6...1.7 m, and the depth from the floor level is 3.1...3.2 m. The columns of the building are rigidly embedded in monolithic glasses. The base of the foundation is sand of medium strength (EGE – 2), a layer thickness of 1.8 m, underlain by coarse Sand large and gravelly medium strength (EGE-4), either directly the large sand gravelly medium strength (EGE-4).

It was decided according to [9, 10] to perform applying drillinjected technology for the base strengthening.

Injection strengthening of foundations are performed in the following sequence:

– 1 drilling under the protection of casing pipes with a diameter of 133 mm to the design level with the help of compact drilling machine;

– 2 in the well it was lowered two injector where the lower ends were closed with plugs;

– 3-hole on top filled with dry sand medium with simultaneous extraction of the casing after the wellhead was cement-sand mortar to a depth of 0.3 m;

– 4 after curing the cement-sand mortar was produced by the injection of cement mortar W/C = 0.45 initially, after a short injection tube T2 (60L), and then through a long (70hp); The fix solution was made smoothly, gradually increasing the pressure to value not more than 0.3 MPa for the short injector, and not more than 0.8 MPa, for long the injector. The speed of the set pressure should be 0.1 MPa per minute. Water solution for injection was prepared with a/C=0,45 for the Portland cement grade not lower than 400 with careful mixing each batch for at least 10 minutes prior to pumping in order to facilitate trade and to reduce the ability to sedimentation;

– 5 after performing the injection amplification in accordance with [8] to assess the variability of soil properties and efficiency of gain slab base it was made soils dynamic sensing. The results of the tests are discussed later.

a) b)

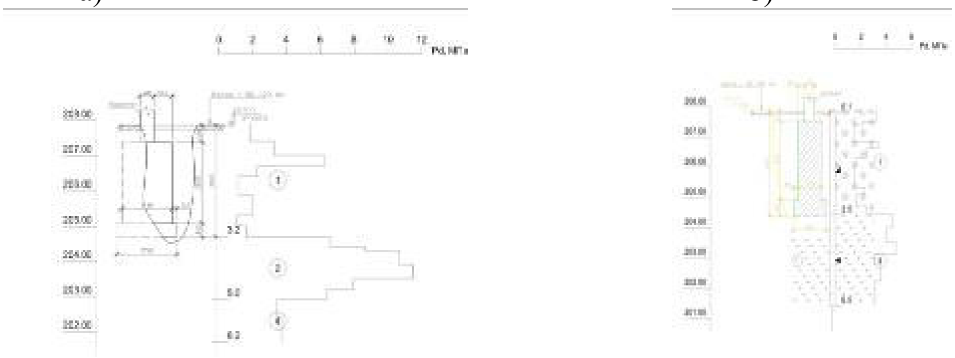


Figure 4 – Geological column with applied foundations.

a) b) c)

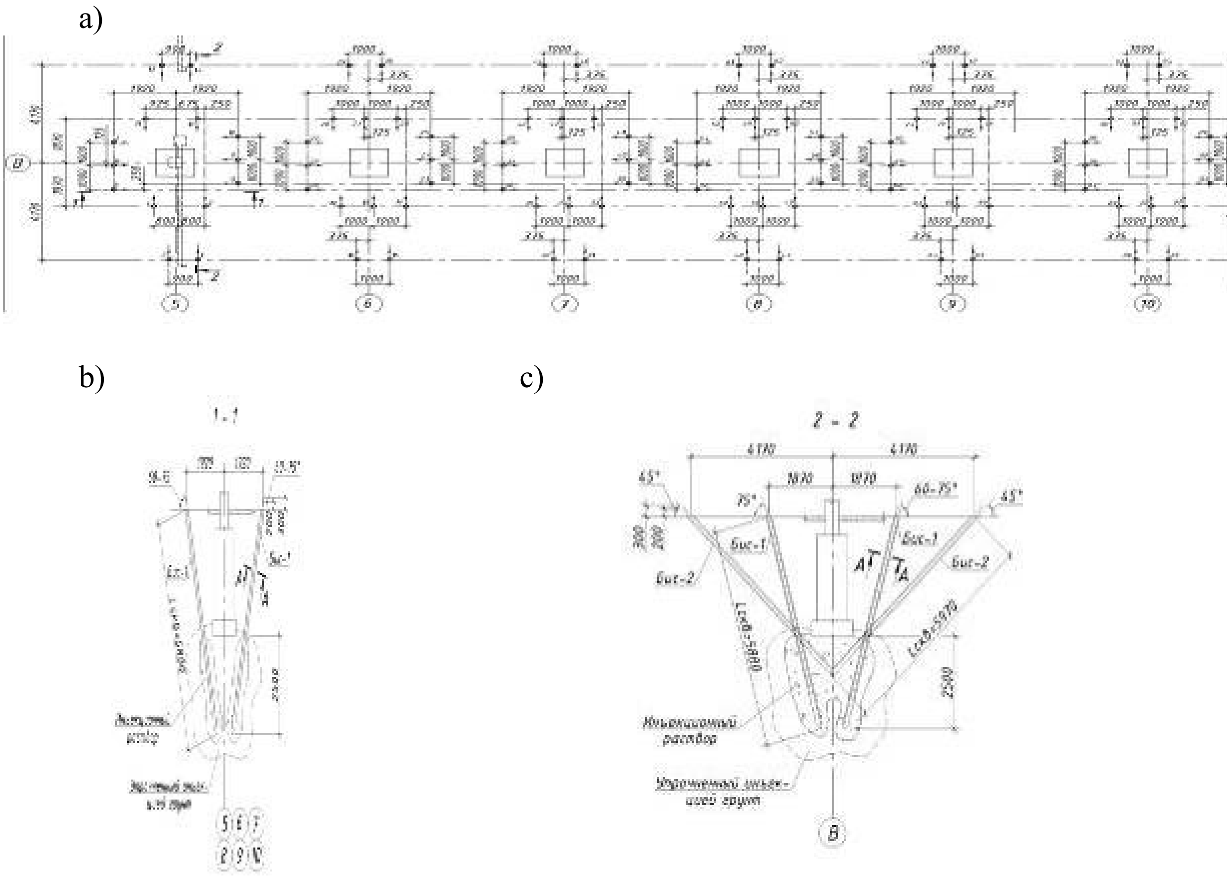


Figure 5 – Design solutions for the strengthening of the foundation bases
a – the scheme of location of injection wells; b – strengthening along the section 1-1;
c – strengthening along the section 2-2

Determining the quality of injection molding. The purpose of injection molding quality control was to check soils bearing capacity after grouting reinforcement. Dynamic sensing of exposed soils located in the area of the columns Nos. 5, 9. It was performed seven trials. The tests were carried out with a probe driven in accordance with STB 1241-2000.

Graphs of the dynamic resistance of the soil sensing and modifying the strength characteristics of silty sand and medium sandy loam according to the results of the performed works are presented on the figure (it should be noted that the graphs of the dynamic sensing give the similarity streaming chart of the static sensing for better visualization of obtained results).

As a result, at the probing set, it was found out that hardened injection solutions thick soils are located in depths of three feet.

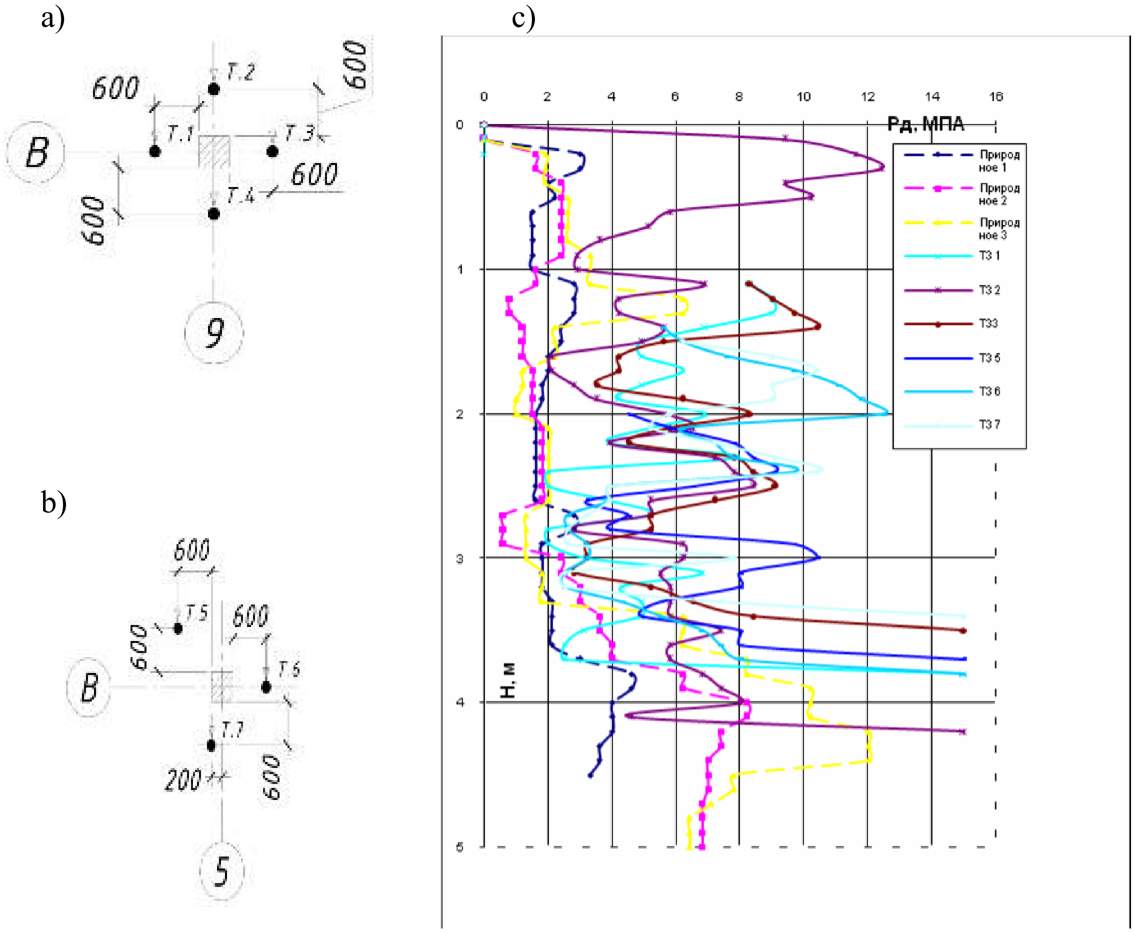


Figure 6 – Quality control in injection molding

a – the arrangement of sensing points at the axis 9/B; b – the arrangement of sensing points at the axis 5/B; c – results of the sensing

Conclusion. Thus the drillinjekted technology usage had allowed in a rather difficult geological conditions and in conditions of the existing building to realize the need for vertical transport connection of the designed production printing, and the monitoring confirmed the suitability of the elevator shaft for operation. Also the results of sensing of soils reinforced injection around the grounds confirmed the hardening of the soil and the adequacy of the base bearing capacity.

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THE ANALYSIS OF THE STRESS-STRAIN STATE OF THE SYSTEM «BASEMENT - PILE FOUNDATION – STRUCTURE» CONSIDERING SWELLING PROPERTIES OF SOILS

This work analyzes factors affecting swelling soils and presents a calculation algorithm of structure on swelling soils base for the plane problem. In this work an analysis of the factors influencing the swelling of soils is made, the algorithm for calculating the structure with the base with swellable soils for a plane problem is given. At the same time, the swelling process is considered as an additional influence, close in nature to temperature, and the swelling soil is considered a material having orthotropic properties. The value of the relative swelling depends on the level of the stress state, while the value of the main stresses is compared with the magnitude of the pressure of swelling. Therefore, to determine the deformation characteristics of the swelling soil, several variants of the stressed state of the soil have to be considered. The effectiveness of the obtained solution has been verified according to the example for pile foundations.

Keywords: *swelling soils, shrinkage of swelling soils, orthotropic properties, final element method, system «basement – pile foundation – structure».*

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АНАЛІЗ НАПРУЖЕНО-ДЕФОРМОВАНОГО СТАНУ СИСТЕМИ «ОСНОВА – ПАЛЬОВИЙ ФУНДАМЕНТ – СПОРУДА» З УРАХУВАННЯМ ВЛАСТИВОСТЕЙ НАБУХАЮЧИХ ГРУНТІВ

Виконано аналіз факторів, що впливають на набухання ґрунтів, приведено алгоритм розрахунку споруди з основою з набухаючими ґрунтами для плоскої задачі. При цьому процес набухання ураховано як додатковий вплив, а набухаючий ґрунт розглядається як матеріал, що має ортотропні властивості. Доведено, що величина відносного набухання залежить від рівня напруженого стану, при цьому величина головних напруг порівнюється з величиною тиску набухання. Для визначення деформаційних характеристик набухаючого грунту доводиться розглядати кілька варіантів напруженого стану ґрунту. Ефективність отриманого рішення перевірено на прикладі для пальових фундаментів.

Ключові слова: *набухаючі ґрунти, усадка набухаючих ґрунтів, ортотропні властивості, метод скінчених елементів, система «основа – пальовий фундамент – споруда».*

Introduction. Swelling soils are widespread. Such soils are common in Egypt, Burma, USA, Canada, South Africa, Ethiopia, Sudan, Iraq, Syria, and in India more than 30% of the territory is occupied by so-called cotton soils. In the CIS countries such soils are found in Kazakhstan, Georgia, Azerbaijan, Moldova, Russia (the Volga region, the North Caucasus and other regions), Ukraine (Crimea, Kupyansk, Kharkov, Poltava, etc.).

A characteristic feature of swelling soils with soaking is sharp increase in their volume and decrease in their load capacity, which leads to considerable deformations in the structure. Therefore, when projecting, it is necessary to consider the effect of swelling on the entire system «base-foundation-structure» for more reliable exploitation [7 – 10]. Existing norms, unfortunately, do not allow to create such a model and advise to calculate deformations from external loads separately from deformation of soil swelling.

A review of the latest sources of research and publications. In work [1] the method of finding characteristics of swelling soils for calculation of the system «basement – foundation – construction» was obtained. An algorithm for calculating the «basement – foundation – structure» system on swelling soils is given, considering the orthotropic properties of these soils for a planar problem. It is proposed to represent a soil massif consisting of swelling soils in the form of a linearly deformed medium, swelling of the base to be taken into account as an additional effect that is close in nature to temperature, and the swelling soil is considered as a material possessing orthotropic properties [2, 3, 6].

Allocation of previously unresolved parts of a common problem. This method allows considering the effect of swelling on the SDC plate foundation more accurately. It is necessary to check the effectiveness of the solution for pile foundations.

Formulation of the problem. Improvement of the soil swelling registration algorithm for SDC system «basement – pile foundation – construction».

Main material and results.

1. Factors affecting the swelling of clayey soils

Swelling soils include clayey deposits, the characteristic feature of which is the increased density and high content (65–85%) of clay particles with a particle size of less than 0,005 mm. In natural occurrence, these soils (clays) are characterized by firm and turgid consistency at a specific gravity from 19,5 to 20,5 kN / m^3. The porosity of soils is in the range from 41 to 48%, with a humidity of 15–18% in the roof and 25–30% in the middle layers and the base of the strata. As a result of the moistening of these clays, their volume increases by 12–25%, and in some cases by 30–36%. As a result of swelling, the specific gravity of clays decreases to 17,7–18,7 kN / m^3, and the porosity increases to 50–58%. The moisture content of the soil increases to 36–48% and indicates the transition of clays to plastic state, which sharply reduces their bearing capacity. The value of the normal clay swelling forces in natural occurrence reaches 350–400 kPa under the pile face, and 30–37 kPa (tangential) along the lateral surface of the piles. Disintegrating when swelling, the clay raises the thickness of the overlying coverslips, which, in turn, tends to raise the foundation slab or the end of the piles.

2. Determination of swelling soils characteristics

The basic relationship for the magnitude of soil swelling deformation is taken according to Sorochan E.A. [4]:

$$\underset{i=1,2,3}{\varepsilon} = \varepsilon_i^y + \varepsilon_{i,sw} + \varepsilon_{i,sh}\,, \tag{1}$$

where ε_i^y – major deformations at the base of stresses in the ground;

$\varepsilon_{i,sw}$ – swelling deformations depending on changes in humidity and main stresses, the ratio [4]:

$$\varepsilon_{\substack{i,sw \\ i=1,2,3}} = m \cdot \Delta w \left(1 - \frac{P}{p_{sw}}\right), \tag{2}$$

where m – coefficient that takes into account the swelling properties of soils and determined experimentally;

Δw – change in soil moisture;

P_{sw} – pressure of swelling;

P – pressure in the direction of swelling.

It is assumed that the dependence of the swelling deformation on pressure and humidity is linear. The coefficient m is similar to the coefficient of thermal expansion. In addition, it considers the properties of the swelling soil.

3. Allowance for the anisotropic properties of swelling soils when calculating the system «base-foundation-structure» under conditions of planar deformation

The value of the relative swelling depends on the level of the stress state, while for σ_x, $\sigma_z < -P_{sw}$ – swelling does not occur. Therefore, to determine the deformation characteristics of the swelling soil, several variations of soil stressed state have to be considered.

For a planar problem, it can be distinguished 9 variants of stresses combination which conditions [1]:

– $\sigma_z > 0$ and $\sigma_x > 0$;
– $\sigma_z < -P_{sw}$ and $-P_{sw} < \sigma_x < 0$;
– $\sigma_x < -P_{sw}$ and $-P_{sw} < \sigma_z < 0$;
– $-P_{sw} < \sigma_z < 0$ and $-P_{sw} < \sigma_x < 0$;
– $\sigma_x > 0$ and $-P_{sw} < \sigma_z < 0$;
– $-P_{sw} < \sigma_x < 0$ and $\sigma_z > 0$;
– $\sigma_x < -P_{sw}$ and $\sigma_z > 0$;
– $\sigma_x > 0$, $\sigma_z < -P_{sw}$;
– $\sigma_x < -P_{sw}$ and $\sigma_z < -P_{sw}$,.

1) $\sigma_z > 0$ and $\sigma_x > 0$

$$E_{np} = \frac{E}{1-\mu^2}, \qquad \mu_{np} = \frac{\mu}{1-\mu}, \tag{3}$$

where E – modulus of soil deformation;

μ – Poisson's ratio.

2) $\sigma_z < -P_{sw}$ and $-P_{sw} < \sigma_x < 0$.

Let

$$\frac{P_{sw}}{P_{sw} + Ekm\Delta w} = \alpha,$$

Then

$$E_x^{np} = \frac{1}{\dfrac{1-\mu^2\alpha}{E} + \dfrac{km\Delta w}{P_{sw}}} \quad \text{(a)} \qquad \mu_{xz}^{np} = \frac{\mu + \alpha\mu^2}{1-\mu^2\alpha + E\dfrac{km\Delta w}{P_{sw}}} \quad \text{(b)}$$

$$E_z^{np} = \frac{E}{1-\mu^2\alpha} \quad \text{(c)} \qquad \mu_{zx}^{np} = \frac{\mu + \mu^2\alpha}{1-\mu^2\alpha} \quad \text{(d)} \tag{4}$$

where P_{sw} – pressure of swelling;

m – coefficient determined experimentally;

Δw – change in soil moisture.

The swelling soil acquires orthotropic properties with deformation characteristics, determined by formulas (a) – (d).

3) $\sigma_x < -P_{sw}$ and $-P_{sw} < \sigma_z < 0$.

$$E_z^{np} = \frac{1}{\dfrac{1-\mu^2\alpha}{E} + \dfrac{km\Delta w}{P_{sw}}} \quad \text{(a)} \qquad \mu_{zx}^{np} = \frac{\mu + \alpha\mu^2}{1-\mu^2\alpha + E\dfrac{km\Delta w}{P_{sw}}} \quad \text{(b)}$$

$$E_x^{np} = \frac{E}{1-\mu^2\alpha} \quad \text{(c)} \qquad \mu_{xz}^{np} = \frac{\mu + \mu^2\alpha}{1-\mu^2\alpha} \quad \text{(d)} \qquad (5)$$

The swelling soil acquires orthotropic properties with deformation characteristics, determined by formulas (a) – (d).

4) $-P_{sw} < \sigma_z < 0$, $-P_{sw} < \sigma_x < 0$.

$$E_z^{np} = \frac{1}{\dfrac{1-\mu^2\alpha}{E} + \dfrac{km\Delta w}{P_{sw}}} \quad \text{(a)} \qquad \mu_{zx}^{np} = \frac{\mu + \alpha\mu^2}{1-\mu^2\alpha + E\dfrac{km\Delta w}{P_{sw}}} \quad \text{(b)}$$

$$E_x^{np} = \frac{1}{\dfrac{1-\mu^2\alpha}{E} + \dfrac{km\Delta w}{P_{sw}}} \quad \text{(c)} \qquad \mu_{xz}^{np} = \frac{\mu + \alpha\mu^2}{1-\mu^2\alpha + E\dfrac{km\Delta w}{P_{sw}}} \quad \text{(d)} \qquad (6)$$

5) $\sigma_x > 0$, $-P_{sw} < \sigma_z < 0$.

$$E_z^{np} = \frac{1}{\dfrac{1-\mu^2\alpha}{E} + \dfrac{km\Delta w}{P_{sw}}} \quad \text{(a)} \qquad \mu_{zx}^{np} = \frac{\mu - \mu^2\alpha}{E\left(\dfrac{1-\mu^2\alpha}{E} + \dfrac{km\Delta w}{P_{sw}}\right)} \quad \text{(b)}$$

$$E_x^{np} = \frac{E}{1-\mu^2\alpha} \quad \text{(c)} \qquad \mu_{xz}^{np} = \frac{\mu + \mu^2\alpha}{1-\mu^2\alpha} \quad \text{(d)} \qquad (7)$$

The swelling soil acquires orthotropic properties with deformation characteristics, determined by formulas (a) – (d).

6) $-P_{sw} < \sigma_x < 0$, $\sigma_z > 0$.

The swelling soil acquires the properties of orthotropy with deformation characteristics, determined by formulas similar to the combination 5.

7) $\sigma_x < -P_{sw}$, $\sigma_z > 0$.

$$E_x^{np} = \frac{E}{1-\mu^2} \quad \text{(a)} \qquad \mu_{xz} = \frac{\mu - \mu^2}{1-\mu^2} \quad \text{(b)}$$

$$E_z^{np} = \frac{E}{1-\mu^2} \quad \text{(c)} \qquad \mu_{zx} = \frac{\mu + \mu^2}{1-\mu^2} \quad \text{(d)} \qquad (8)$$

The swelling soil acquires orthotropic properties with deformation characteristics, determined by formulas (a) – (d).

8) $\sigma_x > 0$, $\sigma_z < -P_{sw}$.

The swelling soil acquires orthotropic properties with deformation characteristics, determined by formulas 7.

9) $\sigma_x < - P_{sw}$, $\sigma_z < - P_{sw}$.

$$E_x^{np} = \frac{E}{1-\mu^2} \quad (a) \qquad \mu_{xz} = \frac{\mu-\mu^2}{1-\mu^2} \quad (b)$$

$$E_z^{np} = \frac{E}{1-\mu^2} \quad (c) \qquad \mu_{zx} = \frac{\mu-\mu^2}{1-\mu^2} \quad (d) \tag{9}$$

Using software complexes operating on the basis of FEM, a finite element calculation scheme of the «basement – foundation – above – ground part of the structure» system is modeled in a flat version and force calculation is performed for the action of specified loads, including loading combinations. In this case, the stressed states of the base are determined, and nine zones are established where the corresponding deformation characteristics are determined and the stiffness characteristics of the finite elements are introduced into the initial information.

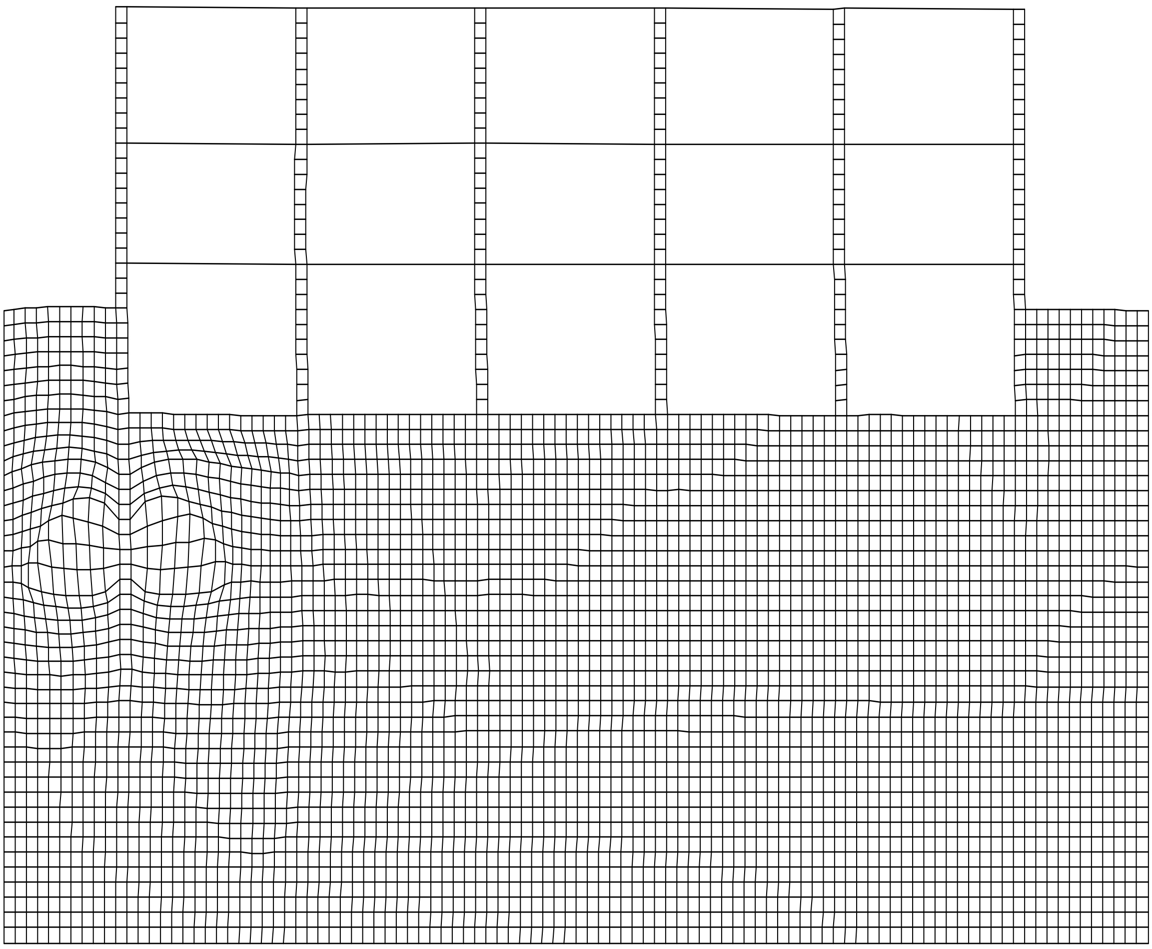

Figure 1– Deformed scheme

Further, the entire system is calculated for swelling from a change in the specified humidity, as temperature problem for temperature action equal to $m\Delta w$. The obtained stressed state of the base is summed with the stressed state from swelling and the position of the zones with different level σ_j is clarified in comparison with the swelling pressure - P_{sw}.

Then, the deformation characteristics for the new zones are refined and new calculation is performed. After calculating the new values of the total stresses, the zones are refined, etc. The calculation ends when change in the zones with different σ_j does not occur.

4. An example of swelling soils designs calculation

An example of a three-story brick building on a pile foundation in the town of Kupyansk, Kharkov Oblast, is considered. At the base there are clayeys with medium-swelling deposits at depth of 2,5 m and layer thickness of 1,5 m. Bored piles 6 m in length and 630 mm in diameter have been adopted. The base, walls and foundation are modeled by rectangular FE, 0,4×0,4 m in size. The overlap is modeled by the rod elements.

In the process of exploitation, it is possible to wet soil, and as a consequence, its swelling. It is assumed that the soaked soil works under temperature influence: $m\Delta w$ = 0,066, with a swelling ratio m = 1,237 [5].

The following results were obtained:

a) b)

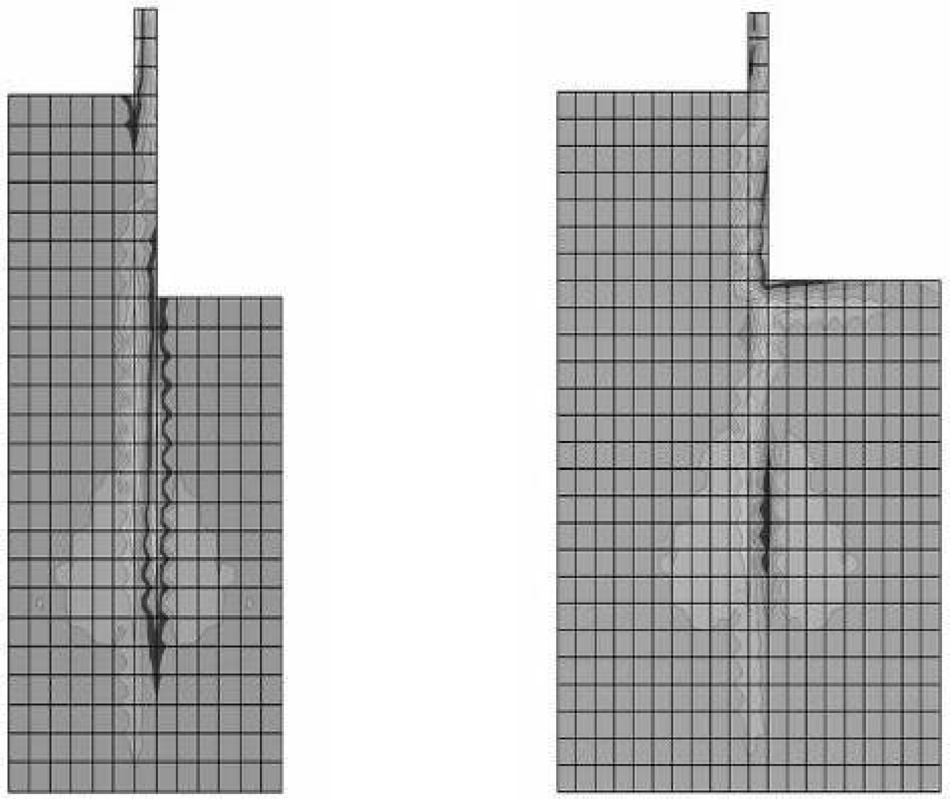

Fig.2 – Fragments of stress fields NZ

a – without considering orthotropy,

b –considering orthotropic properties of swelling soils

The difference between values of the main stresses when calculating with and without considering swelling for foundations reaches 30%, for above-ground structures - 7%.

Conclusions. 1. The swelling of the base can be considered as additional kinematic influence, close in nature to temperature.

2. The obtained solution for calculating the «basement -pile foundation-structure» system for a planar problem makes it possible to consider the orthotropic properties of swelling soils with more accurate results.

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«BUILDING – BASE» SYSTEM BOUNDARY DEFORMATIONS EXCEEDING PROBABILISTIC ESTIMATION METHODOLOGY

In the paper the method is proposed and the algorithm is developed for solving the problem of «building – base» system exceeding boundary deformations in reliability parametric theory framework probability (risk) determination. For calculations, the method of statistical tests (Monte-Carlo) was used. Using the powerful Mathcad complex, the computer program has been developed to implement the proposed calculation method has been developed. The calculations of the existing «building-base» system have been performed from determination of this system exceeding boundary deformations probability (risk). Analysis of the probability calculation indicates the need to improve existing approaches to determination the probability of deformations exceeding boundary value, because new methods will increase the reliability of solving geotechnical problems.

***Keywords:** probability, system reliability theory, statistical test method, distribution function of random variable distribution, «building – base» system, deformation of the base, calculation of deformations by the method of layer summing.*

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МЕТОДИКА ЙМОВІРНІСНОЇ ОЦІНЮВАННЯ ПЕРЕВИЩЕННЯ ГРАНИЧНИХ ДЕФОРМАЦІЙ СИСТЕМИ «БУДІВЛЯ – ОСНОВА»

Запропоновано методику та розроблено алгоритм вирішення задачі з визначення ймовірності (ризику) перевищення граничних деформацій системи «будівля – основа» в рамках параметричної теорії надійності. У процесі розрахунків застосовано метод статистичних випробувань (Монте-Карло). Розроблено комп'ютерну програму з використанням потужного комплексу Mathcad, вона реалізує запропоновану методику розрахунків. Виконано розрахунки існуючої системи «будівля – основа» з визначення ймовірності (ризику) перевищення граничних деформацій цієї системи. З'ясовано, що аналіз імовірнісного розрахунку вказує на необхідність удосконалення існуючих підходів до визначення ймовірності перевищення граничного значення деформацій, тому що нові методики дозволять підвищити надійність рішення геотехнічних задач.

***Ключові слова:** ймовірність, системна теорія надійності, метод статистичних випробувань, функція розподілу випадкової величини, система «будівля – споруда», деформація основи, розрахунок осідань методом пошарового підсумовування.*

Introduction. At present, calculations of foundation bases in accordance with normative documents are carried out by the method of boundary states. To estimate the reliability, a deterministic approach based on the application of partial reliability coefficients is used, which cases does not give an objective assessment in all cases. All the parameters included in the «base-building» system are random, and therefore the development of calculation techniques associated with the use of probabilistic methods for estimating the reliability of such a system is an urgent task.

The latest sources of research and publications analysis. The works of such scientists as Yu.L. Vynnykov [1], B.A. Garagash [2], N.N. Ermolaev and V.V. Mikheev [3] with their fundamental work «Reliability of structures foundations», A.P. Pshenichkin [4], V.A. Pshenichkina [5], A.N. Trofimchuk [6], V.M. Ulitsky [7], A.V. School [8] are devoted to the evaluation of the «base – building» system reliability. A comprehensive assessment of hydraulic structures reliability and safety was proposed by A.I. Weinberg [9]. Foreign researchers, such as – Beacher G.B [10], Honjo Y. [11], Shahin M.A. [12], C. Cherubini [13], Pereira C., also considered some geotechnical problems, solved with the help of probabilistic methods.

Isolation of previously unresolved parts of a mutual problem. The value of base deformation is random and it depends on a number of random variables: the characteristics of building materials, the acting loads and impacts, base deformation characteristics etc. The use of probabilistic methods for estimating foundation reliability by the normative method is an insufficiently studied problem.

Formulation of the problem. To develop calculating base deformations method, which allows probabilistic assessment of base boundary state occurrence by deformations.

Main material and results. Calculation of base deformations according to DBN B.2.1-10-2009 [14] is carried out with the aim of limiting the object (foundation) absolute or relative displacements, together both with the base, and with such boundaries that ensure the object performance and durability, become impossible manifestations of inadmissible deformations, ups, crevices, etc.

In order to estimate the risk (probability) of a particular species boundary state occurrence, it is necessary to solve the problem of parametric reliability theory with the use of probability theory apparatus. As a result of solving the problem, the value of the «base-building» system exceeding boundary deformations probability can be found. The following stages of calculation can be distinguished.

1. To compile the equations of connection between input parameters (loads and effects, properties of materials and soils, etc.) and output (the results of calculation) parameters, considering the system elements. Such an equation can be compiled on the basis of calculated dependencies analysis governed by design standards.

2. To prepare the initial calculation data which input parameters are distinguished into random and nonrandom (deterministic).

3. To distribute parameters of random variables considering determined initial data.

4. To determinate system exceeding boundary deformation value probability (risk) on the statistical dynamics corresponding solution basis.

Compiling the equations of connection.

To ensure operational reliability by the normative method, it is necessary to perform a calculation on the bases deformation where the following condition must be fulfilled:

$$s \leq s_u, \tag{1}$$

where s the base and structure joint deformation, which is determined by one of the DBN methods ; while for foundation pits with a depth of less than 5 m it is:

$$s = \beta \sum_{i=1}^{n} \frac{\left(\sigma_{zp,i} - \sigma_{z\gamma,i}\right) \cdot h_i}{E_i} , \tag{2}$$

where β is a dimensionless coefficient equal to 0.8;

E_i – the deformation modulus of the i-th soil layer along the primary loading branch, kPa;

h_i is the thickness of the elementary layer;

n is the number of layers within the compressible thickness H_c;

average stresses in the elementary layer:

$$\sigma_{zp,i} = \frac{\sigma_{zi} + \sigma_{z,i+1}}{2} , \tag{3}$$

$$\sigma_{z\gamma,i} = \frac{\sigma_{\gamma i} + \sigma_{\gamma,i+1}}{2} . \tag{4}$$

Here $\sigma_{zp,i}$ are additional stresses from the external load at a depth of z:

$$\sigma_{zp} = \alpha \cdot p , \tag{5}$$

where $p = p_{ap}$ is the average pressure below the base of the foundation, equal to:

$$p_{ap} = \frac{N}{b \cdot l} + \gamma_{mt} \cdot d + q , \tag{6}$$

N – vertical load at the level of the foundation upper cutoff;

b, l – dimensions of the foundation base, m;

γ_{mt} – the average value of the foundation specific weights, the soil and the floor located above the base of foundation, assumed to be 20 kN/m^3,

d – depth of the deposit, m;

q – load on the floor;

α is the coefficient of stresses attenuation depending on of the relative depth $\zeta = \frac{2 \cdot z}{b}$ and ratio foundation sides $\eta = l/b$;

$\sigma_{z\gamma}$ are the vertical stresses from the own weight of the soil taken in the foundation pit up to the level of the foundation base, at a depth of z:

$$\sigma_{z\gamma} = \alpha_{\kappa} \cdot \sigma'_{zg,0} , \tag{7}$$

where $\alpha_{к}$ is found in Table B.6 of DBN and depends on the relations $\zeta = \frac{2 \cdot z}{B_{\kappa}}$ and $\eta = l/b$;

B_{κ} – width of the pit;

$\sigma'_{zg,0}$ – vertical stress from the own weight of the soil taken from the foundation pit at the level of the foundation base and equal to $\sigma'_{zg,0} = \gamma_{zp} \cdot d_n$,

d_n – the depth of foundation laying in relation to the level of natural relief;

s_u – the boundary value of the foundation and structure joint deformation, governed by the norms.

When performing such calculations according to the normative methodology, the exploitation reliability of the «base-building» system is considered to be ensured if inequality (1) is satisfied. It is noted that the work does not consider the nonlinear work of the base soil.

Preparation of initial data

All directly or indirectly entering components of equation (1) are random, but some quantities can be assumed deterministic. In the present work, the following quantities are considered to be deterministic:

1) geometric characteristics of the foundation construction – b, l, d, d_n;
2) geometric characteristics of the enclosing structure;
3) geometric size of brick partitions and the thickness of the overlap;
4) the average specific weight of the foundation and soil on its edges γ_{mt};
5) the load on the floor q;
6) specific weight of backfill material and base soil.

Estimated soil resistance is determined deterministically.

All other parameters of the connection equation are considered random variables. These parameters include:

1) physical and strength characteristics of the enclosing structure materials (brick walls), overlapping, partitions;
2) deformation characteristics of the base soil (modulus of deformation E);
3) the vertical load on the foundation N, composed of the structures own weight G, the snow S and the payload Q, which are also random variables;
4) the average pressure under the foundation base p_{cp};
5) the stresses from the additional load σ_{zp};
6) deformations s.

The following distribution laws for random input parameters are adopted in the work.

1. The distributions $P_{\gamma b} = P_{\gamma b}(\gamma_b)$ and $P_{\gamma kk} = P_{\gamma kk}(\gamma_{kk})$ of random variables – the specific weights of concrete and brickwork, which are assumed to be normal with the mathematical expectations $m_{\gamma b}$ and $m_{\gamma kk}$, correspondingly, and the mean-square deviations $\sigma_{\gamma b}$ and $\sigma_{\gamma kk}$. The values of these parameters are determined by the tests results or by design standards.

2. The distributions $P_Q = P_Q(Q)$ and $P_S = P_S(S)$of random variables – payload and snow load, which are assumed to be normal with the mathematical expectations m_Q and m_S, correspondingly, and the standard deviations σ_Q and σ_S. These parameters values can be determined on the analysis basis of water reserves observation arrays in the snow cover, analysis of data on the residential and public fund premises survey, or adopted in accordance with design standards.

3. Distribution $P_E = P_E(E)$ –of the soil deformation modulus random value accepted to be normal with mathematical expectation m_E and the mean-square deviation σ_E. The values of this parameter are determined by analyzing the results of soil tests.

All calculations are performed for the estimated service life T.

Determination of exceeding boundary deformations probability by the Monte Carlo statistical test method

To determine the probability of exceeding the boundary deformations of the «base-building» system, it is advisable to use the method of statistical tests (Monte Carlo) using the normative method of deformations calculation by the layerwise summation method.

According to this method, N statistical tests are performed. For each test, calculations are performed according to the following algorithm.

1. It is set the random probability of the parameters uniformly distributed in the interval from 0 to 1:

1.1. specific weight of concrete $P_{\gamma b}$;
1.2. specific weight of brickwork $P_{\gamma kk}$;
1.3. payload intensity P_Q;
1.4. snow load intensity P_S;
1.5. soil deformation modulus E.

2. Using the values of the probabilities it can be found quantiles – the values of the corresponding parameters from the known distribution functions:

2.1. concrete specific weight $P_{\gamma b}$;

2.2. brickwork specific weight $P_{\gamma kk}$;

2.3. payload intensity P_Q;

2.4. snow load intensity P_S;

2.5. soil deformation modulus E.

3. The random values of the vertical load N are determined depending on the loads values from the own weight G, snow S, payload Q.

4. The random mean values of the mean pressure under the foundation base p_{cp} are determined from formula (6).

5. The calculated resistance of the foundation soil is determined, and the boundaries of layer-by-layer summation method applicability ($p_{ap} \leq R$) are delineated.

6. Stresses from the own soil weight σ_{zg}, stresses from the own soil weight taken in the pit basin $\sigma_{zy,}$ are determined by the formula (7), and the stresses $0{,}2\sigma_{zg}$ are determined.

7. The random values of the stresses from the additional load σ_{zp} are determined by the formula (5).

8. The random values of vertical deformations s_i in each elementary layer and the total value of s are determined by formula (2).

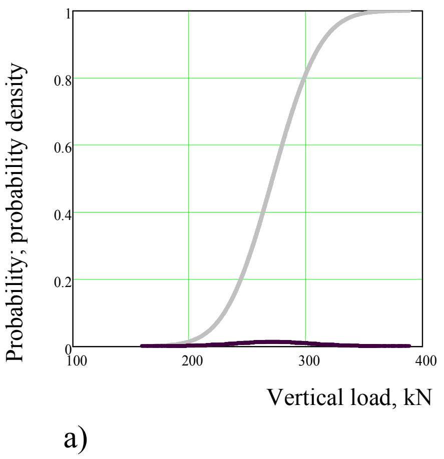
9. In each case, condition (1) is checked.

10. After all N tests being completed, the risk (probability) of exceeding the boundary deformations during the estimated service life P_T is calculated as the number of tests ratio*n* where $Y = s_u - s < 0,$ to the number of all tests N.

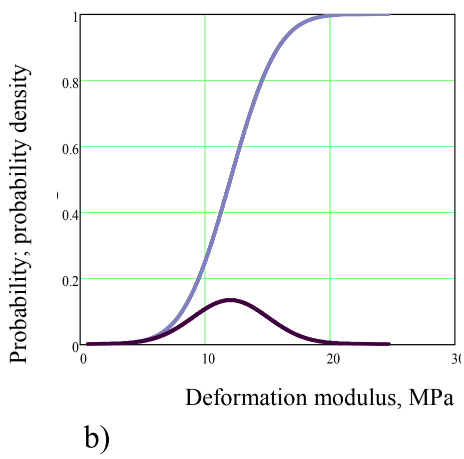
The number of tests N should be increased to more accurately determine the value of Y more accurately, in this case the number of tests was taken as $N = 1\text{x}10^4$. The author has developed the computer program for performing calculations to determine the risk of exceeding boundary deformations in the Mathcad environment.

Example of the calculation. As an illustration of the proposed methodology application using the developed computer program, calculation was performed to determine the probability of exceeding boundary deformations in the example of an ordinary five-story brick building. Consequences (responsibility) class of the building are CC2, the responsibility category of the construction is A. Table 1 shows the deterministic values, in Table 2 – the probabilistic characteristics of the random variables normal distribution functions.

Figure 1 illustrates the function and density of random variables distribution N – the vertical load on the foundation and E – the modulus of the base soil deformation.



a)



b)

Figure 1 – Function and density of distribution of random variables:
a) ***N*** – vertical load on the foundation and b) ***E*** – deformation modulus of the base soil

Table 1 – Deterministic values

№	Name of the parameter	Designation	Units of measurement	Value
1	Width of foundation	A	m	1,0
2	Length of foundation	L	m	1,0
3	Depth of foundation	D	m	1,5
4	Brick wall thickness	H	m	0,51
6	Thickness of overlap	T	m	0,25
7	Height of the storey	H	m	3,0
8	Freight area	A	m^2	3,0
9	The average specific weight of the foundation and soil on its edges	γ_{mt}	kN/m^3	20,0
10	Load on the floor	Q	kN/m^2	10,0
11	Specific weight of backfill material	γ_I	kN/m^3	16,0
12	Specific weight of foundation soil	Γ	kN/m^3	18,0
13	Estimated soil resistance	R	kPa	314,057

Table 2 – Probabilistic characteristics of the normal distribution functions

№	Name of the parameter	Designation	Units of measurement	Probabilistic characteristics	
				mathematical expectation m	mean-square deviation σ
1	Specific weight of concrete	γ_b	kN/m^3	25,0	0,75
2	Specific weight of brickwork of walls and partitions	γ_{kk}	kN/m^3	18,844	0,517
3	Payload on the overlap	P_{pol}	kN/m^2	0,9	0,315
4	Snow load	S_m	kN/m^2	0,46	0,069
5	Soil deformation modulus	E	MPa	12,0	3,0
6	Average pressure under the base of the foundation	P_{ap}	kPa	233,276	28,82

Using the developed program, calculations were performed to determine the probability of base exceeding boundary deformation. The results of the calculations are shown in Fig. 2 and in Tables 3 and 4.

Table 3 – Statistical parameters of base boundary deformation random value distribution density

Parameters	Values
Mean value (mathematical expectation), m	0,049
The coefficient of variation	0,173
Maximum value, m	0,466
Minimum value, m	0,036

Table 4 – Results probability calculations of base exceeding boundary deformation

Name of the parameters	Values
The probability of base exceeding boundary deformation	$2x10^{-3}$
The permissible probability of foundation exceeding boundary deformation according to DBN B.1.2-14-2002	$1x10^{-4}$

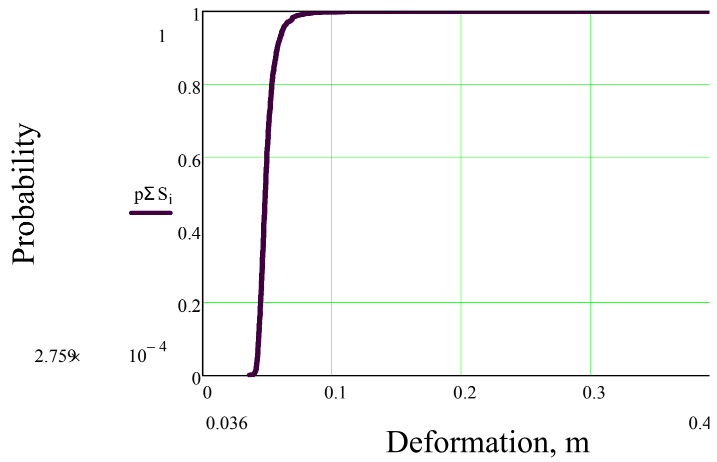


Figure 2 – Distribution function of the random value of deformations (s)

Conclusions. The method is proposed and the algorithm for solving the problem of determining the probability (risk) of exceeding the «base – building» system boundary deformation is developed within the framework of the parametric reliability theory using the method of statistical tests (Monte-Carlo).

A computer program that implements the proposed method of calculations has been developed.

Calculations have been performed to determine the probability (risk) of exceeding the boundary deformation of the «base-building» system for an ordinary residential brick building. The value of exceeding boundary deformation probability, equal to $2x10^{-3}$, is obtained. This value exceeds the normative, governed by DBN B.1.2-14-2009 (Table 3). The value of the deformations, determined by deterministic calculation according to the normative method, is 0,049 m, which is much less than the DBN B.2.1-10-2009 for this type of building (0,1 m). It can be stated that DBN B.1.2-14-2009 has a high level of reliability, and to satisfy this condition, the value of the deformations should be about 9 – 10 times lower than the normative one.

It is obvious that the optimal value of the risk should be in the range of $1x10^{-2}$ ... $5x10^{-3}$, which corresponds to the optimal reliability value of 0,99 ... 0,995 and agrees with the work of N.N. Mikheyev. In the normative document DBN B.2.4-3: 2010, the following values of the probabilities of accidents occurrence on pressure hydraulic structures of class CC2 $3x10^{-3}$ 1/year (for CC2-2) and $5x10^{-4}$ 1/year (for CC2-1), i.e for the entire service life (50 and 100 years correspondently) they should be not more than $15x10^{-2}$ and $5x10^{-2}$.

Analysis of probabilistic calculation shows the need to improve existing approaches to determining the probability of deformation exceeding boundary value, because new methods will improve the reliability of geotechnical problems.

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CONSTRUCTIONAL FEATURES OF HEGUMEN HOUSE IN CHERNIHIV

Research results of the XVII century construction, Hegumen House in Chernihiv, are presented. It is a historically significant local landmark. Attention was paid to historical data on foundation of the monastery and structure erection. The construction was built to serve as a refectory church of Peter and Paul. The purpose of the building was found to have been changed repeatedly during its existence. Constructional features of the structure remains were analyzed. Practical recommendations on its restoration were developed. Underground unit drawing was made, which is accessible to viewing. Detailed research was conducted. Reinforcements and brick wall construction, dated at a later period, were discovered. The wall construction brick was analyzed. Much attention was paid to durability and stableness of the underground unit.

***Keywords**: study, research, underground unit, foundations, constructional features.*

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КОНСТРУКТИВНІ ОСОБЛИВОСТІ БУДИНКУ ІГУМЕНА В ЧЕРНІГОВІ

Наведено результати дослідження споруди XVII ст. – будинку ігумена в Чернігові, пам'ятки архітектури місцевого значення. Приділено увагу історичним даним про заснування монастиря та зведення споруди, що будувалася як трапезна церква Петра і Павла. Виявлено, що за час існування призначення будівлі змінювалося неодноразово. Проаналізовано конструктивні особливості залишків каркаса будівлі з розробленням практичних рекомендацій щодо її відновлення. Виконано креслення підземелля, що доступні для огляду при виконанні робіт з детального обстеження. Виявлено місця підсилення й улаштування цегляних стін більш пізнього часу. Виконано дослідження цегли, що виявлена в конструкціях будівлі. Окрему увагу приділено надійності та стійкості підземної частини.

***Ключові слова:** обстеження, дослідження, підземелля, фундаменти, конструктивні особливості.*

Introduction. In 2013 locally listed building renovation project implementation work– the Abbot's former house in Chernihiv where were only ruins has begun. The Abbot's house is one of the last unconstructed buildings destroyed during the Second World War.

In October 2012 the Chernihiv Regional State Administration held a meeting and discussed the issues of survey, carrying out of research works, preparation of the project documentation for renovation, and in March 2013 at the regional council session the deputies decided to transfer the ownership of locally listed building – the «remains of the Abbot's house» – to Holy Assumption nunnery of Chernihiv diocese of the Ukrainian Church.

In 2013 on the territory of the Yeletsky monastery an archaeological research was carried out and work on the implementation of monument renovation project has began. The research expedition was attended by scientists of the National Architectural and Historical Sanctuary «Chernihiv Ancient».

Until now, not all works on the survey of the substructure, especially those rooms that can not be visited freely, have been completed.

Actual scientific researches and issues analysis. At the time of the survey only a part of the basement, which was used as a cellar for storing vegetables, was in operation and the rest of the building was almost completely destroyed. There are no publications related to the design features of the building or its performance characteristics, as well as any technical documents. The results of the survey are presented in the conferences of 2014 and 2015 [1 – 2].

Uninvestigated parts of general matters defining. Post–war remains of the building superstructure were neglected, no efforts to protect it from external factors were made. This resulted in the walls bearing capacity complete loss of the and overlap. The substructure began to be operated as a cellar, and, moreover, a cargo lift, which closed the passage to another part of the basement, was mounted in one of the basements.

The general condition of the building walls remains can be described as emergency one, in order to restore the building in its original form it is necessary to carry out complex research of the substructure, considering the increased pressure on the basis and foundations.

The objective of the article. The main objective of this research is to specify the reliability and firmness of the Abbot's house substructure, followed by development of recommendations for the superstructure renovation and, if necessary, for the strengthening of the building basis and foundations.

The statement of basic materials.

The architectural ensemble of **the Yeletsky monastery** is located on the elevated right bank of the Desna River, between the ancient Chernihiv citadel (the territory of the modern Val) and the monastery of Saint Elijah and the Holy Trinity. According to legend, in the middle of the 11th century Prince Svyatoslav Yaroslavich founded the Yeletsky monastery in connection with the appearance of God Mother icon on one of fir trees in this area. The date of this event is mentioned February 3, 1060 (Old Style). There is not much chronicle information about the monastery abode. It is known that in 1069 St. Anthony arrived in Chernihiv and founded the cave monastery of God Mother on the Boldyni Hills, in 1177 Ephraim the Abbot of the holy God Mother is mentioned.

In the middle of the 12th century on the spot, where the holy icon had appeared, a brick temple was erected in honor of God Mother Assumption.

In 1239 the Mongol–Tatars stormed Chernihiv. The city was burned down and robbed. The Yeletsky monastery suffered the same fate. It has been in desolation for a long time.

At the beginning of 16th century, Chernihiv fell under the power of Moscow. There is evidence that the ancient monastery was restored at that time. It was strengthened, and Moscow monks were settled in there. In 1611 the voivod Gornostay burned Chernihiv. After that the monks returned to Moscow. After 7 years the city fell under Poland.

The damaged and abandoned Assumption cathedral does not withstand the pressure of time. At first, the side chapters fell, and then the central dome fell too. After 1623 the Yeletsky monastery was renovated and handed over to the Uniates. In 1649 Chernihiv was liberated from the Poles and the Yeletsky monastery soon became Orthodox again.

Nowadays, the architectural ensemble of the Yeletsky monastery includes the Assumption Cathedral of the 12th century, the gate bell tower of the 17th century, refectory St. Peter and Paul church and the cells of the same period. Masonry fence and wooden residential house were built in 1688. Also, on the territory of the Yeletsky monastery a unique building of the 17th century, which in the documents of the Soviet era was called Abbot's house», was partially remained. It was built as a refectory church of St. Peter and Paul in 1676 at the expense of colonel Dunin–Borkovskyi. In 1861 the temple was moved to another premise, while the building accommodated the Chernihiv clerical consistory. During the time of its existence, the building function has changed several times. Herewith there was a redevelopment and architectural decoration was changed.

Built during the Hetmanate, the architectural structure was unique for its time – the largest refectory church on the Left Bank territory (Fig. 1). Its length without extensions and vestibules is more than 46 m and the width is at least 18 m. The wall thickness is more than half a meter, the height of one of the remained premises is about 5 m [3–4].

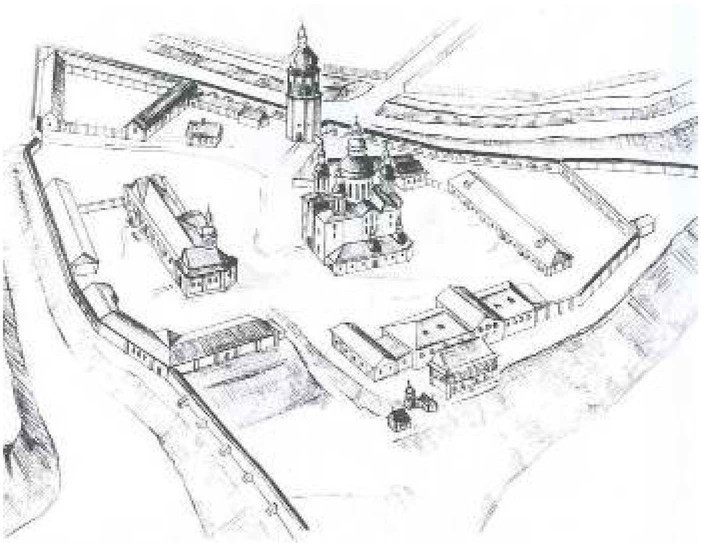

Figure 1 – Renovation of Yeletsky monastery (mid–eighteenth century) by O. Bondar

During the Second World War, the architectural structure was strongly damaged, and later the bricks were dismantled by the people for their own needs. Due to the intervention of the architect P.D. Baranovskyi, it was possible to preserve the remains.

The research of the Abbot's house was conducted with the assistance of the National Architectural and Historical Reserve «Chernihiv Ancient».

According to the brickwork method the construction lasted at least 150 years – there was expansion, extension, and redevelopment [5–6]. This is also evidenced by the brick technological features, recorded in at least five cases of research of the building remains of that era (17th–18th centuries).

The reasons that led to further building destruction in the postwar period may be the following: irresponsible attitude to the historical heritage, untimely response to deformation and initial destruction of bearing structures, significant moisture and weathering of the superstructure brickwork, which led to loss of building stability.

During the preliminary survey, structures measurements and building basement drawing were made (Fig. 2). In the course of measuring works, the location, bearing elements thickness, conditions and quality of structural elements joining, structures deformation, destruction, places of moisture and other defects were determined.

The amount of physical deterioration of the house elements was determined by visual inspection using the necessary devices. The physical deterioration of individual constructions was determined using tables by comparing the indicated attributes with those detected during the survey (Fig. 3) [7].

The general characteristic of the basement technical condition, which was available for inspection, can be estimated as «satisfactory» – the elements of the building in general are suitable for operation, but require renovation, which is most expedient at this stage. In some places cracks and fallout bricks are found. Walls are damp. At the entrance to the basement room, place of excessive moisture and water accumulation was found, which leads to the soaking of the basis.

The general characteristic of the superstructure technical condition can be estimated as «emergency» – the state of bearing structural elements remains needs to be restored. The overlap is destroyed; the wall brickwork has undergone significant destruction (brick and mortar removal, lintel fallout).

Over the period of its existence the extensions, fireplaces, renovations, reinforcement of structures, and other works were carried out in the house. Therefore, examined brick is varied in material and size. The clay brick of the sizes 35×19×6 cm and 26,5×12,6×6,0 cm is more often found.

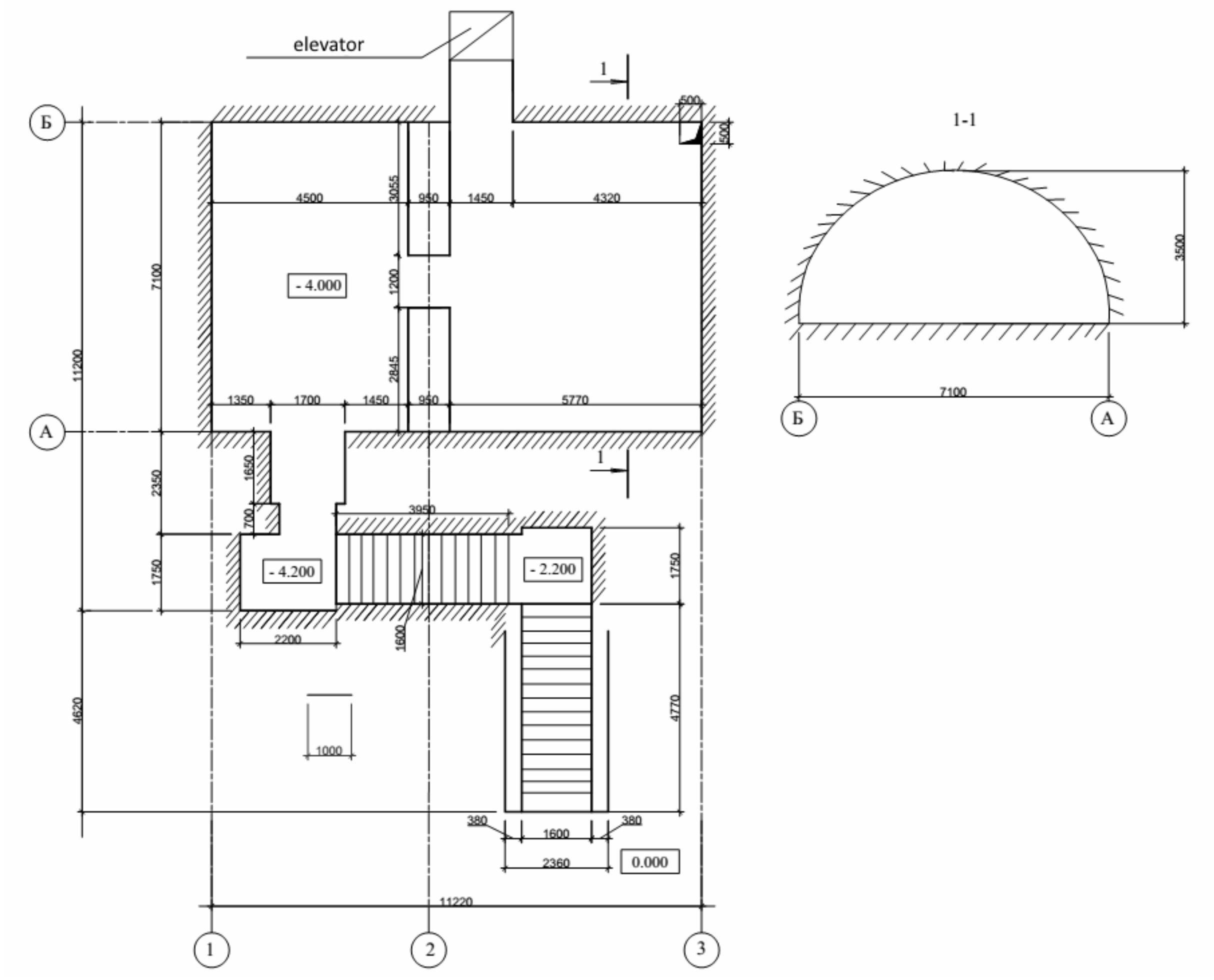

Figure 2 – Plan of the basement section, which was available for inspection

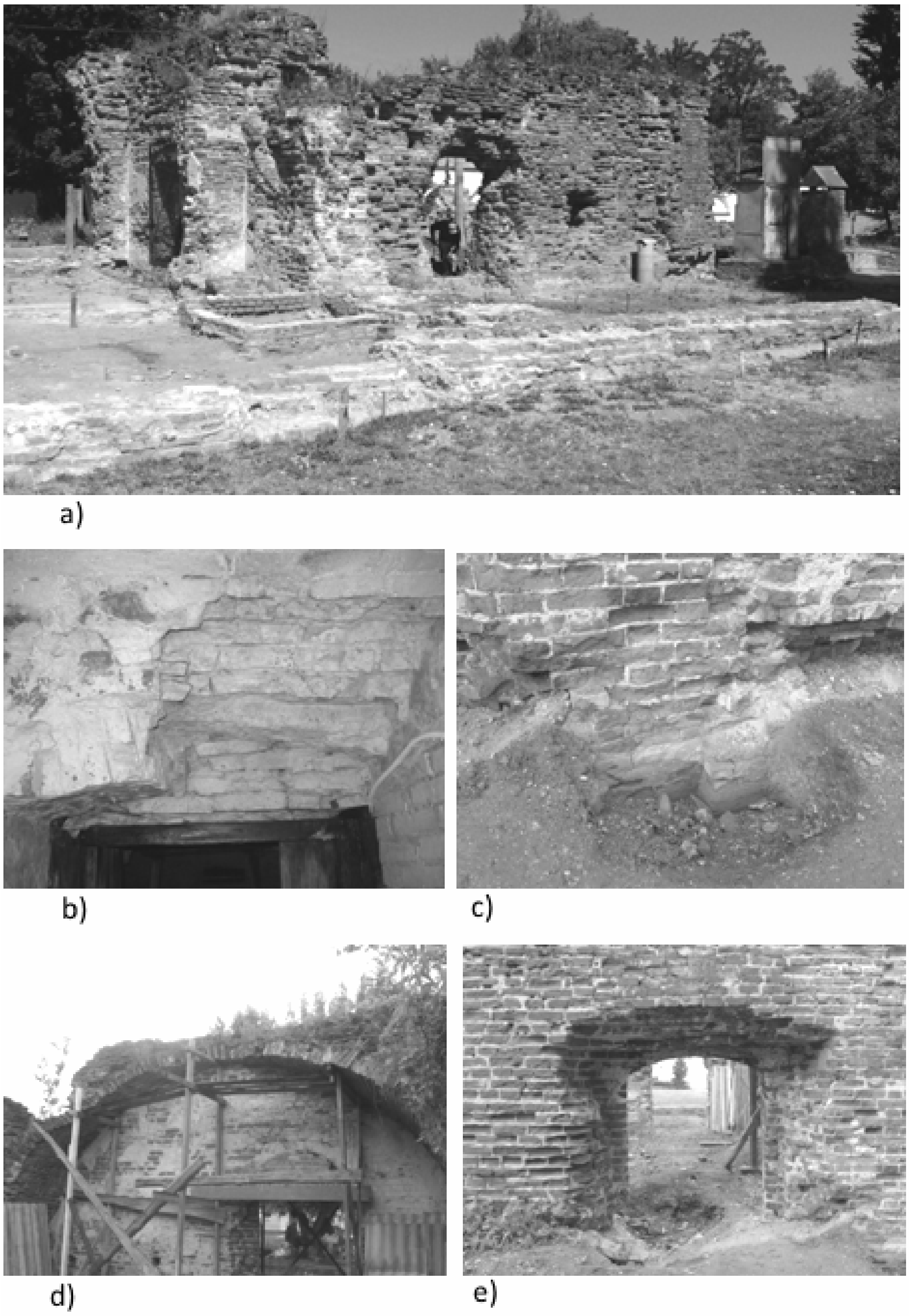

Figure 3 – House destruction:
a) – general view of the building superstructure remains;
b) – destruction of masonry lintel at the basement entrance;
c) – destruction of the house socle;
d) – temporary fastening of the overlap remains;
e) – destruction of the brick lintel and entrance posts.

Conclusions and suggestions.

1. There is no significant damage in the basement, but at the same time superstructure was vastly ruined. It is necessary to conduct detailed survey of basements that are not available for inspection, in connection with the installation of a cargo lift at the entrance to a part of the basement and to investigate the vault filled with earths that and limited to access.

2. Superstructure needs major renovation.

3. The primary objectives are:

– protection of existing structures from external influences (weathering, excessive humidification, freezing of structures) by arrangement of pavement around the building, installation of drainage and waterproofing systems, plastering and application of waterproofing solutions on the wall and socle elements, temporary roof installation;

– reinforcement of separate structure elements by metal overlays arrangement, tightening and concreting of weakened elements;

– execution of architectural drawings of the house facades aimed to renovate the house in its original form;

– execution of integrated project on historical building restoration.

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FILTRATION OF SALINE SOLUTIONS IN SOIL ENVIRONMENTS

The regularities of saline solutions concentration influence and their temperature on soil filtration characteristics have been experimentally investigated, substantiated and established. Significant difference between the filtration coefficient of saline solutions, considering their temperature and filtration of pure water was found out. So, at the concentration of saline solutions from 0 to 40…80 g/l the filtration rate increases sharply. With the further increase in the concentration of saline solutions, there is a phenomenon of soil softening, which leads to sharp decrease in their permeability, and, consequently, the coefficient of filtration. Such a pattern can be explained on the basis of physical and chemical processes occurring between mineral particles (solid phase) and saline solutions (liquid phase). In this case, it is got disperse system. Any disperse system tries to reduce its surface energy. In the system of «mineral particles + solution» it can be diminished by both reducing the total surface size and the liquid the surface tension.

***Keywords**: soils, filtration, concentration, saline solutions.*

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ФІЛЬТРАЦІЯ СОЛЬОВИХ РОЗЧИНІВ У ҐРУНТОВИХ СЕРЕДОВИЩАХ

Експериментально досліджено, обґрунтовано та встановлено закономірності впливу концентрації сольових розчинів та їх температури на фільтраційні характеристики ґрунту. Виявлено суттєву відмінність між коефіцієнтом фільтрації сольових розчинів з урахуванням їх температури та фільтрацією чистої води. Установлено, що при концентрації сольових розчинів від 0 до 40…80 г/л коефіцієнт фільтрації різко зростає. З'ясовано, що при подальшому збільшенні концентрації сольових розчинів спостерігається так зване явище осолонцювання ґрунтів, що призводить до різкого зменшення їх водопроникності, а відповідно і коефіцієнта фільтрації. Таку закономірність можна пояснити на основі фізико-хімічних процесів, які відбуваються між мінеральними частинками (твердою фазою) і сольовими розчинами (рідкою фазою) як дисперсної системи. Будь-яка дисперсна система намагається зменшити свою поверхневу енергію. Виявлено, що у системі «мінеральні частинки + розчин» може відбуватися її зменшення як за рахунок зниження величини сумарної поверхні, так і зменшення поверхневого натягу рідини.

***Ключові слова:** ґрунти, фільтрація, концентрація, сольові розчини.*

Introduction. Regularly, oil bases and structures during construction and operation fall into the effects of such an anthropogenic factor as saline solutions. The presence of saline solutions in the soil massif leads to changes in soil filtration, strength and deformation properties. The magnitude and intensity of such properties change may be significant, which leads to the occurrence of unforeseen additional deformations, strength stability disturbance of soil massifs etc. It can be complicated normal operation of buildings and structures, and in some cases leads to accidents in buildings and structures and can bring significant economic damage.

Analysis of recent research papers and publications. Water resistance of the same soils depends to a large extent on the chemical composition and concentration of the filtering fluid. Studies carried out by VS Sharov and BV Deryagin [1] showed that under the filtration of saline solution with a concentration of up to 10% due to montmorillonite clay, the filtration coefficient increased in 2 times compared with the filtration of pure water. Similar studies were carried out by us [2] for homogeneous quartz sand with a grain size of 0.25 mm fractions. The filtering fluid was a NaCl solution with a concentration of 0 to 16%. These studies showed that under the filtration of saline solution at concentration of 5%, the filtration coefficient reached its maximum value in comparison with the filtration rate of pure water. The results of the above experiments were limited only by the concentration of the filtering fluid. The temperature factor of saline solutions was not considered in details.

Among the recent works by foreign scientists devoted to the study of linear and nonlinear processes of groundwater dynamics, heat and mass transfer in homogeneous and heterogeneous soil environments, it is necessary to distinguish works [3 – 6] and other scientists. However, taking into account mathematical models of nonlinear processes that occur under the filtration of saline solutions in ground media requires their further development.

Highlighting of previously unsettled parts of the general problem. As it is known, the process of saline solutions' filtration in soil media is described by a rather complicated mathematical model. The complexity of this mathematical model lies in the fact that the presence of water in one or another of the associated components of salts affects the viscosity of the filtering saline solution, the permeability of the soil, and through them the coefficient of filtration k. Thus, the coefficient of filtration depends on both the properties of the soil medium and the properties of the filtering liquid. The complexity of this physical process has led to the fact that in many cases, for the simplification of the mathematical model which describes the processes of salts mass transfer, small and average concentrations of components contained in the filtration solution are taken. In addition, it is believed that the rate of solution filtration, that is its volumetric quantity, flowing in a unit of time, through a unit area at a given pressure gradient, does not depend on the change in the solution concentration.

However, the following experimental studies on saline NaCl solution filtration in sandy soils yield the fact that the filtration coefficient, and hence the rate of filtration, can vary considerably depending on the concentration of saline solution and its temperature.

Soil is a complex porous medium that is able to pass through itself the liquids, gases and their mixtures that are to be penetrating. The degree of permeability in different soils is different and is determined by their chemical and mineral composition, structural and texture features, the concentration of the filtering liquid, as well as the conditions under which the filtration takes place (the magnitude of the pressure gradient, temperature, etc.). The least studied and the most important these factors are filtered liquid properties.

Task setting. In order to study the parameters of saline solutions filtration in sandy soils and establishment of a quantitative estimation of their impact on the soils permeability, series of experiments determining the filtration coefficient based on the concentration of saline solution and its temperature were performed in the geotechnical laboratory of bases and foundations department of the National University of Water Management and Environment Engineering. In order to reduce the error of the experiment and the influence of various factors, experiments were conducted for homogeneous quartz sands Filtering fluid was a solution of NaCl with the concentration of 0 to 160 g/liter. Determination of the filtration coefficient of saline solution was performed on a standard KF-1 device. Preparation of the device and the studied soil, as well as carrying out the experiment itself, was fulfilled according to the standard procedure in accordance with State Standard of Ukraine [7].

Main material and results. The research results of the filtration coefficient depending on the concentration of the filter solution and its temperature for a single pressure gradient are given in table 1.

Table 1 – The results of filtration coefficient research depending on the concentration of filtrating solution and its temperature

C, g/l	Filtration coefficient *k*, m/day			
	t = 16 °C	*t* = 25 °C	*t* = 50 °C	*t* = 75 °C
0	0,20	0,22	0,78	0,80
20	0,30	0,32	0,96	1,00
40	1,20	1,43	1,80	1,88
60	0,40	0,50	3,24	3,50
80	0,38	0,43	2,24	4,90
100	0,40	0,44	1,22	4,00
120	0,39	0,44	0,96	1,80
140	0,39	0,45	0,80	1,60
160	0,40	0.47	0,60	1,60

The mathematical processing of the experiment results is carried out by applying multidirectional radial basis function, which is written in general form

$$k_{(c)} = \sum_{i=1}^{9} a_i \sqrt{1+(c-c_i)^2} \quad , \qquad (1)$$

where a_i – free members which are got in the process of statistical processing;

c – any meaning of saline solution concentration for filtration coefficient determination $k_{(c)}$;

c_i – experimental data of concentration in any research.

The mathematical processing results of experimental data in the most convenient way with the choice of optimal scale are shown on Fig. 1 – 4.

The presence and filtration of saline solutions in soil media, as shown by laboratory experiments, is accompanied by processes of interaction between liquid phases (solutions) and solid phase (mineral particles). These processes lead to changes in the composition and concentration of saline solutions and thus affect the rate of filtration (the coefficient of filtration). So, with increasing concentration of saline solution NaCl from 0 to 4 – 8%, the filtration rate increases sharply.

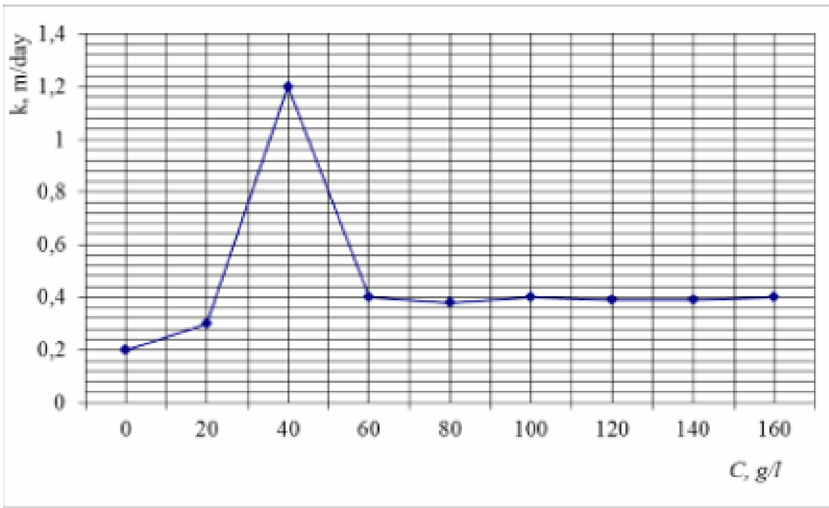

Figure 1 – Graphics of filtration coefficient dependency on the concentration of saline solutions under the temperature of t=16^{0}C

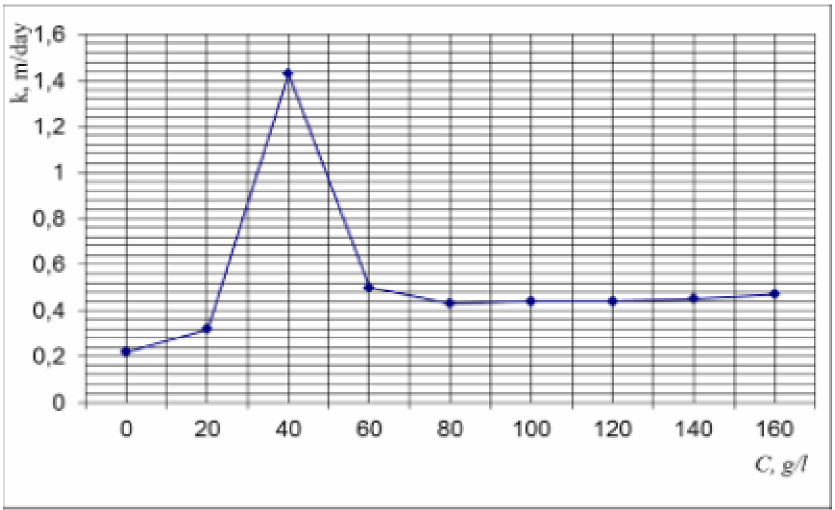

Figure 2 – Graphics of filtration coefficient dependency on the concentration of saline solutions under the temperature of t=25^{0}C

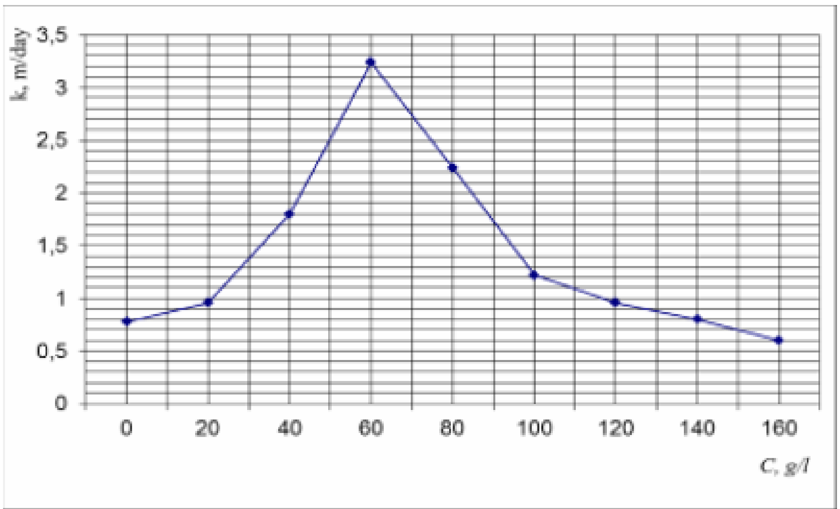

Figure 3 – Graphics of filtration coefficient dependency on the concentration of saline solutions under the temperature of t =50^{0}C

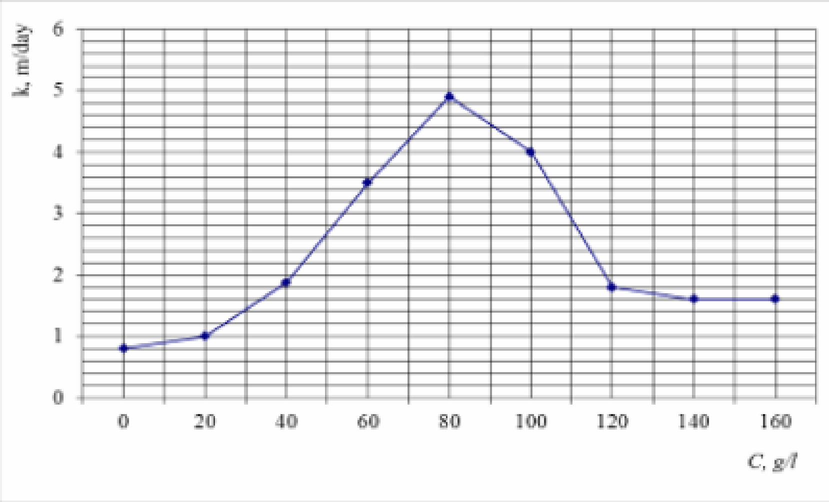

Figure 4 – Graphics of filtration coefficient dependency on the concentration of saline solutions under the temperature of $t=75^0C$

At a given concentration of saline solutions, the maximum value of the filtration coefficient was obtained, which increased by 2 – 4 times compared with the filtration of pure water. With the further increase in the concentration of saline solutions, there is a so-called phenomenon of soils salinization or alkali-affecting, which leads to a sharp decrease in their permeability, and, consequently, the coefficient of filtration. Such a pattern can be explained on the basis of physical and chemical processes occurring between mineral particles (solid phase) and saline solutions (liquid phase). In this case, we have a disperse system. The superficial energy of such a system is measured by the surface tension that occurs at the interface of the disperse phase with the dispersion medium and the magnitude of the total surface of all parts of the disperse phase. Any disperse system tries to reduce its surface energy. In the system of «mineral particles + solution» it can be reduced by both reducing the size of the total surface and reducing the surface tension of the liquid. The latter factor leads to the compression of the liquid diffusion layers, and, consequently, to an additional consolidation of soils and an increase in the coefficient of filtration.

Significant influence on the permeability and compressibility of soils is done by the composition of exchange cations. Thus, soils saturation with sodium ions causes marked decrease in their permeability and compressibility. Sharp decrease in the soil permeability and compressibility from presence of exchange cations is due to its dispersing effect. As a result, pore sizes and formation of a bound water significant amount in soil are reduced, which leads to decrease in the permeability of soils. Furthermore, it should be noted that in presence of saline solutions in disperse systems such as soils, in addition to two components, the solid disperse phase and the liquid disperse medium, there is always the third component, the electrolyte, which is the saline solution. When the salts are dissolved in water, the mineral particles of the soil do not remain passive to the molecules or ions of soluble salts. Dipole molecules of water, orienting in the force field of molecules or ions of soluble salt, form a densified hydration layer around it. Hydrated, or as they say in such cases, solid mineral particles are soaked. So, clay soils are very hydrophilic. Depending on the hydrophilicity degree, the surface retains hydration different amount, or, as it is called, bound water. The content of bound water in clay reaches 25%, which leads to decrease in soils permeability.

Conclusions. Obtained experimental data, as well as their statistical processing, made it possible to evaluate saline solutions filtration process in soil environments depending on filtering liquid properties quantitatively and qualitatively, on its concentration and temperature. Data of experimental researches can be used in assessing the soil bases and structures state by deformation and bearing capacity. Subsequent studies may be an assessment of the stress-strained state of soil bases and structures, considering nonlinear processes that occur during saline solutions filtration of varying temperature.

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DETERMINATION OF THE MAIN DESIGN PARAMETERS FOR SOIL CHEMICAL STABILIZATION

When designing or reconstructing buildings and structures on the swelling soils, it is necessary to be particularly concerned about the pattern of these soils behavior in order to subsequently be able to predict the behavior of the 'basement – foundation – building' system. The most effective way to stabilize soil behavior of the basement contaminated with industrial effluents is injection methods used for stabilization. The main task of soil chemical stabilization is to strengthen the bonds between soil particles with the help of chemical reagents. As a result of the research, the design parameters for foundations stabilization of soils composed contaminated with peroxy acid were determined. Depending on the concentration of peroxy acid in the soil and the density of the used sodium silicate solution, the limiting values initial components volume ratios were determined to carry out qualitative and reliable soil stabilization for the basements.

***Keywords:** silication, chemical swelling, peroxy acid.*

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ВИЗНАЧЕННЯ ОСНОВНИХ РОЗРАХУНКОВИХ ПАРАМЕТРІВ ХІМІЧНОГО ЗАКРІПЛЕННЯ ҐРУНТІВ

Обґрунтовано можливість прогнозування поведінки системи «основа – фундамент – споруда» при проектуванні чи реконструкції будівель і споруд на ґрунтах, що набухають. Визначено, що найбільш дієвим способом стабілізації поведінки ґрунтів основ, забруднених промисловими стоками, є ін'єкційні методи із закріплення. Основне завдання хімічного закріплення ґрунтів полягало в посиленні зв'язків між частинками ґрунту за допомогою хімічних реагентів.

У результаті досліджень були визначено розрахункові параметри закріплення основ фундаментів, складених ґрунтами, забрудненими пероцтовою кислотою. Залежно від концентрації пероцтової кислоти, яка знаходиться в ґрунті, та щільності застосовуваного розчину силікату натрію визначено граничні величини об'ємних співвідношень вихідних компонентів, що дозволяють здійснити якісне і надійне закріплення ґрунтів основ.

***Ключові слова:** силікатизація, хімічне набрякання, пероцтова кислота.*

Introduction. The operating experience of many enterprises has shown that due to the soaking of soils with industrial effluents in case of emergency the chemically active solutions of various concentrations enter the basement of buildings and significantly influence the properties of soils. Thus, upon the interaction of soil minerals with the solutions of acids and alkalis the physical and chemical processes and exchange reactions result in the increase of soil volume. This phenomenon was called the «chemical swelling». [1, 2]. The process has complex physical-and-chemical character and can lead to extremely negative consequences for both foundations and the «basement–foundation–building» system as a whole.

The main task of the soil chemical stabilization is to strengthen the bonds between soil particles using chemical reagents. There is a number of different ways of soil chemical stabilization. Some of them are very common and often used in construction, while others are used in rare, exceptional cases.

The Analysis of Recent Research and Publications. The problems of chemical swelling have been studied by many authors [3 – 5]. Previous studies have been aimed at studying the effect of chemical solutions, basement soil physical and chemical state, as well as latter structural properties [5, 6]. Chemical swelling in most cases leads to unacceptable deformations of foundations and supra fundamental structures of various industrial buildings and structures.

At the current stage of construction, various technology-related problems should be solved with due regard to the influence of the accepted decisions on the ecological state of environment. This issue is particular relevance to areas with a large share of chemical and food industries.

In the course of intensive economic development of territory areas the soils are exposed to various contaminants, both organic and inorganic. The impact of the former affects the composition, structure and properties of the soils. Different soils react differently to contaminants; some of them are more «sensitive» to them, while others are less sensitive. The most significant structural changes during contamination are typical of clay, loam, and partly – of sandy loam [5]. To a lesser extent the contaminants affect the structure of sandy soils and soils with coarse waste. The structure of rocky soils that is of magmatic, metamorphic and sedimentary cemented is the least subjected to the influence of contaminants.

The properties of contaminated soils are very different from those of the original soils. Various contaminants can affect physical properties of soils, changing the density, porosity, filtration properties, physical-and-chemical or physical-and-mechanical properties, etc.

At present in the food industry it is widely used the detergent and disinfectant Oxonia Active-150 based on the 25% peroxy acid. Due to dribbles and because of the sewage effluents this substance reacts with the chemically active substances of the soil when occurring in the basement resulting in the basement swelling.

When designing or reconstructing buildings and structures on the swelling soils, it is necessary to be particularly concerned about the pattern of these soils behavior in order to subsequently be able to predict behavior of the «basement–foundation–building» system.

The most effective way to stabilize the soil behavior of basement contaminated with industrial effluents are the injection methods used for soil stabilization [2, 7 – 9].

Stabilization of soils that are prone to contamination with industrial sewage is a complex of various measures that consider both technical and environmental issues. When carrying out work on stabilization of soils contaminated with industrial wastewater, the following requirements must be considered:

1) when carrying out work on stabilization of the contaminated soil mass it must be considered stabilization of section ecological condition, to minimize the introduction of chemicals necessary while work performing;

2) proposed method is supposed to ensure deformations stabilization in the «basement–foundation–building» system;

3) after the performance of work, that is after stabilization, the mechanical parameters and the parameters of deformation of the soil mass are supposed to be improved;

4) it is absolutely necessary to ensure the continuity of the contaminated soil mass stabilization process throughout its volume.

The prerequisite for choosing one or the other stabilization technologies is the observation of all the above requirements simultaneously. Is is the most optimal method for searching and selecting the best method of chemical stabilization of soil mass contaminated with industrial wastewater.

The Identification of Previously Unresolved Parts of the Problem. Most of the recent performed [1, 2, 5, 9] was devoted to the study of regularities of single swelling, i.e. swelling in a single humidification cycle. Regularities of cyclic swelling are studied considerably less, i.e. the behavior of soils with repeated moistening and drying. Meanwhile, basements soils of the industrial buildings where wet processes has variable regime of moistening.

The behavior of soils under these conditions is determined by the regularities of the cyclic swelling, but not of the single swelling. Cyclic moistening and drying lead to significant increase in the rates of swelling compared to single swelling.

Statement of the Task. The purpose of the study is to determine basic design parameters for the stabilization of foundations composed basements of soils contaminated with peroxy acid.

The Major Material and Results. The use of silicate and peracetic components for chemical stabilization of sandy and silt-and-clayey soils contaminated with industrial effluents of peroxy acid reduces to the injection of a mono-solution of sodium silicate into the soil.

The author studied the parameters influencing the swelling properties of soils from the action of organic acid. Dependences were found between the concentration of infiltrated acid and the amount of free swelling. However, the question of the effect of organic acids on the mechanical properties of both sandy and silty-clay soils remained unresolved.

From the results of the work performed, it can be seen that with an increase in the acid concentration from 0% (water) to 3%, the strain modulus decreases by 15% -40%, from the value in the natural state. Specific adhesion decreases by 25% - 60%, and the angle of internal friction decreases by 11% - 19%.

As for sandy soils, when soaking with acid solutions, a swelling pressure is observed. With an increase in the concentration of the infiltrated acid to 3%, 0.022 MPa increases. While with soaking water in sandy soils, swelling does not occur at all. For clay soils, swelling appears even at 0.5% acid and with an increase to 3%, the swelling pressure reaches 0.175 MPa.

After the research by the author on fixing contaminated soils, they acquire significant strength, mechanical characteristics are significantly improved. In some cases, the mechanical characteristics of the anchored soil exceed their values in the natural state.

R increases by 1.81 to 3.01 times, the specific cohesion C increases by 9.6 times, the strain modulus E increases by a factor of 2.48, the angle of internal friction increases by 1.56 times.

The parameters include:

– the concentration of peroxy acid in soil pores $\mu_к$,%;

– the design time interval for gelling $t_г$, min;

– the proportion of volumes of the original components Ω;

– the required volume of sodium silicate solution per 1 m^3 of the stabilized soil V_c;

– the coefficient of volume filling.

The soil mass with the raised acid content level is divided into separate sections which are stabilized according to individually calculated parameters. The lines of these sections are determined by the concentration of peroxy acid in the pores of the soil $\mu_к$. The calculations are carried out on the basis of the physical-and-chemical characteristics of soil with raised acid content level determined in laboratory conditions:

– $m_к$ is the mass of peroxy acid in 1 cm^3, in grams (determined by titration);

– m_W is water mass in 1 cm^3 of the soil with the raised acid content level, in grams (determined by the weighting method);

– n – porosity, %.

Having designated the weight of the peroxy acid solution in the pores of the soil m_r, it is possible to determine the concentration of peroxy acid in the pore solution μ_k from the proportion:

$$\mu_к = \frac{100 \cdot m_k}{m_r}, \%, \tag{1}$$

where m_r is the mass of the stabilizing solution;

considering that

$$m_r = m_k + m_w \ , \tag{2}$$

t is find

$$\mu_к = \frac{100 \cdot m_k}{m_k + m_r}, \% \ . \tag{3}$$

On the basis of the design parameter Ω_i, the formation of silicate peracetic gels with the formation time from 1 to 60 minutes, with the concentration of pore solutions of μ_k, is possible. Depending on the value of μ_k, separate sections for stabilization are identified.

The possibility of obtaining positive result when stabilizing the soils with raised acid content levels, the strength, density, continuity, the degree of ecological purity of the stabilized mass, as well as the cost and laboriousness of work performed depending on the chosen design time interval for gelling $t_г$. Depending on $t_г$, the number of sections with individual parameters, stabilization is determined, where the entire area with the raised acid content level is divided.

The required volume of sodium silicate solution V_c is calculated from the ratio:

$$\Omega = \frac{V_к}{V_c} \ . \tag{4}$$

To clarify the value of the required volume of sodium silicate solution in a liter per 1 m^3 of soil, the known function [5] is used which, considering the obtained dependences, becomes:

$$V_c = \frac{10^3 (m_k + m_w)^2}{\sqrt[b]{\frac{t_г}{a}} (1.58 m_k + m_w)}, \tag{5}$$

In Figure 1 the graphical scheme for determining the limiting acid concentrations of the pore solutions at individual sections of the stabilized soil mass $\mu_к$ and the volumetric ratios of the initial components involved in the gelling reaction Ω is given.

It has been established that with the increase in the concentration of peroxy acid in pore solutions the required density of sodium silicate solutions increases.

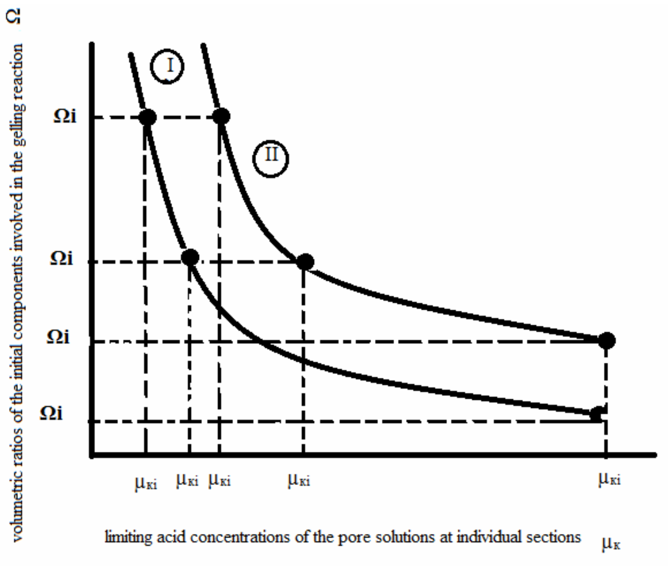

Figure 1 – Determination of the stabilized sections and the parameter *Ω* lines:
I – for gels with $t_г$ <10 min, II – for gels with $t_г$ > 10 min

Table 1 – The recommended values of volumes ratios of the initial components for the intervals of peroxy acid concentration in the individual stabilized sections are shown

The stabilized section number	The concentration of peroxy acid in pore solutions, μ_K, %	The density of sodium silicate, ρ_c	The design of the initial components ratios value, Ω
1	1,0	1,10	3,89 – 8,5
2		1,15	4,25 – 9,76
3		1,20	7,5 – 13,5
4		1,25	10,6 – 17,5
5	2,0	1,10	3,5 – 8,92
6		1,15	5,02 – 10,6
7		1,20	7,96 – 14,49
8		1,25	10,57 – 17,93
9	3,0	1,10	4,36 – 9,02
10		1,15	5,69 – 11,37
11		1,20	8,5 – 16,52
12		1,25	12,63 – 18,34

Considering required volume of sodium silicate and its density are different for different sections of stabilization and for different types of soils when stabilizing the sections of soils with the raised acid content levels according to the method it is recommended to use drilling-and-mixing or jetting technologies for delivering soil stabilizing solutions [4, 6].

Conclusions. As a result of the research, the design parameters for stabilization of soil foundations composed contaminated with peroxy acid were determined. Depending on the concentration of peroxy acid in the soil and the density of the used sodium silicate solution, of volumes limiting values ratios of the initial components were determined allowing to carry out qualitative and reliable soils stabilization for the basements.

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COMPRESSIBLE THICKNESS DEPTH DETERMINATION UNDER DIFFERENT CALCULATION METHODS OF THE SETTLEMENTS ACCORDING TO NATIONAL AND EUROPEAN STANDARDS

The comparison of the compressible thickness values and the settlements values according to the cone penetration test data in accordance with the current regulatory documents of the Belarus Republic and EUROCODE 7 «Geotechnical design» (part 1, 2) is presented. Two calculation methods of the foundation settlement for the limiting state of SLS and two methods for calculating of the settlements according to National standards are considered according to European norms. The ratios of the compressible thickness and the upset distances are determined using different calculation methods according to European and National standards. The proportions of the compressible thickness and the settlements values for different types of foundations are determined according to European and National standards.

***Keywords:** compressible thickness, cone penetration test, foundation settlement, method of layer by layer summing up, method of equivalent layer, Schmertmann's method.*

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ВИЗНАЧЕННЯ ГЛИБИНИ СТИСЛИВОЇ ТОВЩІ ПРИ РІЗНИХ МЕТОДАХ РОЗРАХУНКУ ОСІДАНЬ ЗА НАЦІОНАЛЬНИМИ ТА ЄВРОПЕЙСЬКИМИ НОРМАМИ

Наведено порівняння величин стисливої товщі й величин осідань за даними статичного зондування згідно з чинними нормативними документами Республіки Білорусь і EUROCODE 7 «Geotechnical design» (part 1, 2). Розглянуто відповідно до європейських норм два методи розрахунку осідань фундаменту для граничного стану SLS і два методи розрахунку осідань за національними нормами. Визначено співвідношення величини стисливої товщі й величини осідань при різних методах розрахунку за європейськими і національними нормами. Установлено співвідношення величини стисливої товщі та величини осідань для різних типів фундаментів за європейськими і національними нормами.

***Ключові слова:** стислива товща, статичне зондування, осідання фундаменту, метод пошарового підсумовування, метод еквівалентного шару, метод Шмертмана.*

Introduction. At present, the design of buildings and structures foundation is based on the foundation bases calculations under limit states. In many cases, it is sufficient to use the methods of calculation based on the simplest models of soil behavior under load: the theory of limit equilibrium - for calculations taking into consideration the bearing capacity and the theory of linear deformation – for calculations according to deformations. However, in a number of cases, such approach leads to excessive reserves when the buildings' design or, in particularly difficult cases, it is insufficient for project objectivities.

According to Vynnykov Yu. L. [13] they point out the following models of: linearly deformable surroundings, nonlinearly elastic, the theory of limit equilibrium; elastic-plastic surroundings and the plastic flow theory; and those that are based on the concept of critical state of ground coat, etc.

In models of linearly deformable surroundings [1 – 4, 13], apart from one batch of ground coat and linear dependence between stresses and deformations under load, only its general deformation is considered not specified by elastic and plastic components. The first assumption makes it possible to use the elasticity theory when stress calculations in the solid, and the second, having the known stresses, makes it possible to determine the finite deformations of the primary structure.

The question of present interest in the field of geotechnics is to determine the value of the compressible thickness of the foundation base. The depth of the compressible thickness is one of the boundary conditions on which the value of the foundation settlement depends.

The depth of the compressible thickness depends on many factors [15]:

– width of the bottom of foundation, shape, rigidity, depth of its laying;
– pressure on the bottom of foundation and its depth distribution;
– ground condition and properties;
– groundwater level;
– growth rate of excess pressure at various depths;
– values of structural strength of soil etc.

The effect of the strength characteristics of the soil on the amount of the compressible thickness will be visible under loads that exceed certain level, when the value of the compressible thickness ceases to depend on the loads acting on the foundation base. As in the National and European normative documents [1 – 4] it is envisaged to apply the theory of linear deformation wherein the average pressure under the base of the foundation should not exceed the limit when the force deformation relationship $S = f(p)$ is close to linear, so the depth of compressible thickness will not depend on the strength characteristics of the foundation base.

Konovalov P.A. [12] divides the determination various calculation methods of soil thickness into three groups. The first group includes compressible foundation thickness determination methods without considering settlement calculation the compression of those frost blanket courses where the additional vertical pressures from the loads on the foundations constitute a certain constant fraction of K from these layers natural pressure on the roof.

The second group includes the methods where the value of compressible thickness depends on the ratio of layers deformations at its boundary. The third group includes the techniques that cannot be referred to the first and the second group.

There is no single method to determine the depth of the compressible thickness either in National or European normative documents [1 – 4]. In the most methods of calculation of the settlement, the depth of the compressible thickness is limited by that soil layer where the deformations are insignificant and, therefore, they can be not considered.

To calculate the foundation settlement and to determine the depth of the compressible thickness in National normative documents three methods are presented [1, 2]:

1) the method of layer by layer summing up using the computational scheme of linearly deformable half-space, limited by the conditional depth of the compressible thickness H_c;

2) the finite thickness linearly deformable layer method;

3) the equivalent layer method.

In the method of layer by layer summing up, the lower boundary of the compressible foundation's thickness is limited by the depth $z = H_c$, on the ground of the following conditions:

a) when $b \leq 5$ m – $\sigma_{zp} = 0{,}2p_{zg}$;

b) when $b > 20$ m – $\sigma_{zp} = 0{,}5\ p_{zg}$;

c) when $5 < b \leq 20$ m – σ_{zp} are taken according to the linear interpolation of values $0{,}2\ p_{zg}$ and $0{,}5\ p_{zg}$,

where σ_{zp} – is the additional vertical normal stress at depth $z = H_c$;

p_{zg} – is a vertical pressure of sole weight;

b – is the width of foundation bottom.

If within depth H_c, determined according to the above mentioned conditions, a layer of soil with a strain modulus $E > 100$ MPa underlays, so the thickness of the compressible layer is taken up to the top roof of this soil.

If the lower boundary of the compressible thickness is in a layer of weak soil with a strain modulus $E < 5$ MPa or such layer lies beyond of the specified boundary at depth not exceeding the width of the foundation b, the found value H_c increases up by the thickness of this layer, and the most minimum value is taken for H_c, corresponding to the weak underlying bedrock or the depth where the condition $\sigma_{zp} = 0{,}1p_{zg}$ is satisfied.

When using the calculation of settlement by the equivalent layer method, on the assumption of the triangular diagram of the pressure distribution at the foundation base, the depth of the compressible thickness is determined by the formula 5.48 [2]:

$$H = 2 \cdot h_э = 2 \cdot A\omega \cdot b \tag{1}$$

In this case, the depth of the compressible thickness does not depend on the load transferred to the foundation.

In Eurocode 7 [3, 4] there is no unified approach in determination of foundation bases settlements. General requirements and recommendations are provided to determine the ultimate operational states and limit values of foundation displacements. Appendix F describes the simple analytical methods for the calculation of the settlements [3]. In Eurocode 7 part 2 [4] four methods are given (although they are not given in Eurocode 7 part 1 [3]) to calculate the settlements, based on the results of field tests using semi-empirical calculation models (Appendix B2, C2 and D4 [4]). The use of this particular method is usually specified in National Annex to Eurocode.

The general definition of the compressible thickness is given in [3] and does not depend on the foundation settlement calculation method. Usually this depth is taken as effective vertical stresses from the foundation make up 20% of the stresses from the external load. But in many cases this depth is assigned approximately equal to one or double width of the foundation, but this depth can be reduced for lightly loaded wide foundation plates.

Analysis of recent achievements and publications. In the lectures [5 – 7], the examples of subsurface settlement calculation are given. In the work [9] the example of subsurface foundation settlement calculation is given considering consolidation on a multilayer foundation and determination of compressible thickness depth. The authors [8, 10] explain and comment on the articles of Eurocode 7 containing new approaches to design, give the examples of foundation settlements calculation according to European standards. The authors [12, 14 – 15, 17 – 19] give examples of compressible thickness determination using the different calculating of the settlement calculating methods.

Identification of the previously unsolved parts of a common problem. In spite of the increased interest of well-known scientists in the selected problems, compressible base thickness depth determination according to National and European norms continues is relevant. These issues remain undisclosed that requires their further development.

The object of the work is the following – comparison of the results of the compressible thickness values and the foundation settlement values according to cone penetration test data in accordance with National and European design standards.

The basic part. Different methods of determining of compressible base thickness value correspond to one or another technique for calculation of the final foundation settlement. This article discusses the settlement determination method according to the data of cone penetration tests in sandy soil and silty-clayed soil.

Stage 1. Determination of the compressible thickness value when calculating the subsurface foundation settlement in sandy soil.

We are going to consider the simplest case of subsurface interaction foundation with the homogeneous soil base. It is restricted by the problem of final stabilized foundation settlement definition due to the load action transmitted to the soil through the foundation base.

To calculate the foundation settlement, the results of cone penetration test within Vitebsk region, the Republic of Belarus were admitted in this work.

The soil – is sand of medium size, medium strength. Calculation values of soil characteristics:

q_s = 6,53 MPa, E = 26,85 MPa, γ_{II} = 18,8 kn/m^3, c_{II} = 0,001 MPa, φ_{II} = 35,5^o .

Pressure on the base P=100, 150, 200 kPa (given conditionally).

1.1. The determination of compressible thickness depth and the calculation of the pad foundation settlement according to National and European standards.

Pad foundation with the dimensions in terms 3×3 m. The embedment depth is of 2 m.

For the comparative analysis of the compressible thickness determination and the settlement value, three methods for settlement calculation were chosen: the method of layer by layer summing up [3], the equivalent layer method [3], and the Schmertmann’s method Appendix D.3 [4]. The final foundation settlement S using the design scheme in the form of a linearly deformable half-space with conditional restriction of the compressible thickness determined by the method of layer by layer summing up according to the formula:

$$S = \beta \cdot \sum_{i=1}^{n} \frac{\sigma_{zp,i} \cdot h_i}{E_i}, \tag{2}$$

where β – is the nondimensional coefficient, equal to 0,8;

$\sigma_{zp,i}$ – is the average value of the additional vertical stress in the i-th elementary layer of the soil, equal to half of the sum of the stresses at upper and lower boundaries of the i-th elementary layer, kPa;

h_i and E_i – are respectively, the thickness and the modulus of deformation of the i-th elementary layer of soil. The thickness of the layer h_i should not exceed 0.4 times of the foundation width;

n – is the number of layers where the compressible thickness of the soil is divided.

Table 1 – Results of the foundation settlement according to the formula (2)

Pressure on foundation kPa	100	150	200
Value of the settlement according to the formula (2), m	0,004	0,009	0,012
Value of the compressible thickness, m	3,17	4,14	4,83

The final impaction settlements (maximum final – for flexible and medium – for rigid foundations), when using the equivalent layer method, are determined using the theory of filtration consolidation on the assumption that the base is a linearly deformable body, according to the formula:

$$S = h_э \cdot m_v \cdot p_0, \tag{3}$$

where m_V – is a coefficient of relative compressibility of the ground coat of a homogeneous base,

p_0 – is an additional pressure at the level of the bottom of foundation, MPa;

$h_э$ – is the depth of the equivalent layer, m, is determined by the formula:

$$h_э = A_\omega \cdot b, \tag{4}$$

where A_ω – is the coefficient of the equivalent layer, accepted depending on the soil type and the shape of the foundation bottom, (table 5.14 [2]);

b – is the width (diameter) of the foundation, m.

Table 2 – Results of the foundation according to the formula calculation (3)

Pressure on base of foundation, kPa	100	150	200
Value of the settlement according to the formula (3), m	0,0085	0,015	0,022
Value of the compressible thickness, m	5,94	5,94	5,94

The third method to determinate the foundation settlement according to the data of cone penetration test is the use of the Schmertmann's method [4, 11] when calculation. The foundation settlement s due to the load q is determined by the formula:

$$s = C_1 \cdot C_2 \cdot \left(q - \sigma'_{v0}\right) \cdot \int_0^{z1} \frac{I_z}{C_3 \cdot E'} dz, \tag{5}$$

where $C_1 = 1 - 0{,}5 \cdot \left[\sigma'_{v0} / \left(q - \sigma'_{v0}\right)\right]$, $C_2 = 1.2 + 0.2\, lg\, t$,

C_3 – is a correcting coefficient depending on the foundation shape (for square foundations C_3 = 1,25 for square foundations, 1,75 for spread foundations) ,

σ'_{v0} – is the initial effective vertical stress at the level of the bottom of a foundation,

t – is the time, year,

I_z – is the factor of the influence of stress,

E' – is the Young's modulus, E' = 2,5 q_c for square foundations, E' = 3,5 q_c for spread foundations

Table 3 – Results of the calculation of the foundation according to the formula (5)

Pressure on base of foundation, kPa	100	150	200
Value of the settlement according to the formula (5), m	0,0048	0,011	0,0165
Value of the compressible thickness, m	6,0	6,0	6,0

1.2. The compressible thickness depth determination and the calculation of the settlement of raft foundation according to National and European standards.

The raft foundation with the dimensions in terms is of 12×12 m. The embedment depth is of 2 m.

Table 4 – The calculation foundation design results according to the formulas (2, 3, 5)

Pressure on base of foundation, kPa	100	150	200
Value of the settlement according to the formula (2), m	0,009	0,021	0,031
Value of the compressible thickness according to the formula (2), m	4,23	6,43	8,32
Value of the settlement according to the formula (3), m	0,034	0,061	0,089
Value of the compressible thickness according to the formula (3), m	23,76	23,76	23,76
Value of the settlement according to the formula (5), m	0,0126	0,028	0,044
Value of the compressible thickness according to the formula (5), m	24	24	24

1.3. The compressible thickness depth determination and the calculation of the settlement of the spread foundation according to National and European standards.

The spread foundation with the dimensions in terms 3×30 m. The embedment depth is of 2 m.

Table 5 – Results of the foundation calculation according to the formulas (2, 3, 5)

Pressure on base of foundation, kPa	100	150	200
Value of the settlement according to the formula (2), m	0,006	0,014	0,021
Value of the compressible thickness according to the formula (2), m	4,5	6,44	7,84
Value of the settlement according to the formula (3), m	0,02	0,037	0,053
Value of the compressible thickness according to the formula (3), m	14,28	14,28	14,28
Value of the settlement according to the formula (5), m	0,0043	0,0097	0,0153
Value of the compressible thickness according to the formula (5), m	12	12	12

<u>Stage 2. Determination of the compressible thickness when the calculation of the subsurface foundation settlements in silty-clayed soil.</u>

The soil – is the stained low-plasticity sand clay having the average strength,. Calculated values of ground coat characteristics: q_s = 1,78 MPa, E = 6 MPa, γ_{II} = 21,4 kH/m^3, W_L = 23,02%.

Pressure on foundation P = 100, 150, 200 kPa (given conditionally).

2.1. The determination of the compressible thickness depth and the calculation of the pad foundation settlement according to National and European standards.

The pad foundation with the dimensions in terms 3×3 m. The embedment depth is of 2 m.

The foundation settlements can be determined according to European standards using different formulas. In this case, the characteristics of the soil were obtained with the help of cone penetration test and in [5] it is said «both semi-empirical and analytical methods of calculation should be used».

The total settlement of the subsurface foundation in silty-clayed soil

$$S_d = S_e + S_c. \tag{6}$$

where S_e – is the immediate settlement according to the formula (7),

S_c – is the soil consolidation settlement according to the formula (8).

For calculation it is applied the formula of the immediate settlement Annex F [4].

$$S = \frac{P \cdot b \cdot f}{E_m} \tag{7}$$

where P – is the pressure imposed to the soil;

E_m – is the design value of the Young's modulus of elasticity. The ratio between the odometer module and the cone resistance is shown in the table D2 [4];

b – is the width of the foundation bottom;

f – is the coefficient of the settlement:

The formulas for the consolidation settlements of the subsurface foundations calculation according to the data of CPT for silty-clayed soil are not available in [3, 4], however, it is suggested to use the formula given in [16]:

$$\Delta H = \sum_{1}^{n} H_1 \left(\frac{C_c}{1+e_1} \right) \cdot \log_{10} \left(\frac{P_1^{/} + \Delta \bar{P}_{1-2}}{P_1^{/}} \right) \tag{8}$$

where C_c – compression index;

n – no. of compressible sublayers used;

e_1 – is the initial void ratio;

$P^{/}_1$ – is the initial vertical effective stress (fig.1.);

$\Delta \bar{P}_{1-2}$ – is the expected increase in stress (fig.1);

H_1 – is the initial thickness (fig.1).

The pressures distribution method under the bottom of foundation «2:1» is compared fairly well with the theoretical Boussinesq method at depth of the compressible thickness from B to 4B, but it cannot be applied at depth of the compressible thickness from 0 to B [18].

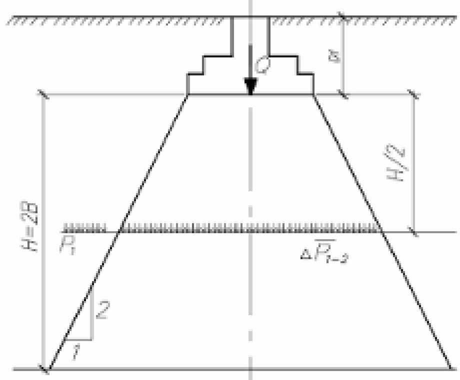

Figure 1 – Distribution of pressures in soil body using the method 2:1

As the depth of the compressible thickness is not regulated in this settlement calculation, so according to [3] it is taken the value of the compressible thickness equal to double width of the basement.

Table 6 – Results of foundation calculations according to the formulas (2, 3, 6)

Pressure on foundation, kPa	100	150	200
Value of the settlement according to the formula (2), m	0,016	0,034	0,054
Value of the compressible thickness according to the formula (2), m	2,86	3,85	4,54
Value of the settlement according to the formula (3), m	0,031	0,058	0,085
Value of the compressible thickness according to the formula (3), m	5,64	5,64	5,64
Value of the settlement according to the formula (6), m	0,092	0,133	0,171
Value of the compressible thickness according to the formula (6), m	6	6	6

2.2. The compressible thickness depth determination and the calculation of the foundation slab settlement according to National and European norms.

The foundation slab with the dimensions in terms 12×12 m. The embedment depth is of 2 m.

Table 7 – Results of foundation calculations according to the formulas (2, 3, 6)

Pressure on base of foundation, kPa	100	150	200
Value of the settlement according to the formula (2), m	0,027	0,077	0,138
Value of the compressible thickness according to the formula (2), m	3,46	5,97	7,96
Value of the settlement according to the formula (3), m	0,124	0,232	0,34
Value of the compressible thickness according to the formula (3), m	22,56	22,56	22,56
Value of the settlement according to the formula (6), m	0,371	0,531	0,69
Value of the compressible thickness according to the formula (6), m	24	24	24

2.3. The compressible thickness depth determination and the calculation of the spread foundation settlement according to National and European norms.

The spread foundation with the dimensions in terms 3×30 m. The embedment depth is of 2 m.

Table 8 – Results of foundation calculations according to the formulas (2, 3, 6)

Pressure on base of foundation, kPa	100	150	200
Value of the settlement according to the formula (2), m	0,023	0,051	0,084
Value of the compressible thickness according to the formula (2), m	3,96	5,86	7,31
Value of the settlement according to the formula (3), m	0,074	0,139	0,205
Value of the compressible thickness according to the formula (3), m	13,56	13,56	13,56
Value of the settlement according to the formula (6), m	0,174	0,248	0,32
Value of the compressible thickness according to the formula (6), m	6	6	6

Results of the performed researches.

Table 9 – The compressible thickness value ratio according to National and European norms

Foundation type, calculation method	Pressure, kPa		
	100	150	200
Sandy soil			
Pad foundation, formula 2+5	0,53	0,69	0,805
Pad foundation, formula 3+5	0,99	0,99	0,99
Spread foundation, formula 2+5	0,375	0,537	0,653
Spread foundation, formula 3+5	1,19	1,19	1,19
Raft foundation, formula 2+5	0,176	0,268	0,347
Raft foundation, formula 3+5	0,99	0,99	0,99
Clay soil			
Pad foundation, formula 2+6	0,477	0,642	0,757
Pad foundation, formula 3+6	0,94	0,94	0,94
Spread foundation, formula 2+6	0,66	0,977	1,218
Spread foundation, formula 3+6	2,26	2,26	2,26
Raft foundation, formula 2+6	0,144	0,248	0,332
Raft foundation, formula 3+6	0,94	0,94	0,94

Table 11 – The settlement value ratio according to National and European norms

Foundation type, calculation method	Pressure, kPa		
	100	150	200
Sandy soil			
Pad foundation, formula 2+5	0,833	0,818	0,727
Pad foundation, formula 3+5	0,177	1,36	1,33
Spread foundation, formula 2+5	1,39	1,443	1,37
Spread foundation, formula 3+5	4,65	3,81	3,46
Raft foundation, formula 2+5	0,714	0,75	0,704
Raft foundation, formula 3+5	2,69	2,17	2,02
Clay soil			
Pad foundation, formula 2+6	0,174	0,256	0,316
Pad foundation, formula 3+6	0,337	0,436	0,497
Spread foundation, formula 2+6	0,132	0,206	0,262
Spread foundation, formula 3+6	0,425	0,56	0,64
Raft foundation, formula 2+6	0,072	0,145	0,2
Raft foundation, formula 3+6	0,334	0,437	0,493

Conclusions:

1. For spread foundations and raft foundations on a sandy base the value of the compressible thickness by the equivalent layer method [2] and by the Schmertmann's method [4, 11] is slightly different, however, in this case the value of the settlement increases more than 2 times for raft foundation, and 3.5 - 4 times spread foundation. Therefore, it is advisable to develop National tables to determine the coefficient that will depend on the value of the compressible thickness and the dimensions of the foundation in the terms.

2. There is no universal approach for determination of the compressible thickness value in European normative documents. Therefore, the designer should choose the value of the compressible thickness considering the recommendations given in the normative documents [3, 4] and in the literature sources given after each section in [3, 4].

3. Using the Schmertmann's method to determinate the settlement for silty-clayed soil [4, 16] it is required to accept the bulk of the compressible thickness for spread foundations and raft foundations within 1 – 1,5 of the width of the foundation base, then there will not be such divergence in the value of settlements according to National and European standards

4. It should be noted that in this article the compressible thickness value and the settlement values determination method according to data of the cone penetration test were considered and, accordingly, when calculation of the settlement by other methods, the compressible thickness values ratio and the settlements values will differ from the above mentioned values.

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ACTUAL DESIGN MODEL DEVELOPMENT OF PROTECTIVE SCREEN OVER THE ACTIONS OF SUBWAY LINES

It is given calculation model development results of protective shield overexisting tunnels of subway in influence zone of transport interchange construction in the city of Minsk. The analysis of modeling problems of the system «construction - soil massif» is performed. It is proposed the approach to development of an adequate design model for complex engineering-geological conditions. An adequate calculation model of the transport interchange site was developed. The applying modern methods results of calculating the system «construction – soil massif» are given. The stress-strain state nature of the tunnel structures at all stages of the transport interchange construction is analyzed. The results of the calculation are compared with the data obtained during testing of the protective screen in full-scale conditions. The special practical significance of using such calculation methods at the design and construction stage of transport facilities is noted.

***Keywords:** transport facility, calculation model, soil, deformation, stresses.*

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РОЗРОБЛЕННЯ АКТУАЛЬНОЇ РОЗРАХУНКОВОЇ МОДЕЛІ ЗАХИСНОГО ЕКРАНА НАД ДІЮЧИМИ ЛІНІЯМИ МЕТРОПОЛІТЕНА

Подано результати розроблення розрахункової моделі захисного екрана над діючими тунелями метрополітену, які потрапляють у зону впливу будівництва транспортної розв'язки в м. Мінськ. Виконано аналіз проблеми моделювання системи «споруда – ґрунтовий масив». Запропоновано підхід до розроблення коректної розрахункової моделі споруд, що зводяться в складних інженерно-геологічних умовах. Наведено основні результати застосування сучасних методів розрахунку системи «споруда – ґрунтовий масив».. Проведено аналіз напружено-деформованого стану конструкцій тунелів на всіх етапах будівництва транспортної розв'язки. Виконано співставлення результатів розрахунку з даними, що отримані при випробуванні захисного екрану в натурних умовах. Відмічено особливу практичну цінність застосування таких методів розрахунку на стадіях проектування та будівництва транспортних споруд.

***Ключові слова:** транспортна споруда, розрахункова модель, ґрунт, деформація, напруження.*

Introduction. When calculating any structures, especially in complex engineering and geological conditions, it is necessary to consider the interaction of the structure and the foundation soils. For various reasons, designers pay great attention to modeling directly constructions of new structures, and their interaction with the ground is treated in a simplified manner.

To assess construction impact of transport facilities on existing structures, it is necessary to perform calculations using modern settlement systems. The use of general-purpose program complexes working on the finite element method for modeling soil work is the most common mistake. Typically, such programs use elastic models that are not capable to describe correctly soils work. As for the stress-strain state of the soil when applying the load, it cannot be adequately described in standard construction programs for calculation. Proper numerical simulation should be performed using specialized geotechnical software.

Knowing the reasons for the negative impact of new construction on existing structures, it can be minimized their possible deformations. This task solution is carried out by creating a computational model of new construction impact on the surrounding building, generalizing the experience of building analogues.

Use of a set of finite element analysis programs MIDAS and SOFiSTiK provides the possibility of joint calculation in one model of spatial systems «construction-base», including physically nonlinear properties, and including modules for solving static, dynamic, filtration and thermophysical problems.

The latest sources of research and publications analysis. At present, specialists in the field of tunnel and metro construction are no longer satisfied with traditional calculations [9]. Based on the results of the latest studies and publications analysis[1 –- 6, 9, 11 – 17], it is possible to single out a group of designers who perform calculations in specialized programs. Basically, the obtained data are compared in different software complexes. The calculations are forecast, and the determination of the stress-strain state is not corrected in the course of the construction and installation works due to the lack of actual data. Often, the design team of accountants is not directly related to the organization performing the construction of the object and performing geodetic control. Studies to determine the actual state of objects in many cases are practically not carried out or are performed in insufficient volume [17].

Allocation of common problem unresolved parts. The problem of designers dealing with calculations in a three-dimensional setting is not even in the complexity of creating a computational model, but in the inadequacy of objective data. Such data include the engineering and geological conditions of the construction site, the absence of a specific decision on the mechanisms used at the stage of erection, their impacts on structures and the soil massif, the complete lack of information on the results obtained as the facility is erected and during its operation. Without solving these problems, it is impossible to construct an actual analytical model corresponding to the actual work of the structures and the surrounding array.

Formulation of the problem. For the construction of an adequate calculation model, its correction is necessary in accordance with the data obtained as the object is being built, and the testing of the structures by loading. Only according to the actual data obtained directly on the construction site before construction and during the production of works it is possible to adequately determine the work of structural elements, their stress-strain state and exact values for precipitation. According to these data, a public database should be formed and experimental data can be accumulated.

Structural and technological solutions/ In the zone construction influence of the overpass there were tunnels of the underground on the station Moscow – station East, the penetration of which was carried out in closed way, as well as the ventilation and ventilation chamber were made in the open way. The supporting structures of the tunnels are cast-iron and concrete lining, consisting of individual rings with a width of 1.0 m. The distillation tunnels are structurally separated from each other by a casing.

The reinforced concrete lining of both tunnels is made with the use of rings consisting of ribbed reinforced concrete blocks. At the top of each ring there are key liners (blocks). The rings of the lining are connected together by bolts in the longitudinal and transverse directions.

Bearing constructions of the vent block are prefabricated-monolithic bottom, monolithic reinforced concrete frames, prefabricated and monolithic walls. The covering consists of prefabricated reinforced concrete slabs and monolithic sections.

Over the subway tunnel, a city road overpass was designed (Figure 1). The scheme of the overpass – 24.0 + 21.0, width 37.5 m.

To stabilize the deformations of the ground above the tunnel, protective shield is provided, consisting of monolithic reinforced concrete slab on bored piles with a diameter of 0.63 m and 0.8 m. The protective screen is located above the tunnels, the thickness of the backfill between the arches of the tunnels and the bottom of the protective screen plate varies from 2 to 3 meters. The length of the bored piles of the protective screen is 14 m, in the case of the ventilation – 22 m. The slab thickness of the protective screen above the tunnels is 500 mm, above the vent block – 1000 mm [14].

Figure 1 – General view of the projected transport interchange at the intersection Independence Avenue from the Filimonova Street

Development of the calculation model. Calculations of modern projects of structures under construction in complex engineering-geological and constrained conditions are impossible without the use of modern software complexes that could consider real work of the soil. The process of establishing soil parameters for subsequent numerical modeling is the most important component of ensuring a qualitative assessment of the stress-strain state of the soil massif. Therefore, it is necessary to pay special attention to the choice of soil model and input parameters. It is also necessary to remember that it is necessary to conduct a series of preliminary research calculations on modeling the work of the system «construction – soil» [1].

The success of the design calculation depends to a large extent on how adequately the selected elements and models as a whole reflect the actual design.

In order to ensure the safe operation of existing metro lines, it becomes necessary to assess the negative impact of tunnel linings on the surface of construction work, which in the future will reduce the risk of bearing capacity loss and structure destruction. In the event based on the results of calculations, there are unacceptable deformations for normal operation, it is necessary to develop a set of protective measures to avoid the negative impact of new construction [7].

Simulation of the stress-strain state of the transport interchange and its changes during construction were carried out using the programs Midas GTS NX and SOFiSTiK (WinTube). These programs allow to determine the stress-strain state, both in soil massif and in structures interacting with the ground at any stage of structure erection. When modeling the «construction-ground» system, a spatial design scheme was used.

In the design scheme, the model was developed considering stage-by-stage development from the level of the existing surface in the zone of the operating subway lines, as well as assessing the possibility of raising excavation bottom and, correspondingly, the tunnel caused by elastic deformations during unloading of underlying subsoil.

When creating the design scheme, measures were taken to prevent dangerous uneven deformations of tunnels lining by stabilizing the surrounding soil mass to ensure uniformity in raising the entire circumference of the lining while maintaining the operational size and reducing the amount of raising the head of the rail.

The first and most important stage in the development of the calculating model for the transport interchange was geometric modeling. The input and output data were determined, simplifying assumptions were made about the determining ratios, the boundary and initial conditions of the object, the stage of production, i.e. the idealization was carried out-the transition from the initial physical system to the three-dimensional computational model. Next, the final parameters of the model were set considering the condition of the objects operation.

The transition from a design to a calculating model made up of basic models is most often done on an intuitive level, and the first motive behind this transition is geometric considerations («likeness» form) [10].

Step by step all the elements of the interchange site were created, namely: geological conditions, existing constructions of underground tunnels with vent block, as well as a protective screen with bored piles. All the elements of the model were as close as possible to the real ones.

The idealization of the computational model and the inability to make it absolutely adequate to the actual construction there was created a situation of some uncertainty, and it is precisely in this uncertainty that the design solutions [10].

One of the important aspects of creating transport structure computational model is that in difficult engineering-geological conditions there is the correct choice of the dimensions of the calculation area. It is known that it depends on the type of structures being calculated and can be adopted according to the following parameters, given in Figures 2 and 3 [10].

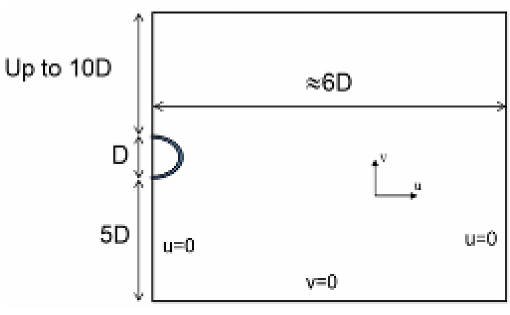

Figure 2 – Calculation area for shallow tunnels

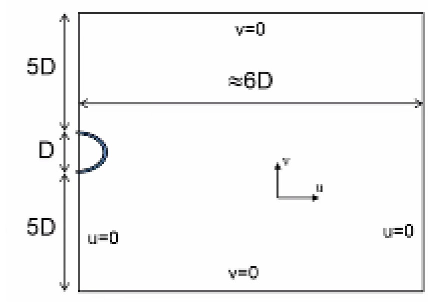

Figure 3 – Calculation area for deep tunnels

For this task, the design area was chosen for both shallow tunnels. However, in the course of the research calculations, it was corrected basing on the dimensions of the compressible earth stratum, within which deformations of the soil massif were considered..

The next stage in the development of the computational model is the idealization of structures materials and properties of soils. In most cases, the material is provided with the properties of ideally elastic, ductile or loose material. To select an adequate design model of the material, it is necessary to perform a series of experiments, rather than choosing more or less suitable geometric image [17]. As a rule, this stage is omitted, and known data on physical models of material work, previously obtained from experiments on absolutely other objects, are used in the calculation.

When calculating structures built in complex engineering and geological conditions, the adequacy of the model can be achieved only if a number of laboratory and virtual studies are carried out for the given object.

The geotechnical model was built on the basis of analysis and engineering materials generalization and geological surveys performed at the construction site at different times.

In the geological structure of the projected construction site are: sand with an admixture of sandy loam, sandy silts and loam. To simulate soil behavior, the following soil models were adopted: 1) the Mora-Coulomb model; 2) modified model of the Mora-Coulomb; 3) modified Mohr-Coulomb model with hardening. The type of the model was used in accordance with the properties of bedding.

To model reinforced concrete tunnel structures, Liner Elastic linear elastic model is used. In the design model, the characteristics of construction materials based on design drawings were laid.

To differentiate the results obtained between the elastic behavior of the protective screen plate and bored piles where small displacements occur and the surrounding soil massif where plastic behavior is possible, special interface element is used [10]. This is done to exclude the appearance of strains and stresses concentrations of that have no real physical meaning. The spatial rigidity of all structural elements of the structure was set in accordance with the design solutions.

It should be noted that prior to the beginning of the calculations, a survey of underground structures was conducted and their actual parameters were considered in the calculations.

The pre-geometric model was created in the AutoCad program and then imported directly into the Midas GTS NX calculation system. Each element of the model was given certain properties, in which the material, section or other properties were specified. The entire model is a database of 1D, 2D and 3D properties.

As a result, a calculation model was obtained, which is the volume of the soil (Figure 3) where existing underground structures are located (Figure 4).

After completing the construction of the geometric model, it was divided into a grid of finite elements. The process of constructing the grid is based on the stable principle of triangulation where the optimal sizes are found, and an unstructured grid is constructed with condensation in the most critical places (Figure 5) [10]. As for the boundary conditions and the weight of the structures themselves, they can be set automatically, which greatly facilitates the solution of the problem.

Before the calculation was started, all the necessary stages of design calculation were considered, including production technology. The calculation was carried out according to the stages of erection in two variants.

The performed calculations correspond to the technical codes of the established practice of TKP EN, as well as the national annexes to them for the design of bridges and pipes, identical to the design standards of the European Union.

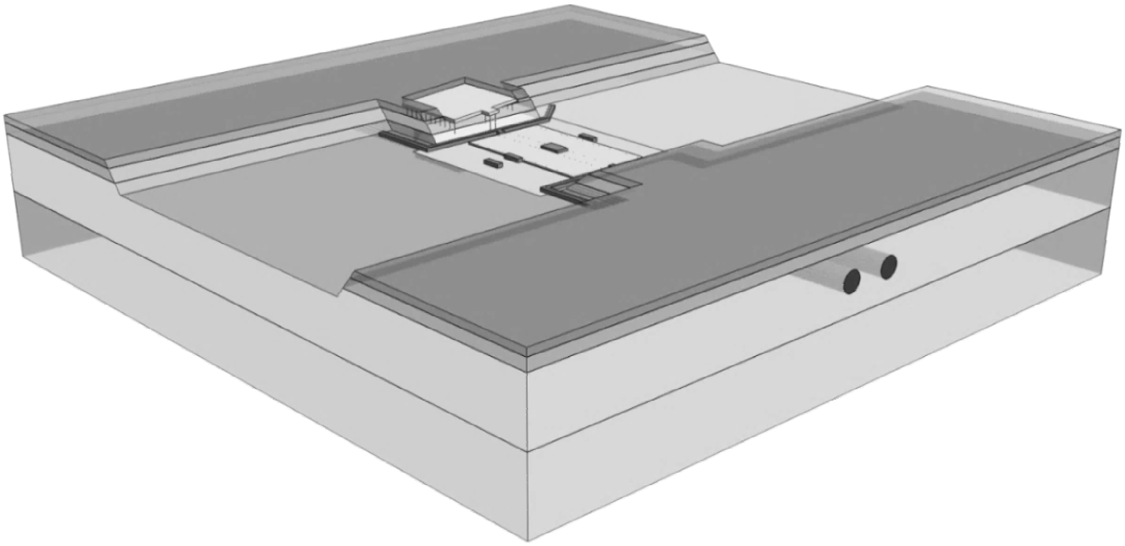

Figure 4 – General view of the geometric model of the transport interchange area

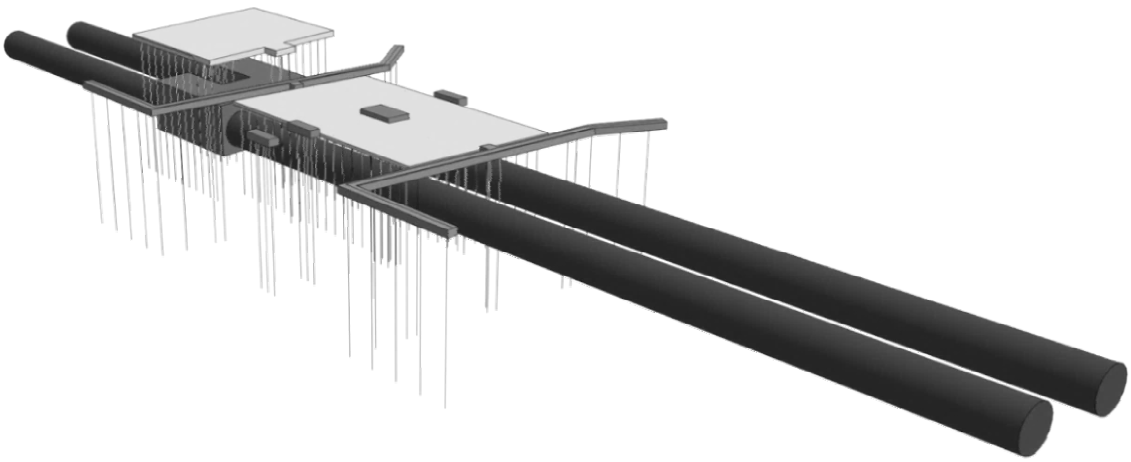

Figure 5 – General view of underground structures falling into the zone influence of construction

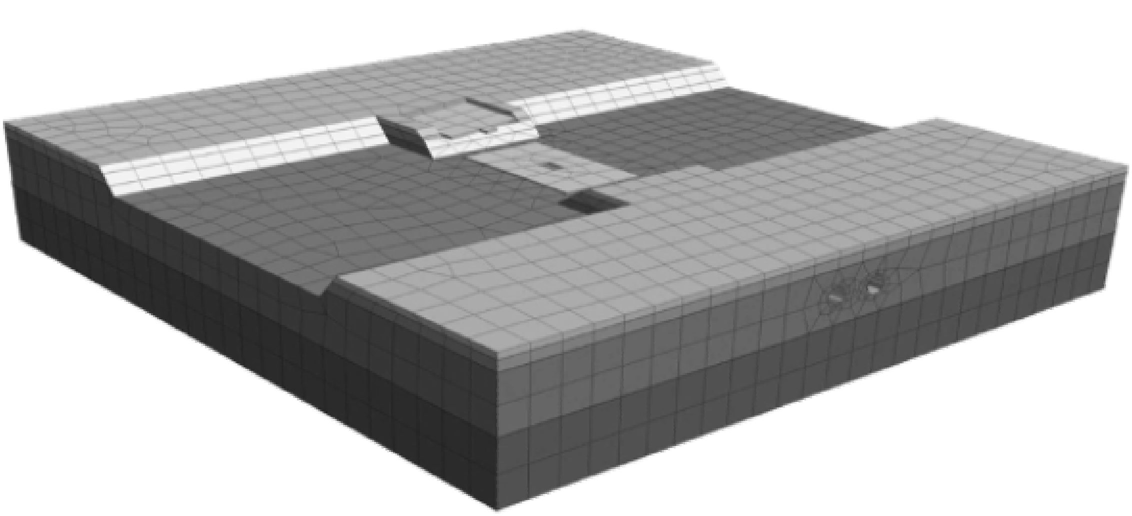

Figure 6 – Volumetric finite element calculation model of the transport interchange

Calculation results.

At the stage of soil development without performing protective measures, as it was expected, the bottom of the excavation and, correspondingly, the tunnels, caused by elastic deformations occurred when unloading the underlying soils. Raising the level of the railhead and the lining was 10…11 mm (Figure 7).

When creating a calculation model, measures were taken to prevent dangerous uneven deformations of the tunnels lining by stabilizing the surrounding soil mass to ensure uniformity in raising the entire circumference of the lining while maintaining the operational size and reducing the amount of rail head raising [14].

Considering carrying out of protective measures along the sides of tunnels in the form of rows from bored piles with a diameter of 630 mm in increments of 1650 mm, considering the stage of ground development to the design level, with the device of a monolithic reinforced concrete plate with a thickness of 500 mm, the stresses in the soil massif stabilize and the tunnels are raised evenly by 6 – 8 mm (Figure 8) [14]. This ensures the preservation of the geometric shape of cast-iron lining tunnels.

With the subsequent construction of pavement on a monolithic slab over the cast-iron lining of the tunnel, the weight of the cargo will increase and the overall rise will decrease.

Figure 7 – Deformations arising in the lining of tunnels, without performing protective measures

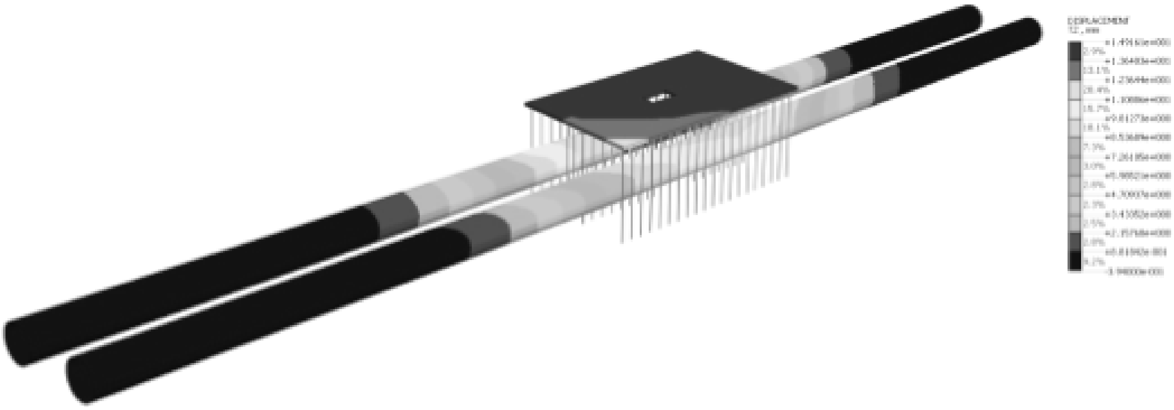

Figure 8 – Deformations arising in the lining of tunnels, with the implementation of protective measures

To study the operation of the protective screen plate in full-scale conditions and to verify compliance with the calculated assumptions, static backfilling tests were performed on the erected protective shield, thereby stabilizing the long deformations of tunnel rise. The results of the test correlate with the calculation and confirm the adequacy of the developed model.

It should be noted that an adequate calculation model it is not only the correctly chosen models of structures and materials, but also timely correction considering change in the actual state of structures and soils during construction, also considered when performing calculations.

Based on the performed calculations, minimizing the impact on the operating tunnels of the subway during the construction and subsequent operation of the traffic intersection, the necessary protective measures were assigned and strictly carried out.

Conclusions. Timely and objective assessment of new construction impact on the existing underground facilities of the subway allows developing design solutions that hinder the negative impact of construction, affecting the existing metro facilities operational reliability.

Modeling and calculations in a three-dimensional setting open access to the creation of full-fledged complex models with complex geometry, geology, boundary conditions and loads. However, the methodological support for such models development is at the very beginning of formation and must be compared with real data.

Such methods introduction in the design of various transport facilities would help in creating a database of experimental field studies, and avoid a huge number of emergency situations.

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ASSESSMENT OF THE TECHNICAL STATE OF TIMBER OUTLET ELEMENTS OF UNDERGROUND SUPERSTRUCTURES WHILE HOLDING THE MONITORING OF OPERATING UNDERGROUND RUNNING LINE TUNNELS IN MINSK

Timber elements computation methods are presented in the article including the endurance of load action and humidity for timber strength in accordance with statutory documents. The analysis of humidity and temperature conditions of the underground constructions is performed; defects and damages, timber outlet elements of underground superstructure of normal operating are described in details. An alternative method of timber elements underground superstructure longevity rising is proposed in obedience to the mentioned reasons of the decreasing longevity.

Keywords: *transportation construction, longevity, reliability, defects, damages, sleepers, tunnels, monitoring, deformations.*

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ОЦІНЮВАННЯ ТЕХНІЧНОГО СТАНУ ДЕРЕВ'ЯНИХ ВИВІДНИХ ЕЛЕМЕНТІВ ПІДЗЕМНИХ НАДБУДОВ ПРИ МОНІТОРИНГУ ПРАЦЮЮЧОЇ ГІЛКИ МЕТРО У МІНСЬКУ

Наведено методи розрахунку елементів із деревини, в тому числі вплив дії навантаження та вологості на міцність деревини відповідно до нормативних документів. Проаналізовано вологість та температурні умови підземних споруд, дефекти та пошкодження, поверхневі деталі підземної надбудови з деревини з метою їх нормальної роботи. Запропоновано альтернативний спосіб підвищення витривалості елементів деревини підземної надбудови згідно зі згаданими причинами зменшення їх довговічності.

Ключові слова: *транспортне спорудження, довговічність, надійність, дефекти, пошкодження, шпали, тунелі, моніторинг, деформації.*

Introduction. In the underground a rail-tracked and cross-tied lath is spread on the concrete base where the base is a granular sub base. The most important question while designing the running line tunnels is the reduction of destructive effect on the tunnel both with vibration and noise. In comparison with reinforced concrete sleepers, timber sleepers operate better. Another complicated task is fixing of a reinforced concrete sleeper with track concrete considering its possible change. The appliance of running line tunnel superstructure of treated timber sleepers in construction work is technically and economically reasonable as Belarus is a member of top 10 wooden European states upon key indications and the production of marketable timber in 2016 is 15,1 million cu.m. [8]. But the process of sleepers' change is a complicated and expensive operation that requires a lot of financial needs and hand-labour. For reliability and longevity resistance increase of track circuits there is a need of the investigation, elaboration, testing and implementation of constructions' production of constructions that meet modern requirements, improving vibration and noise insulation as well as protection from corrosion and as a consequence introduction of a new assessment of the status of the road.

Analysis of recent sources of research and publications. According to the analysis of recent research and publications, the research in specific areas of this subject can be highlighted [6, 7, 9, 15]. The research is conducted and improved type of the rails and the bases of substructure are implemented in constant research, while the question of sleepers structures modernization (the exception is their substitution with concrete and polymer) is not properly considered.

Identification of general problem parts unsolved before. The main characteristic of underground trains save movement the rail head level. In Minsk underground rail tracks are laid on the monolithic timber and at the stations - on the sleepers. Sleepers as an element of the upper line are disregarded in the calculations. Without switching the sleepers, it is impossible to build the actual analytical model corresponding to the actual operation of the structures and surrounding of the array. Therefore, determination of the underground surface line as a whole is impossible.

The **goal** is comprehensive study of buildings and constructions conditions, use of combined research based on the type of design, required social significance and conditions.

Basic material and results. Considering the fact that since 2015 a five-year period of transition to European standards of design and construction of the Eurocodes in the Republic of Belarus (order Mais No. 404 dated 10.12.2009 g) all calculations of wooden structures are produced according to TKP EN 1995-1-1-2009 "Design of timber structures. Part 1-1. General - Common rules and rules for buildings" [1, 12].

According to section 6.1.5 of TKP EN 1995-1-1-2009 [1] the calculation of the cross section on the first group of limit States direct solid wood in compression perpendicular to the fibers should be made as follows: (1) in case (2):

$$\sigma_{c,90,d} \le k_{c,90,d} \cdot f_{c,90,d} \;; \tag{1}$$

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_d} \;, \tag{2}$$

where $\sigma_{c,90,d}$, $F_{c,90,d}$, $f_{c,90,d}$ – is a calculated compressing press, forcing and resistance are in the functional contact area perpendicular to fibers;

A_d – a functional land area perpendicular to fibers;

$k_{c,90}$ – coefficient that takes into account the load configuration, possibility of splitting and degree of deformation.

The functional contact area perpendicular to the fibers A_d should be determined taking into account an functional contact length parallel to the fibers, which is the actual length of contact l is increased by 30 mm in each direction, but no more than a, l or l/2 (Fig. 1).

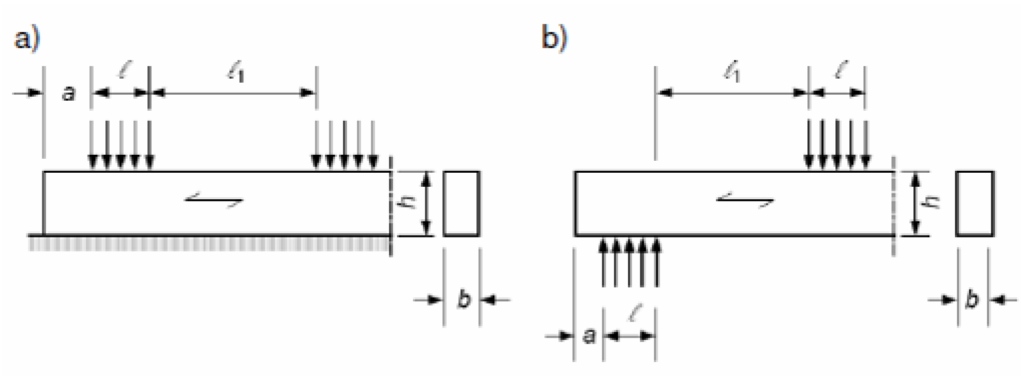

Figure 1 – continuous Element on (a) individual and (b) supports

The dimension $k_{c,90}$ is taken as 1.0 until the condition of this paragraph is done. In such cases, you should take the highest value $k_{c,90}$ but not more $k_{c,90}$ = 1,75.

For the object on the continuous support $l_1 \geq 2h$ (Fig. 1, a) value $k_{c,90}$ should be taken:

$k_{c,90}$ = 1,25 – for solid soft wood;

$k_{c,90}$ = 1,5 – for laminated soft wood;

where l is the length of the contact; h – the height of the element.

For the item in certain poles $l_1 \geq 2h$ (picture b), the value $k_{c,90}$ should be taken:

$k_{c,90}$ = 1,5 – for solid soft wood;

$k_{c,90}$ = 1,75 – for laminated soft wood with mm.

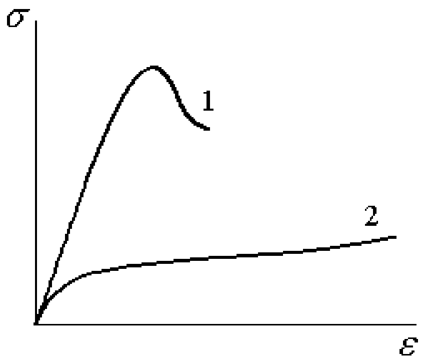

Figure 2 – Diagram of compression wood along fibres (1) and transverse (2)

According to curve 1 from the strain diagram (Fig. 2) the sample tested along the grain undergoes a small permanent deformation to failure. The destruction of the sample begins when the highest values of compressive force P_{max}, in the future there is a drop in load. A sample of the wood in the process of destruction of the fibers splits, then separation and breaking of the fibers, the formation of transverse folds and longitudinal cracks on the side surfaces of the sample happen.

Curve 2 shows that the diagram acquires a different character in the test sample in compression across the grain. To load P_{RG} curve chart looks like a sloped line, after overcoming this value, the sample is quickly deformed with a faint load change. This nature of the curve allows us to consider that the capacity of the test sample is exhausted. In this regard, the load corresponds to a tensile strength σ, P_{max} take such a load at which the sample is compressed to one third its original height.

When designing wooden structures, it should be considered the direction of the fibers in accordance with the diagrams of deformation, according to which the ratio of strength of wood in compression along and across the fibers is 1/8 – 1/10. Best option is the location of the compressive forces along the grain, i.e. in the direction of most resistance.

It is also necessary to calculate the increases and decreases in the temperature of rail lashes, allowable conditions of strength and stability.

Based on the studies of the stability of the path which includes directly considered in this article, sleepers, sets the permissible increase in temperature of rail lashes Δty. Calculation of the strength of the rail determines the allowable temperature drop.

Further investigation of the question of the computation of joint work of the concrete base, sleepers and thread rail is needed.

Important transport facilities safety depends on temperature and humidity conditions, operating conditions, nature of material and storage conditions. The speed of the aging process, in turn, determines the durability of any material.

In accordance with GOST 22830-77 [5], before placing in the path of the wooden sleepers, there is impregnated protective equipment on oily basis. For the sleepers made of pine plantations considering 50.1% of species composition of the Republic of Belarus [8], GOST [5] have additional requirements:

– prestressing humidity should not exceed an average of 18%;

– the depth of impregnation on a naked core should not be less than 10 mm;

– the average rate of antiseptic absorption should be 150 kg/m^3, with a minimum value of 125 kg/m^3.

The direct influence of the environment on the wooden structural elements in the form of metal dust, groundwater seepage can be reduced with constructive measures of protection or by the use of wood with high natural durability, or timber that has been previously protected against biological influences.

Temperature change leads to linear and volumetric deformations of structural elements, the nature of which depends on the nature of the materials.

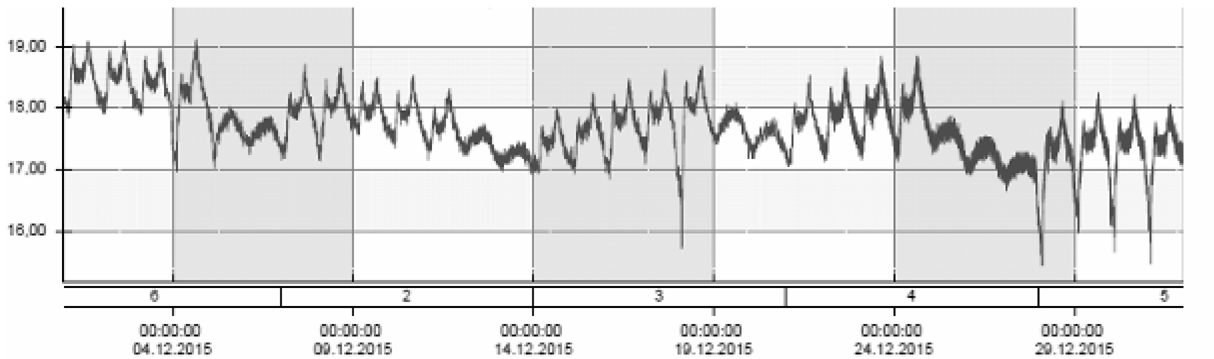

Figure 3 – Graph of temperature change in the running tunnel of the Minsk metro in winter

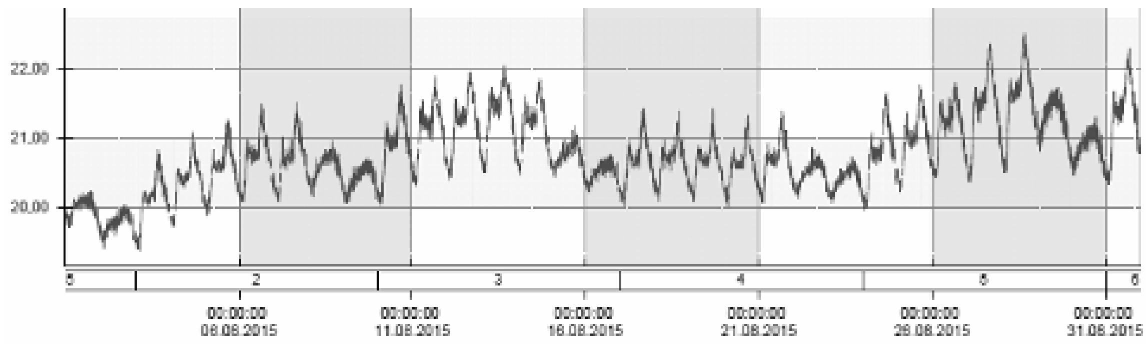

Figure 4 – Graph of temperature change in the running tunnel of the Minsk metro in the summer

Biological boundaries and intensity of damage is determined by the rate of chemical hydrolysis processes, gas and vapor permeability, and accelerated high humidity. The main cause of biological damage is the relative humidity exceeding 70%. Water change content causes a change in the volume of the material, which depends on the capillary movement of water in the material.

The change in moisture content of the air does not impact directly on the moisture content of the entire facility. Changes in relative humidity, which affect not immediately lead to different volumetric changes of connected materials, creating an invalid power of compression and extension.

The equilibrium moisture content in the material is established as a result of moisture exchange between him and air. The degree of absorption of moisture from the air (sorption) and return it back (desorption) determined by the temperature and relative humidity. Therefore, the main rule that must be respected, – the minimization of the absorption and desorption of moisture, that is, the achievement of thermal and humid dynamic equilibrium in the materials.

Occurring over a long period of time, alternate saturation with water and drying of structural materials leads to material fatigue and, as consequence, to decrease in strength.

The main causes of disruption of the normal operation of wood are rotting, cracking, mechanical wear of the wood under the pads and shoes develop holes from screws and crutches.

The whole rotting wooden sleepers is due to:

– the penetration rate of wood-destroying fungi through the cracks in the impregnated layer;

– improper storage and stacking, which directly affect the humidity conditions;

– damage to the impregnated surface.

The subway system is characterized by the decay in the upper third of the wooden sleeper's thickness.

There are two main reasons for the cracking (fracturing) wooden elements of the upper line:

– shrinkage of wood;

– rolling stock impact.

Timber sleepers are cracking - cracking of shrinkage of wood, developing mainly on the upper bed, and them falling into the water, particles of metal, dust and sand contribute to its decay.

The impact of train loads leading to tensile stresses from the lower bed, which in turn leads to developing over time, cracks as well as cracks with a length up to 30 cm in places of penetration of the main elements of rail fasteners.

The main causes of mechanical wear of the wood sleepers:

– vibration pads;

– infringement of sleepers installation technology;

– the way frequent remaking;

– the use of materials that do not meet these standards requirements.

Duration effect of load and moisture content on the timber strength in accordance with clause 2.3.2.1 EN 1995-1-1 [1] is calculated:

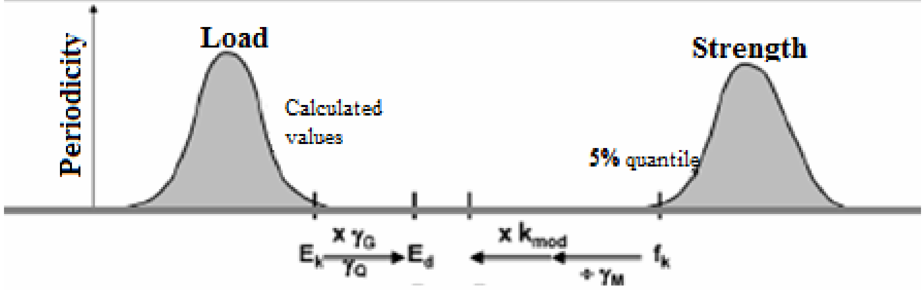

Figure 5 – Wooden structures safety

$$\sigma_d \le f_d \ ; \tag{3}$$

$$f_d = \frac{k_{mod} \cdot f_k}{\gamma_M} \ ; \tag{4}$$

$$E_d = \frac{k_{mod} \cdot R_k}{\gamma_M} \ . \tag{5}$$

Table 1 – Values of k_{mod}

Material	Standard	Operating class	Loadcase duration				
			Constant	Long -term	Medium -term	Short-term	Special
Solid wood	EN 14081	1	0.60	0.70	0.80	0.90	1.10
		2	0.60	0.70	0.80	0.90	1.10
		3	0.50	0.55	0.65	0.70	0.90

The equation of durability of wood [1] in accordance with the kinetic concept of solids strength can be written:

$$t = t_0 e^{\frac{U_o - \gamma f}{RT}} \ ; \tag{6}$$

$$lg\, t = lg\, A - \alpha f, \tag{7}$$

where

$$\alpha = \frac{\gamma}{2{,}3RT} \ ; \ \ lg\, A = \frac{U_0}{2{,}3RT} + lg\, \tau_0 \ , \tag{8}$$

U_0 – initial activation energy of fracture, kJ/mol;
t_0 – period of the thermal vibrations of the atoms.
f – stress, MPa;
t – time to failure (durability);
γ – structure-sensitive coefficient, kJ/(mol·MPa);
R – characteristic of the thermal motion (gas constant), kJ/(mol ·deg);
T – temperature, K.

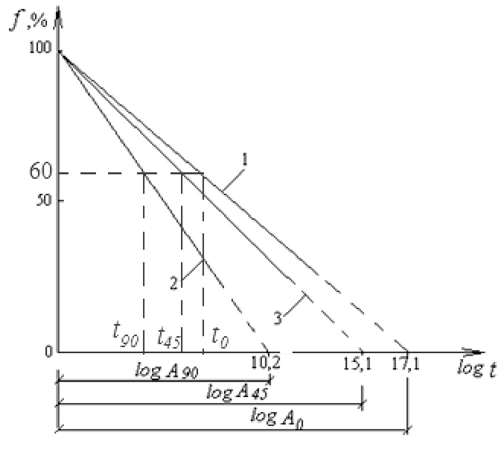

Figure 6 – The long-term strength dependence of wood for different types of the stress state:
1 – tension, compression and shear parallel to the grain (Adopted in EN 1995-1-1 regardless of SSS);
2 – tension across the grain; 3 – stretching at an angle of 45^0 to the fibers

An alternative to the traditional materials to increase durability and improve the operating conditions of the sleepers permanent way of the Metropolitan is the following composite system: working together wooden ties, concrete Foundation and GFPR-shell. This system better distributes the load from the temporary rolling stock between elements and at the same time is protected from corrosion, degradation of materials, because it is insensitive to environmental influences.

The modulus of elasticity of this material is twice the modulus of elasticity of concrete, its strength in tension, compression and bending exceeds the strength of steel more than doubled, and the resistance to the effects of lateral forces is only 2.5 times less than the strength of steel.

Durability for more than one hundred years (forecast), minimum operating costs, manufacturing plant - advantages of alternative composite material

Currently, this system is tested for spans in Canada, where it is built and successfully operated on roads (highways), and dozens of bridges with spans of 11 to 90 m.

Conclusions. For the analysis of structures it should be needed to consider all known science impact on design. Only comprehensive interpretation of the studies outcomes tests conducted by different specialists, enables the establishment of objective diagnosis and, as a consequence, development of the necessary activities as for the design and buildings restoration.

Accordingly, without considering the actual condition of the sleepers it is impossible to determine the surface structures of the metro as a whole.

When the timber outlet elements of underground superstructures are applied in the design, construction and monitoring of the metro tunnels structures, attention should be paid to:

– modeling and calculation of the above elements in accordance with applicable regulations;

– mounting technology, as the second stage, the possibility of defects and damages, leading out of normal operation;

– operation temperature and humidity conditions control, which directly affects the properties and the strength characteristics of the used materials.

For a better evaluation of the structure it is needed to study concrete base, sleepers and thread rail joint work calculation.

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PREPARATORY STAGE FOR MONITORING THE LOAD-BEARING STRUCTURES OF THE OPERATED SUBWAY STATION

Modern technologies for the construction of a subway underground allow reconstruction of operated stations without stopping the movement of trains and subway disruptions. However, this problem is quite complicated, because at the device of transitive tunnels it is possible to violate the stress-strain state of the main load-bearing structures and ground of the station base, track upper-structure drawdown appearance that is impermissible for the safe operation of the subway. The article presents the results of the preparatory stage of theMinsk subway station main load-carrying structures state monitoring, as well as the results of spatial calculations based on design data, considering the stage of transition tunnel construction work from the first to the third branches.

***Keywords:** monitoring, subway station, transitional tunnel, reconstruction, design model, ground, deformation, stresses.*

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ПІДГОТОВЧИЙ ЕТАП ДЛЯ КОНТРОЛЮ НЕСУЧИХ КОНСТРУКЦІЙ СТАНЦІЇ МЕТРО, ЩО ЗНАХОДИТЬСЯ В ЕКСПЛУАТАЦІЇ

Проаналізовано сучасні технології будівництва підземного метрополітену, які дозволяють реконструювати експлуатовані станції без припинення руху поїздів та перебоїв метро. Досліджено досить складну проблему порушення напружено-деформованого стану основних несучих конструкцій та основ фундаментів станції та появи перенапружень надбудови над колією, яка необхідна для безпечної експлуатації метрополітену при влаштуванні транзитних тунелів. Наведено результати підготовчого етапу моніторингу стану основних несучих конструкцій станції метро м. Мінська, а також результати просторових розрахунків на основі конструктивних даних з урахуванням етапу роботи з будівництва перехідного тунелю з першої до третьої гілок.

***Ключові слова:** моніторинг, станція метро, перехідний тунель, реконструкція, проектна модель, земля, деформація, напруження.*

Introduction. In the Republic of Belarus, as in many other countries, there is a planned approach to the creation of metro lines. It allows to consider the current and future interests of the city and organizations involved in the construction and operation of subways. From the planned approach the principle of the metro design of the metro follows: the development of general scheme and the construction phase-out in conjunction with the master plan and integrated scheme for the development of city public transport all types.

Usually the general scheme of the subway is created before the development of the first-line project. The general scheme and the stages of construction constitute the strategic plan for underground development.

However, there often is situation when changes are made to the general plan of city development in connection with the current situation in the city economy and the country as a whole. It leads to the fact that the decisions made in the design and construction of the first stations may not be relevant at the time of new stations and tunnels construction.

A similar kind of problem arose during the design and construction of the interchange node from the first to the third branch of the Minsk Metro. The design the one of the intermediate stations of the first branch was carried out considering the long-term development of the master plan for the development of Minsk city. However, since the construction of the first branch, the general plan has been repeatedly changed, which led to the formation of a large transport hub in the place of the previously designed third branch. The modern city has a site on this site, which simultaneously crosses the railway station where trains run in four directions, a turning ring of tram routes, bus and trolleybus lines that have end points of the route in this part of the city, a metro line and intensive pedestrian traffic. When constructing the third branch of the station in an open location and constructing a transitional tunnel between the first and third lines, the construction would paralyze the traffic of the entire city. In this regard, it was decided to remove the third branch station beyond the transport hub. However, when designing a transition tunnel from the opposite side, it is necessary to make a breakthrough in the supporting wall, and also to make a penetration into the base of the station tray board, which can adversely affect the stress-strain state. During the construction of transitional tunnel, the project for the production of works does not provider stopping trains movement, which greatly complicates the construction task many times.

When solving such problems, it is necessary to ensure strict control over the state of structures and the upper structure of operated subway line track, as well as for the state of the surrounding development that falls into zone of influence, without changing the operation mode of the station [1 – 5]. Using automated systems for monitoring the state of bearing structures allows to solve such problems: carrying out a complex of measures to monitor the state of the load-carrying structures of the station in the period before construction, during construction, and also during a certain period after construction. It allows comprehensive assessment of the technical condition of the structure main load-bearing and assigns appropriate criteria for the operation of the transport hub. The use of finite elements complex analysis programs provides the possibility of joint calculation in one model of spatial systems «soil – construction», including those with physically nonlinear properties [12, 13].

Recent sources of research and publications analysis. Analysis of the latest research and publications on this subject suggests that it is increasingly common in the practice of domestic and foreign metro construction to face the need to monitor existing structures of the underground during its reconstruction. This set of measures significantly facilitates the production of construction and installation works on the reconstruction of existing facilities of the underground, while ensuring its normal operation [1 – 5, 7, 8].

There is also a complex of tasks where monitoring is the most important component of the entire construction industry. These tasks arise as a rule in the reconstruction of underground structures located in the historical, business and cultural centers of the city, where the opening of the day surface of the ground has very negative effect on the state of the surrounding buildings falling into the zone underground structures construction and reconstruction influence [6, 9, 11 – 14].

The most important measure of the preparatory stage of monitoring is the creation of an adequate calculation model. The model must consider real work of all units and elements of the structure before the start of production. Only after a model is as close to real conditions as possible, it allows further to adjust it, basing on the data obtained during the monitoring. However, the design model is often not adjusted considering the changes occurring during the construction or reconstruction of underground structures [9, 14, 15].

Identification of general problem parts unsolved before. Modern technologies of construction of stations and tunnels of the underground allow to carry out construction and installation works practically under any engineering-geological and hydrological conditions, including in dense urban buildings. Realization of any ideas in difficult urban conditions is impossible without careful monitoring of the existing situation surrounding state in the city. It is necessary to determine only those specific tasks accurately and reliably, the solution of which will allow realizing any engineering ideas without significant damage to the environment [10].

The **goal** is to obtain the real values of changes in the stress-strain state, depending on the state where the transition occurs to a later date before the start of production, to know the initial stress-strain state of the structures during operation in the planned mode, considering the years of defects and structural damage operation. Only knowing the initial state of the structure it can be determined the criteria for changing the stress-strain state during the construction and operation of the structure. It should be noted that it is necessary to consider the results of monitoring when developing design solutions for the facility reconstruction.

Basic material and results. The station type is single-span with a vaulted overlap. The length of the station is 97.5 m.

The covering of the platform area is made in the form of a monolithic reinforced concrete vault, rigidly connected with the walls of the station (Fig. 1).

Figure 1 – General view of the platform site of the station before reconstruction

The outer walls of the platform area are monolithic reinforced concrete, using the «wall in the ground» method. To the inner surface of the walls there were fixed reinforced concrete blocks with a thickness of 200 mm with holes for the passage of engineering communications. Above these blocks, decorative granite slabs are made. On the bearing walls there are made consoles with service passages and lighting.

The tin plate is made of monolithic reinforced concrete on concrete preparation. The plate has a variable thickness in the form of an inverted arch. On the slab there is laid track concrete.

Overlaps are in the level of the platform - prefabricated reinforced concrete slabs and monolithic areas in the middle of the platform, as well as in the area of the removable overlap. The slabs are mounted in ribbed sizes 1.48x9.95 m with cantilever overhangs on both sides. The support plates are made on brick walls. Bearing walls of platform plates are made of bricks. Wall thickness is 380 mm.

Between the axis of the walls of the platform slab there are monolithic reinforced concrete walls separating the projected stairwell descents. The above-listed structures are designed and built considering the subsequent reconstruction of the station from the intermediate to the interchange.

Monitoring provides for continuous monitoring in full-scale conditions for deformations and forces in the bearing structures of the station, and verification of compliance with their design values ensuring accident-free operation of structures, and includes [13]:

– Visual monitoring;
– Geodesic-surveying above-ground monitoring;
– Geodesic-mine surveying underground monitoring of the state of structures;
– Instrumental monitoring of structures state;
– Electronic remote monitoring of structures state;
– Scientific support of monitoring work.

The monitoring work is carried out in several stages. At present, the preparatory stage has been completed (before the reconstruction begins). The main work at this stage was work on the analysis of the initial information on the results of the survey. The results of geodetic (mine surveying) measurements for the two years period operation, preceding the start of the reconstruction works of the station, were subjected to careful analysis. Information on the technical condition of underground structures falling into the risk zone received from operating organizations was obtained. Measurements have been made to determine rolls, misalignments, deformations and uneven sediments of the bearing reinforced concrete track walls of the station. Geodetic marks were installed on the structures and the metro station with their binding to the city reference network. Beacons and detection sensors were installed on the cracks fixed in the station structures. Electronic sensors are installed on the reinforced concrete bearing wall, reinforced concrete tray plate. Full control over compliance with the technological regulations of works and scientific support of the preparatory phase was carried out.

The monitoring program provides for the emergence of data critical values obtained from an automated monitoring system (deformation of station structures, drawdowns and misalignment of running rails, where it is necessary to carry out measures to ensure the safety of train traffic), immediate coordination of decisions with the project organization, track service and tunnel structures of the operating organization and organization that provides scientific support for monitoring activities.

Visual monitoring prior to the beginning of the reconstruction included an inspection prior to the commencement of construction work and the installation of video surveillance in the planned development zone. The data from the CCTV cameras of the operating organization, obtained on request, are necessary to monitor the installed equipment and determine the VAT from the temporary operational load. The results of the work on visual monitoring will be provided in reports with photographs of the detected damages and their values, as well as recorded in the database - systematic results of observations suitable for processing and use with the help of specialized software packages.

Geodesic-surveying above-ground monitoring before the reconstruction included the construction of benchmarks for geodetic measurements and the performance of instrumental measurements in accordance with established benchmarks with reference to the city planned high-altitude network. Geodetic instrumental observations are conducted using optical digital and laser equipment in order to obtain high-precision results of changes in the planned altitude position of the installed brands and the speed of these changes. Based on the measurement results, the data will be entered in a single database.

Geodesic-surveying underground monitoring of the structures state included the installation of geodesic marks on the structures of the metro station and 3D scanning of the work site. Based on the measurement results, the data will be entered in a single database.

Instrumental monitoring of the state of structures prior to the beginning of reconstruction determined the width and depth of the crack opening, considering existing leaks in quantitative terms. The results of measurements are entered in a single database

Pre-commissioning of electronic remote monitoring of structures condition prior to the start of the reconstruction included the development of a scheme for the location of sensors at the station and the wiring diagram, the installation of electronic overhead strain gauge sensors on a reinforced concrete bearing wall and a tray plate, the installation of a wireless data transmission link via the Internet or cellular communications; installation of software on the automated workplaces of the construction organization and emergency technical assistance of the way service and tunnel facilities of the subway to control the stress-strain state of the station structures.

Electronic monitoring data has an internal record. Based on the measurement results, the data is stored in a single database.

Scientific support is provided at all stages of the station reconstruction. Prior to the commencement of monitoring activities, a database should be developed, where the recording of information must be strictly followed by all participants in the process. Periodic reports in the process of works production on the reconstruction of the station should contain the results of monitoring, information on the state of the facility, possible deviations and the forecast of changes in the state of engineering structures. It was made comparison of deformations and stresses in station structures, obtained from the results of calculations by design organizations with actual values and refinement, if necessary, of computational models. To specify the types and volumes of further work; there were developed recommendations for adjusting design decisions, the timing of repair or restoration work, proposals for further monitoring. The order and composition of the monitoring can be adjusted in the process of performing the work, depending on local conditions, opportunities and preliminary results.

To analyze the possible displacements in the structures of the platform area, a three-dimensional model of the monitoring object was developed. The model was created on the basis of design drawings.

The simulation was carried out in specialized geotechnical software. The stress-strain state of the «soil – structure» system was calculated by the finite element method in the conditions of the three-dimensional problem. The information model is a collection of volumetric elements and is designed in such a way that all the stages of the stress-strain state of the structures can be considered in the calculation. All the elements of the model were as close as possible to the real ones.

At the first stage of the model creation, the study of theoretical foundations and the collection of information about the object were carried out, causal relationships between the variables describing the object were revealed, namely, the interaction of the system «engineering structure – soil massive», its main components were studied.

The ground was set by the Mora-Coulomb model. In general, the material model is a system of mathematical equations describing the relationship between stresses and strains. The Mora-Coulomb model is based on the relationship between the effective stresses change rate σ 'and the strain rate ε'.

When creating this model, the traditional parameters of materials were used: the modulus of elasticity, the intrinsic weight, the specific cohesion, the angle of internal friction and the Poisson's ratio, etc.

In the design model, the characteristics of construction materials based on design drawings were laid. The characteristics of the soils were determined on the engineering and geological survey results basis.

An important question in the simulation was the adequate specification of the geometric and rigidity parameters of the structure.

At this stage, the input and output data were determined, simplifying assumptions were made about the determining ratios, the boundary and initial conditions of the object, the stage of production and the history of stresses, i.e. the idealization was carried out-the transition from the original physical system to the three-dimensional model. Next, the final parameters of the model were set considering the condition of the object operation.

In the design scheme, the model was developed considering the technology for the reconstruction of the platform section of the station.

After completing the construction of the geometric model, a finite element grid can be constructed. The process of building a grid is based on the stable principle of triangulation, where the optimal sizes of triangles are found, and an unstructured grid is constructed.

Calculations were carried out considering the sequence of production work developed by the project organization.

1. Construction of an abutment excavation pit to a station platform site and the device of its fastening.

2. Arrangement of vertical soil-cement piles reinforcement of the "wall in the ground" of the station.

3. Development of a pit with a fastening device to the bottom of the tray. Strengthening existing station structures.

4. Development of excavation soil with a protective screen device made of pipes under the tin plate.

5. Fixing the ground under the tin plate. Development of soil to the design mark.

6. Arrangement of technological apertures 800×2300 mm in existing structures of «wall in the ground» of the station.

7. Development of soil.

8. Installation of monolithic areas after each entry.

9. Opening of openings in the station existing «wall in the ground» at full cross-section.

After completing the construction of the geometric model, it was divided into a grid of finite elements. During the process of constructing, the grid is based on the stable principle of triangulation where the optimal sizes are found, and an unstructured grid is constructed with condensation in the most critical places (Fig. 4). With regard to the boundary conditions and the weight of the structures themselves, they can be set automatically, which greatly facilitates the solution of the problem.

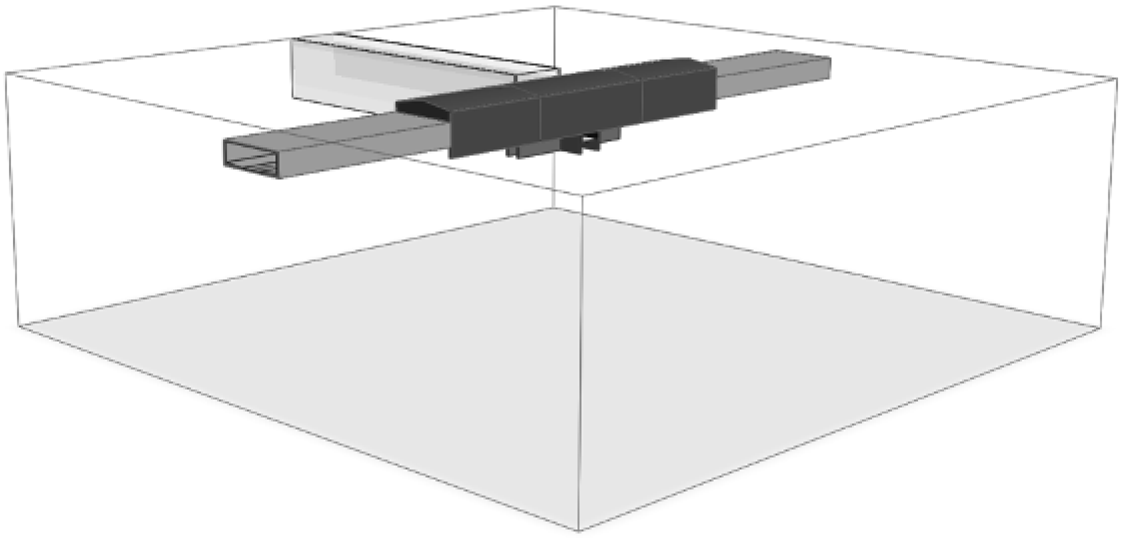

Figure 2 – General view of the geometric model of the construction site

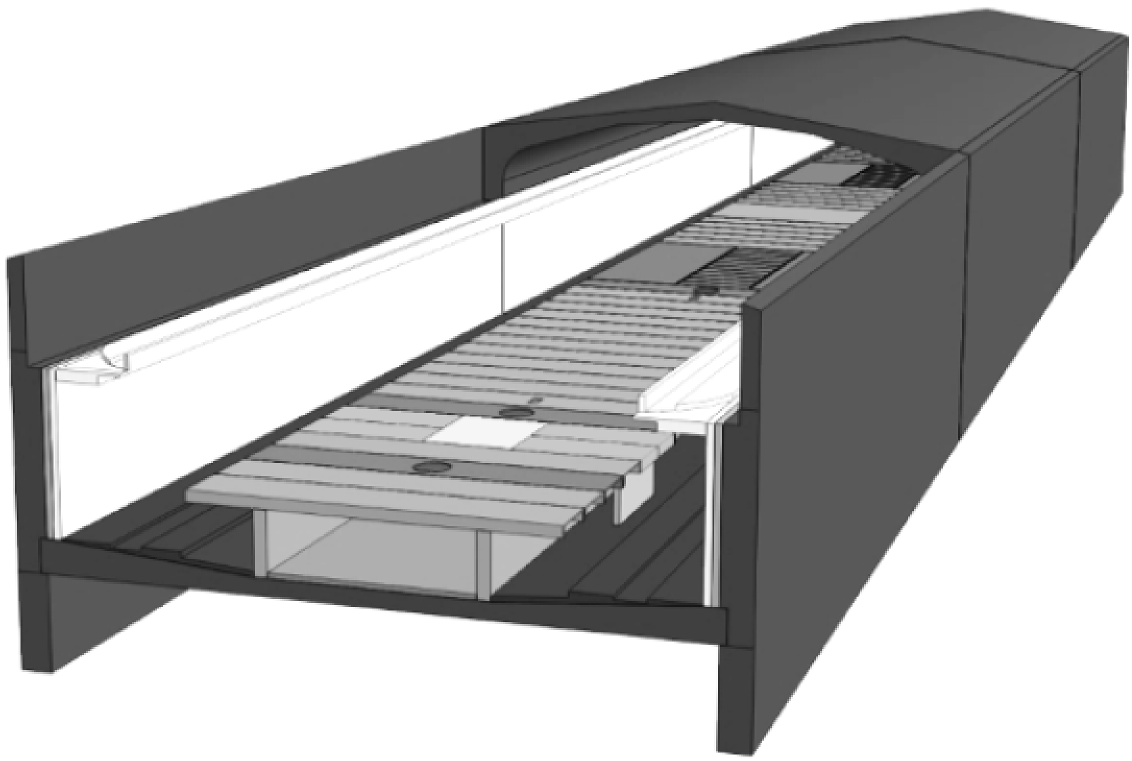

Figure 3 – General view of underground structures that fall into the zone of influence during reconstruction

Before the calculation was started, all the necessary stages of design calculation were considered including production technology. The calculation was carried out according to the stages of erection.

Based on the results of spatial calculations, the predicted displacements of the bearing structures of the platform area in the work area amounted to 7 – 8 mm. The predicted changes in the stresses in the bearing structures of the station are 5 – 11 MPa.

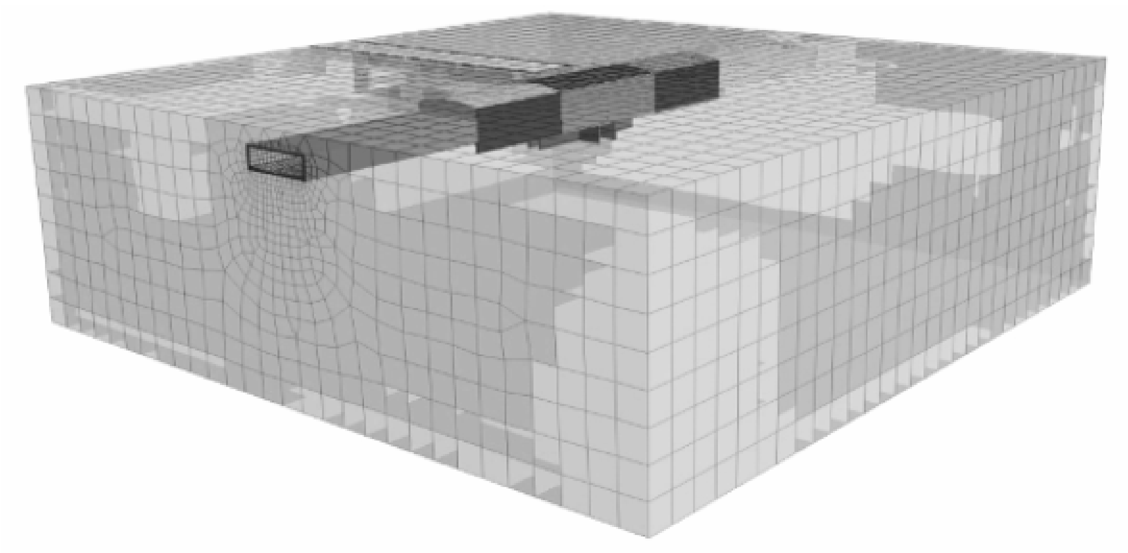

Figure 4 – The volume finite-element calculation model

On the basis of the performed calculations analysis, the maximum stresses occur in the wall in the zone of technological openings device and amount to 6 MPa. It was recommended to make metal or reinforces concrete clip of a wall part between adjacent apertures.

The maximum displacement of the supporting wall and the trough plate with track concrete occurs during the construction of the excavation pit. It is due to the removal of the active pressure on the wall. In addition to creating an active lateral pressure on the existing wall of the station, the surrounding rock massif of the station perceived part of the rasp from the vaulted cover of the station.

Removal of soil from the excavation in the zone of construction and installation works production on the junction of the transition tunnel will result in the horizontal component of the reference reaction of station vault being perceived only by the rigidity of the wall and the junction of the wall with the vault, which will lead to a significant change in the stress-strain state of the wall in the ground, as well as to possible cracking. To reduce the negative impact of the horizontal reaction of the vaulted cover on the wall, the project provided construction of so-called buttresses, which partially compensate for the removal of lateral pressure.

Metro facilities belong to highly responsible facilities with an increased risk of exploitation, thus it is necessary to monitor the movements and deformations of the existing structures of the platform site during the works.

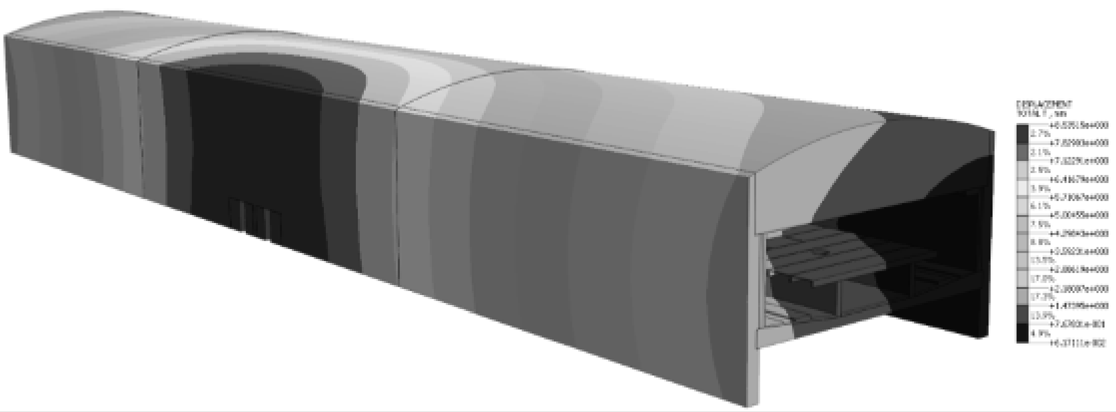

Figure 5 – Deformations occurring in the wall and vault of the station, at the time of tunneling

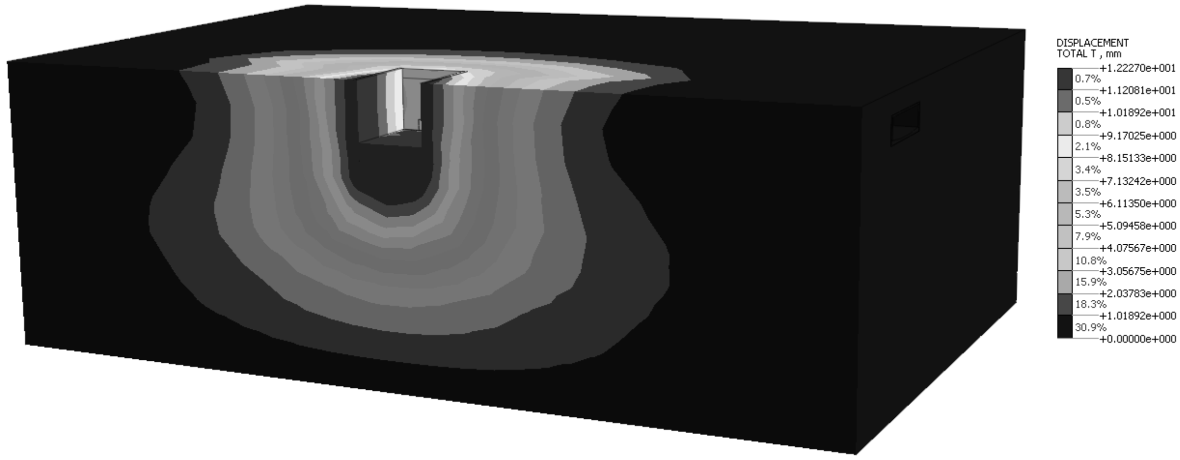

Figure 6 – Deformations of the soil massif after opening of the excavation

Conclusions. The preparatory stage of monitoring the construction and installation works for the reconstruction of the metro station in operation plays one of the key roles of the entire work course. It allows to consider the possible risks associated with the impact on the current strained-deformed state of the «soil – construction» system in a dense urban environment. This will significantly save labor costs and the cost of all construction in general. But mainly it will allow to prevent possible emergency situations related to the safe operation of the underground in the period of production. Preliminary calculations carried out with the help of modeling in a three-dimensional formulation enable the creation of full-fledged complex models considering the joint work of «soil – construction».

It should be noted that the computer simulation of the structure behavior under various loads can in no way replace the carrying out of field studies and testing of materials samples. It is necessary to improve the methods for calculating complex spatial problems associated with the operation of underground structures.

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PRINCIPLES OF CALCULATION AND MONITORING OF STRESS-STRAIN STATE OF THE GRADE-SEPARATED TRAFFIC INTERCHANGES IN MINSK

The results of scientific accompaniment and monitoring of construction of the transport interchange at the intersection of Independence Avenue and Filimonova Street over tunnels and other structures of Minsk subway. In order to ensure (in three shifts) the construction and installation works at construction of transport interchange around a number of innovative technologies in both for designing and work performance has been used. Construction monitoring envisaged continuous control of deformations and stresses of constructions of underground tunnels in the online mode and data transmission to all interested organizations. The calculation model of the existing tunnels has been developed, which includes the design of the lining and the surrounding soil massif. A theory for calculating underground structures based on the deformation of materials of building structures and geomechanical models composing a soil massif has been proposed.

***Keywords:** transport structure, stress-strain state, traffic intersection, subway tunnel*

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ПРИНЦИПИ РОЗРАХУНКУ ТА МОНІТОРИНГУ НАПРУЖЕНО-ДЕФОРМОВАНОГО СТАНУ ТРАНСПОРТНИХ РОЗВ'ЯЗОКУ МІНСЬКУ

Подано результати робіт з наукового супроводження та моніторингу будівництва транспортної розв'язки на перетині пр. Незалежності з вул. Філімонова над перегінними тунелями та іншими спорудами Мінського метрополітену. Для забезпечення цілодобового (в три зміни) виконання будівельно-монтажних робіт при будівництві транспортної розв'язки використано ряд інноваційних технологій як для проектування, так і для безпосереднього виконання робіт. Моніторінг будівництва передбачав безперервний контроль деформацій і напружень конструкцій перегінних тунелів метрополітену в режимі онлайн і передачу даних всім зацікавленим організаціям. Розроблено розрахункову модель перегінних тунелів, що включає в себе конструкцію оздоблення та її навколишній ґрунтовий масив. Запропоновано теорію розрахунку підземних споруд на основі деформаційної моделі з застосуванням діаграм деформування матеріалів будівельних конструкцій і геомеханічних моделей складових ґрунтовий масив.

***Ключові слова:** транспортна структура, напружено-деформований стан, перетин доріг, тунель метро.*

Introduction. Construction of a traffic junction at the intersection of Nezavisimosti Avenue with Filimonova Street was carried out over operated tunnels and other facilities of the Minsk subway.

The section of the tunnels in question is located on the first line of the Minsk metro between the Moskovskaya and Vostok stations and was put into operation on 26.12.1986. The project envisaged the construction of an overpass tunnels with typical prefabricated ferroconcrete spans for road bridges 24 m long and 21 m.

Initial data was provided by proposals for constructive and technological solutions for the traffic interchange at the intersection of Nezavisimosti Ave. with Filimonova street institutes «Minskinzhproekt» and «Minskmetroproekt».

Analysis of the latest sources of research and publications. Normative documents on the design of building structures underwent constant changes and additions and were often reprinted. The normative document SNiP 2.05.03-84 * «Bridges and pipes» [1] that operated until 2010 in the Republic of Belarus did not reflect the achievements of science and modern trends in the field of structural design.

The traditional approach to the construction of unique structures that have no analogues in underground construction is not always acceptable. Therefore, scientific research methods using innovative equipment and software complexes are of great importance [3–15].

Allocation of unresolved parts of a common problem. In recent years, the weight parameters of vehicles, the strength characteristics of concrete have increased, the industry has mastered the production of new efficient types of steels.

In 2002, in the Republic of Belarus, national standards for the design of concrete structures of the SNB 5.03.01-02 [2], harmonized with the Eurocodes (Eurocodes) [3.5] were put into effect.

Formulation of the problem. The development of unified European standards for the design of building structures – the Eurocodes, is aimed at «... ensuring the free movement between states of products, materials, technologies, services and scientific thought in the field of construction».

The Republic of Belarus also participates in this international program.

Structural and technological solutions. Initially, over the tunnels of the subway, a 200-meter overpass was designed at the base, which was intended for the installation of a monolithic reinforced concrete distribution plate.

The original version with a 200-meter overpass due to its high cost was rejected, after which other design solutions with a shorter length of overpass were considered. The final version of the decoupling assumed the construction of a 45-meter overpass, and access roads to it were carried out in the form of a mound.

The above-ground building structures of the multilevel transport crossing were above the operated tunnels of the first line of the Minsk Subway.

The adopted space-planning decisions required a change in the planning marks and the removal of soil over the tunnels of the subway, which led to a change in the initial stressed-deformable state of the underground structures on the site in question.

Metropolitan was put a criterion about the inadmissibility of moving the head of the rail more than 10mm. This criterion provided for the execution of construction and installation works in the «night windows». This circumstance would lead to an increase in the timing and cost of construction.

To carry out construction and assembly work in a short time and ensure the work in three shifts in the production of construction and installation works, a number of innovative technologies are applied.

Unique underground structures constructed in difficult engineering-geological conditions represent a special class of building structures. Unlike conventional building structures that are designed for specified loads, the design of the lining and the surrounding soil mass should be considered as a single system when calculating the limes operating in the regime of interacting deformations with the soil massif.

The calculation scheme of the system «lining – soil massif» is represented as a medium, divided into finite elements, which are joined together by rigid or elastic bonds.

The half-space under consideration is divided into three-dimensional elements. Since the complexity of such a calculation increases significantly, it is required for its implementation the use of software complexes specifically designed for the design of bridge and tunnel structures.

The finite element method allows calculating the lining not only on the basis of an elastic model of their interaction with the array, but also taking into account the nonlinearity of the deformation of the array and the lining. In this case, the geomechanical model of the soil massif corresponding to the engineering and geological conditions of the structure is taken into account, taking into account its anisotropy, heterogeneity, stratification, nonlinear nature of the soil operation and lining.

The finite element method makes it possible to calculate the design of the transport interchange at various stages of their construction in accordance with the technological sequence of opening the workings and erecting the object. Step-by-step disclosure of excavations is simulated in the design scheme by successive removal of the corresponding finite elements.

As a result of the decision, efforts, strains and deformations in the elements are obtained. The software complex used to solve this problem provides for the derivation of this information in a simple and visual form in the form of pictures of the distribution of stresses and deformations.

In this paper, using the integrated software package, an assessment was made of the effect of the constructed transport interchange complex on the existing structures of the Minsk subway tunnels.

Figure 1 – General view of multilevel transport interchange

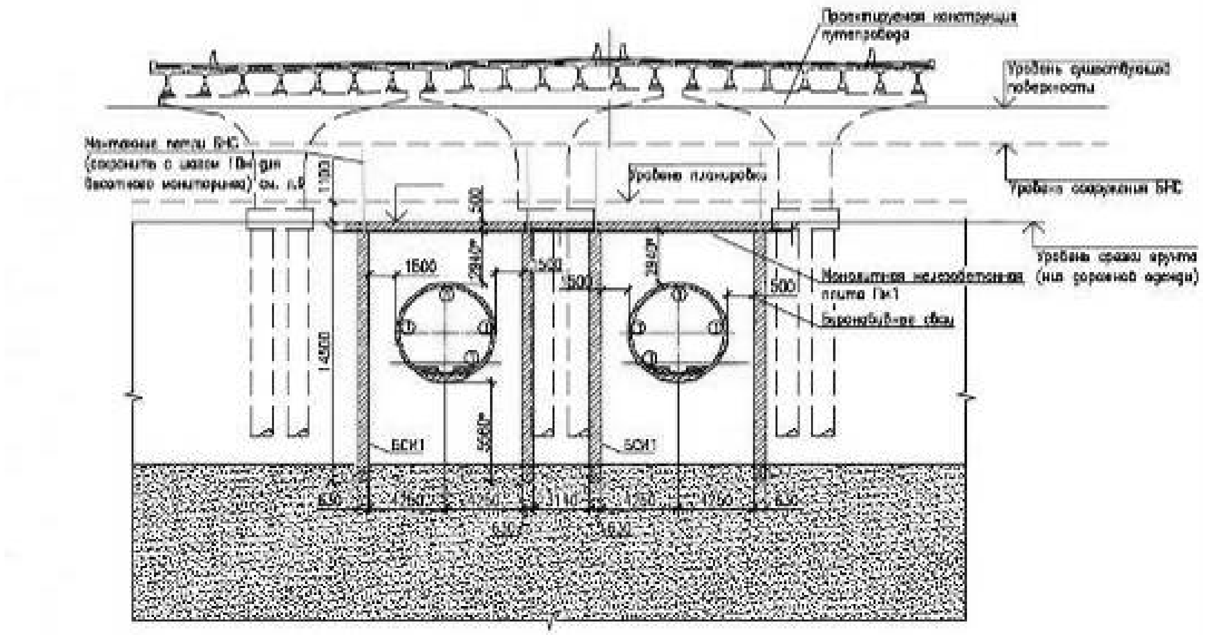

Figure 2 – Cross section of a multilevel transport interchange

The Republican Unitary Enterprise «Belarusian Road Research Institute BeldorNII» together with the «Bridges and tunnels»department of BNTU developed technical codes of established practice (TKP) and national annexes to them on the design of transport facilities identical to the design standards of the European Union: TKP EN 1991-2 , TKP EN 1992-2: TKP EN 1993-2 and TCKP EN 1994-2.

The National Standard implementing the relevant Eurocode includes the full text of the Eurocode as issued by the CEN, preceded by the National Title Page and the National Foreword, and which is accompanied by the National Annex.

The National Annex contains information on only those parameters that were left undefined in the Eurocode and are subject to national choice and are intended for use in the design of transport facilities to be built in the Republic of Belarus.

Calculations of structures for load capacity (1 group of limit states) and suitability for normal operation (2 group of limit states) in normal sections for any form of cross-sections with any damages, any arrangement of reinforcement within the cross-section and an arbitrary system of forces is made on the basis of a general deformation model section, which is based on the following assumptions:

– longitudinal deformations of concrete and reinforcement in normal sections are distributed according to the single-plane law at all loading levels;

– complete diagrams of uniaxial deformation are used for compressing and stretching concrete and reinforcement, including descending branches;

– sections can have any shape (rectangle, Taurus, I-beam, channel, circle, ring, triangle);

– sections are given in a discrete form by a combination of elementary sections of concrete and reinforcement; geometry, size and number of them are determined by the concrete situation;

– the stresses in concrete and reinforcement, as well as the deformations corresponding to them, are distributed evenly within the limits of elementary areas;

– the relative deformations of the reinforcement having adhesion to concrete are assumed to be the same as for surrounding concrete;

– sections may include concretes and reinforcement of different classes;

– equilibrium equations for external and internal forces are recorded in a uniform form under various force effects at all loading levels without seeking the position of the neutral axis.

The peculiarity of the new generation normative document for the design of bridge structures and pipes is the introduction of a unified approach to the calculation of sections of building structures from various materials.

At the present time, a lot of computer programs have been developed that use the «deformation model» to calculate normal cross sections, which allow calculations of arbitrary cross sections from any material, including multiply connected and non uniform ones.

For deformation of inhomogeneous elastoplastic soil models, deformation diagrams are given, as for metal and concrete.

Materials deformation diagrams are shown in Figures 3 – 5.

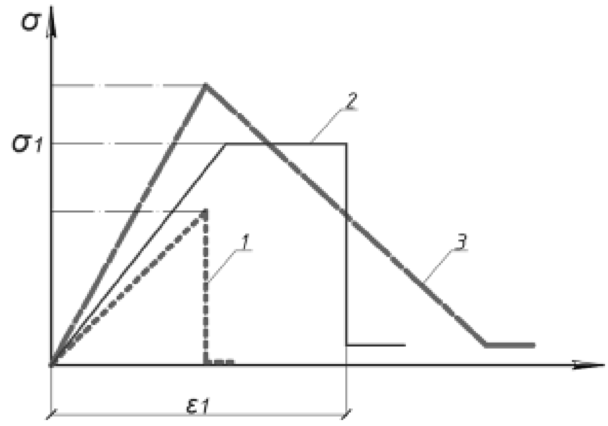

Figure 3 – Diagram of deformation of inhomogeneous elastoplastic soil models that take into account the fracture:
1 – brittle; 2 – elastoplastic; 3 – with a linear decrease in resistance beyond the ultimate strength [7]

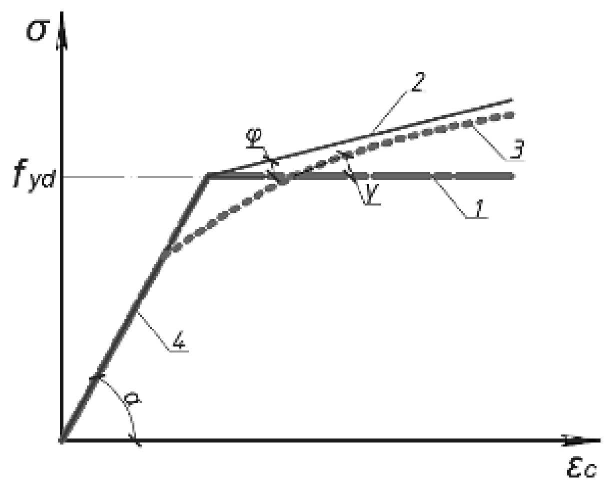

Figure 4 – Idealized steel deformation diagrams:
1 – Prandtl; 2 – with linear hardening; 3 – with nonlinear hardening; 4 – perfectly elastic

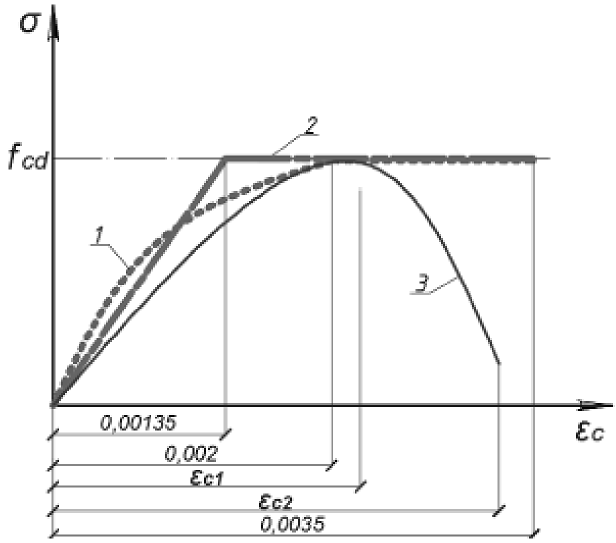

Figure 5 – Diagrams of concrete deformation during compression:
1 – parabolic; 2 – elastoplastic; 3 – nonlinear

To determine stresses and deformations in the cross-section of the elements, an iterative procedure is used, which makes it possible to follow the development of zones of plastic deformations and establish the limiting state by the criterion of limited plastic deformations for a metal and the criterion of limited limiting deformation for concrete.

Experience in the operation of building structures of transport interchange shows that their reliability and durability depend on a large number of random factors that vary with time [6].

The development of a method for calculating reinforced concrete structures with time factor is a further development of the method of calculating structures by limiting states.

In calculating the strength of sections normal to the longitudinal axis of the precast monolithic reinforced concrete element, use:

– equations of equilibrium of moments and longitudinal forces;

– equations of the distribution of relative longitudinal deformations within the compound section.

For the general case of calculating the section of a precast-monolithic element, the equilibrium condition in the matrix form

$$\begin{Bmatrix} N_{Sd,z} \\ M_{Sd,x} \\ M_{Sd,y} \end{Bmatrix} = \begin{bmatrix} R_{1,1} & R_{1,2} & R_{1,3} \\ R_{2,1} & R_{2,2} & R_{2,3} \\ R_{3,1} & R_{3,2} & R_{3,3} \end{bmatrix} \begin{Bmatrix} \varepsilon_z \\ k_x \\ k_y \end{Bmatrix}, \qquad 1)$$

where εz – longitudinal relative deformation at the level of the selected longitudinal axis z;

kx, ky – changes in the curvatures in planes coinciding with the x and y axes;

R1,1 – R3,3 – elements of the matrix of instant stiffnesses for the composite section.

The introduction into the normative documents of deformation diagrams connecting the stresses and deformations of materials in the process of loading made it possible to significantly improve the methods for calculating building structures (steel, cast iron, reinforced concrete, prefabricated monolithic reinforced concrete and others) to approximate the calculated models to the actual physical work of elements from various materials.

Monitoring provides for continuous monitoring in full-scale conditions for deformations and stresses in the load-bearing structures of the subway tunnels and verification of their compliance with design values ensuring accident-free operation of structures and includes:

– geodetic above-ground monitoring;
– geodetic underground monitoring;
– visual monitoring.
– instrumental monitoring of the state of structures;
– electronic remote monitoring;
– scientific support of monitoring activities.

Construction of the facility was carried out in 3 shifts without stopping traffic. Data transmission was carried out by all interested organizations on-line.

Scientific support included the following types of work:

– analysis of the results of geodetic observations (underground and above-ground monitoring) submitted by its organizations and recorded in a single database;

– provision of periodic reports containing monitoring results and their analysis, information on the state of the facility, possible deviations and forecast of changes in the state of engineering structures;

– control of geometric dimensions and position of the protective screen;

–conducting verification calculations and comparing the results with the spatial model, if necessary, correcting the model with the actual values obtained;

– clarification, taking into account the results of monitoring the types and amounts of further construction and installation work, developing recommendations in case of need for adjusting the design decisions, the timing of repair or restoration work, proposals for further monitoring.

Observations on the technical state of the structures of distillates tunnels during the construction work showed that the maximum vertical deformations were 11 mm, and the change in stresses in the cast-iron tubing of the lining reached 30 MPa (Figure 6).

Observation of the stressed-deformed state of the tunnel lining constructions allowed conducting construction and installation works around the clock.

At present, the tunnels of the underground are operated in a planned mode. Based on the results of inspection of defects that reduce bearing capacity, unacceptable rolls and other deformations of tunnel structures after the completion of construction have not been identified.

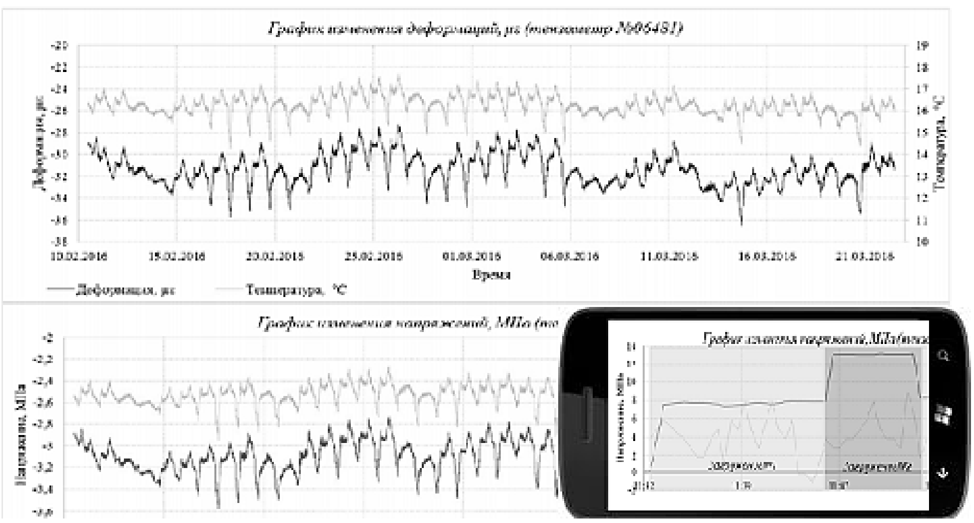

Figure 6 – Results of measurements and their display in specialized software

Conclusions

1. The Republican Unitary Enterprise «Belarusian Road Research Institute «BeldornNII» together with the Belarusian National Technical University prepared by the Ministry of Construction and Architecture of the Republic of Belarus from 01.01.2010 introduced technical codes of established practice (TKP EN) and national annexes to on the design of bridge structures, harmonized with the design standards of the European Union.

2. Harmonization of normative documents will contribute to improving the quality of construction, expanding the capacity of design and construction organizations to create bridge structures with a level of reliability that guarantees their safe operation during the project lifetime.

3. Experience in the operation of reinforced concrete structures of bridge structures shows that their reliability and durability depend on a large number of random factors that vary with time.

4. The design model of cross-sections of rod building structures is applicable for cross sections of arbitrary shape from any materials and is based on the use of transformed material deformation diagrams.

5. Refusal from the hypotheses of rock pressure and the transition to the theory of calculating underground structures on the basis of a deformation model using the deformation diagrams of materials of building structures and geomechanical models of the constituent soil massifs were introduced when monitoring the stress-strain state of structures.

6. The implementation of the adopted accounting regulations and the stress-strain state monitoring program allowed carrying out all construction and installation works on the operating tunnels in a short time.

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FORMATION OF THE FOUNDATIONS STRESS-STRAIN STATE FROM THE LOCATION CHANGE AND PILES PARAMETERS

The work of high-rise buildings with different piles arrangement foundations is investigated rationally and on a regular grid. The comparative analysis of the results is presented. Typical zones of foundation such as central, lateral, and angular ones are separated. The redistribution of efforts between piles and a grillage is shown. The interaction of piles with different lengths and the grillage in the foundations of high-rise buildings is considered. The numerical modeling of the «base – foundation – superstructure» system is performed. A finite-element model of high-rise buildings comples and a multilayer soil mass is developed. The choice of soil parameters for the deformation model of soil environment on the basis of their identification is shown.

Comparative results of calculations with data of field observations on bearing structures behaviour. The features of buildings complex base deformation are revealed.

***Keywords**: pile foundation, high-rise buildings, location of piles, piles with different lengths, numerical modeling.*

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ФОРМУВАННЯ НАПРУЖЕНО-ДЕФОРМОВАНОГО СТАНУ ФУНДАМЕНТІВ ПРИ ЗМІНІ РОЗТАШУВАННЯ ТА ПАРАМЕТРІВ ПАЛЬ

Досліджено роботу фундаментів висотних будинків при різному розташуванні паль: раціонально та за регулярною сіткою. Наведено порівняльний аналіз результатів. Виділено характерні зони у фундаменті: це центральні, бічні й кутові. Виявлено перерозподіл зусиль між палями і ростверком. Розглянуто взаємодію паль різної довжини й ростверків у фундаментах висотних будівель. Дослідження проведено за допомогою числового моделювання системи «основа – фундаменти – надземні конструкції». Розроблено скінченно - елементну модель комплексу висотних будинків та багатошарового ґрунтового масиву. Показано вибір параметрів ґрунтів для моделі деформування ґрунтового середовища на основі їх ідентифікації. Порівняно результати розрахунків з даними натурних спостережень за поведінкою несучих конструкцій. Виявлено особливості деформування основи комплексу висотних будинків.

Ключові слова: *пальовий фундамент, висотні будівлі, розташування паль, палі різної довжини, числове моделювання.*

Introduction. High-rise buildings are widely spread in the urban development. Pile foundations are often used for high-rise buildings due to heavy load of superstructures on the base. In those cases, engineers are faced with a number of tasks: a) modern methods of structures calculation should be developed; b) computer aided design (CAD) systems should be used to solve various problems of geotechnics; c) various calculation models involving materials deformation and the soil base nonlinear laws should be applied.

Promising direction of the pile foundations design is increase in the economic efficiency and the reliability of design solutions due to the use of a rational number of piles and the efficient use of their supporting ability. For this purpose, changes in a stress-strain state of the «base – foundation – superstructure» system with different numbers of piles, their different locations, and their different lengths must be simulated with regard for the actual parameters of a soil base, be carried out monitoring of bearing structures, be compared them with the results of calculations for accumulation of design experience.

Review of the latest research sources and publications. Often, piles are arranged on a regular grid in the foundation [1, 4, 8], considering that the foundation plate combines piles and evenly distributes the load between them above superstructures. But the design experience shows that it is not true. For example, A. Bartolomei in his works [4] notes that when loading on a group of piles, which is about to the boundary, the angular piles perceive 20% more than the middle pile of the extreme series, and 50% more than the central piles. This fact, he explains, that the friction forces are larger at the lateral surface of the angular piles than at the other piles of the foundation. Many scientists were engaged in research on the location of piles and the distribution of load between them: A. Bartolomei [4], I. Boyko [1, 5], V. Holubkov, A. Pyliahyn [8], R. Katzenbach [2, 3] and other. In works R. Katzenbach [3] considered a high-rise building, in which the foundation was designed and built with piles of different lengths. In the works of I. Boyko [1, 5] there are highlighted the joint work of the system «base – foundation – superstructure» elements, identification of soil parameters, choice of the correct model of soil foundation deformation, monitoring of bearing structures behaviour. Several decades ago Nikolayevsky [6] raised the pressing questions about the choice of soil model and the complexity of soil parameters determination.

Definition of unsolved aspects of the problem. Choosing the location of piles in foundations requires a designer of an integrated approach and advanced knowledge not only from the field of geotechnics, but also from the technology of laying foundation structures. Therefore, researching piles in the foundation is a difficult task. The rational position of the piles in the plan must be found, set their different lengths to align the internal efforts in the foundation and superstructure. In this paper, the results of the rational location of piles and on a regular grid are comparable, piles with different lengths are installed in zones of maximum efforts, identification of soil parameters is shown, the results of calculations are compared with the data of experimental observations. A detailed analysis of the stress-strain state of foundation structures allows to design reliable and economical foundations.

Problem statement. Purpose is to study the redistribution of the efforts in piles depending on the location of piles and their length. Changes in the stress-strain state of a pile foundation with short and long piles will be determined. The interaction of the pile foundation with the base within a model taken for calculations will be analyzed.

Calculations of a stress-strain state of building bearing structures with the soil base are performed by the finite-element method (FEM) in the three-dimensional statement with the use of the software «VESNA».

In simulation of soils deformation it is used a model of nonlinear elastoplastic soil medium based on the theory of dilatancy [6]. The non-associated law of plastic flow is used to determine the increment of plastic strains. The Mises-Guber criterion modified by Professor I.

Boyko [1] is used as the condition of a plastic flow, which provides the agreement of the stimulation results with experimental data in a wide range of loads on the soil medium.

Basic material and results. High-rise buildings are often constructed on pile foundation joined by grillage. The interaction of these buildings with the soil base has a number of specific features. It depends, in the first turn, on the ratio of the grillage width and the pile length, on the dimensions of the grillage plate, and on the number and arrangement of piles in the foundation.

The design scheme includes all elements of the building as a «base – foundation – superstructure» system. The finite-element model includes the volumetric soil massif described according to geological studies, pile foundation, and bearing superstructure of the building (Fig. 1).

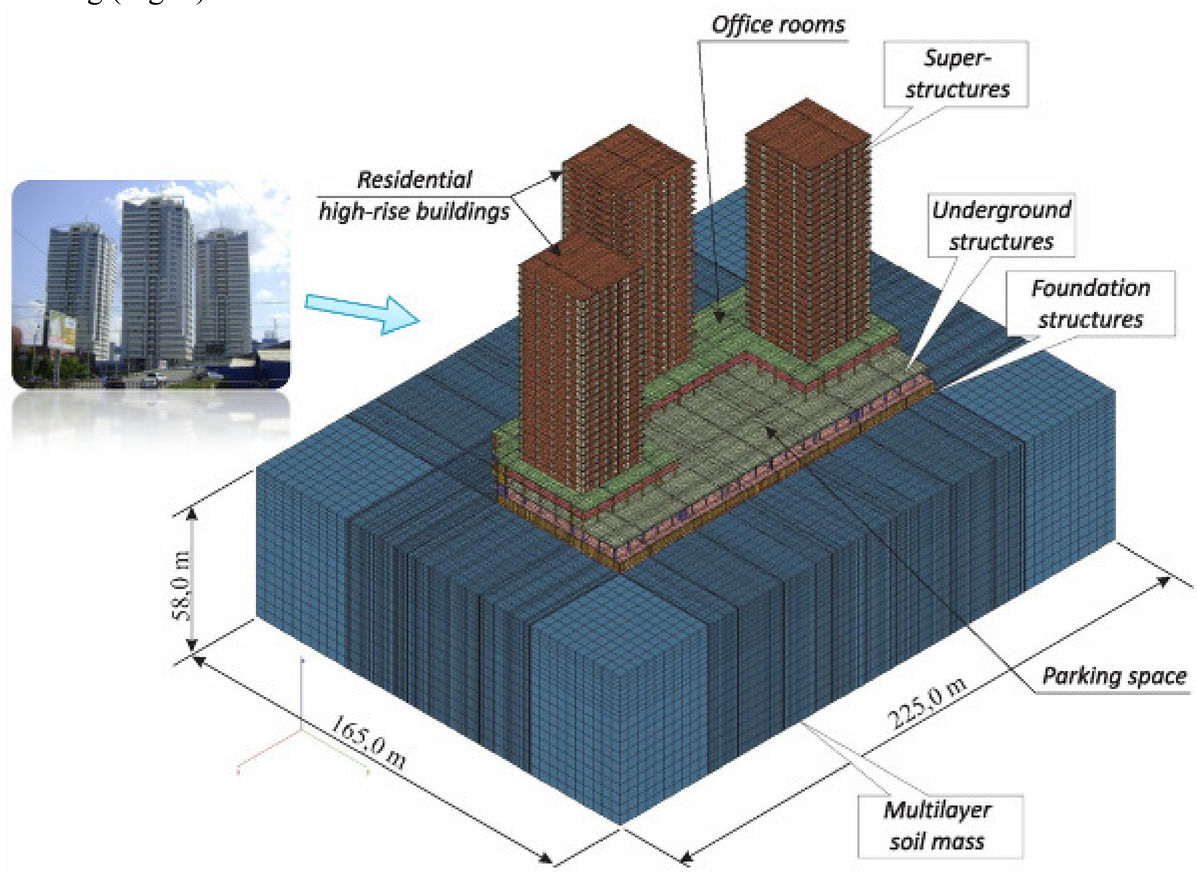

Figure 1 – Finite - element model of the building complex

The soil massif bottom is limited by a plane without vertical displacements (it is assumed that settlements of soil can be ignored at this depth). On the lateral planes, it was taken the boundary conditions that prevent normal displacements.

The location, capacity, and mechanical properties of soil layers correspond to the data of geotechnical studies.

The application of nonlinearly deformed soil base model results in the complexity of determination of many soil parameters that change during deformation. Standard methods of geotechnical investigations do not provide complete collection of soil parameters. Therefore, it is necessary to carry out additional studies and to implement the interpretation of parameters. Considering these facts, it is presented the results of piles tests as a «load – settlement» plot (Fig. 2) [7]. According to it, it is executed the modeling of piles tests, as well as the identification of parameters for the accepted model of soil medium deformation. It allowed to obtain the specified values of stresses in soil and the redistribution of efforts in foundation.

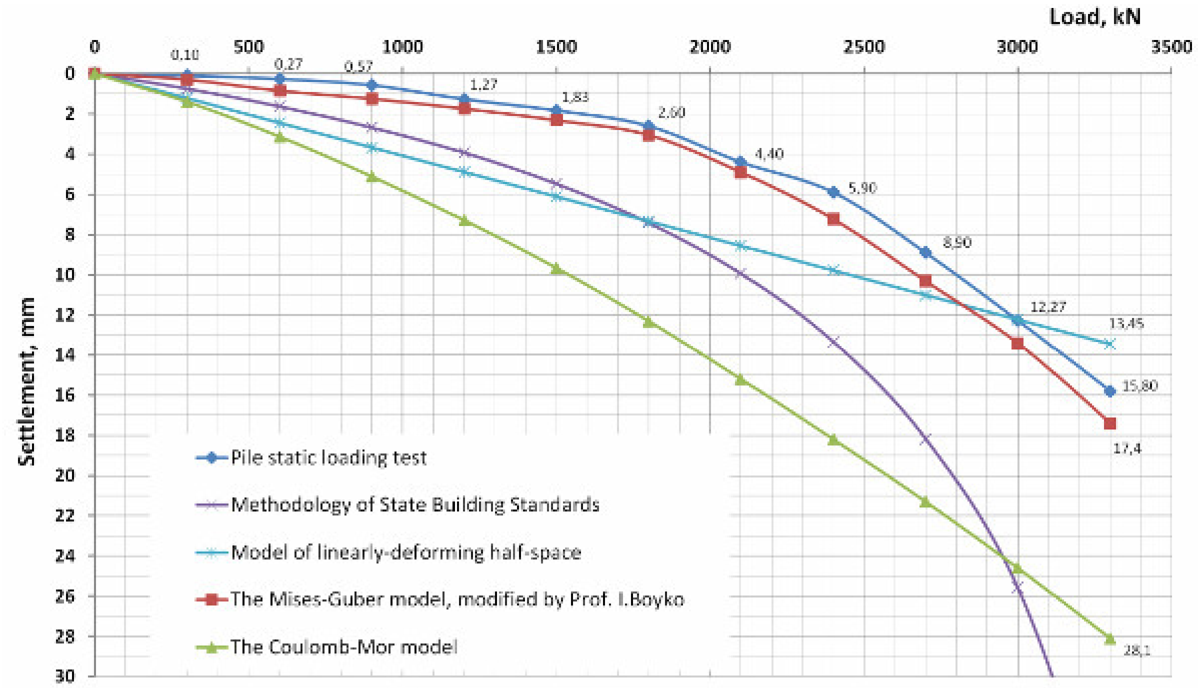

Figure 2 – Graph of «loading-settlement» of experimental pile using different soil models

As it is known, piles in a pile foundation are not equally loaded. It is confirmed by experimental data [3, 4] and the results of numerical modeling [5, 9]. In those works, it was noted that the piles of marginal zones are loaded most of all, and the load on the piles of central zones is the least. It depends on many factors, one of which is the location of piles within the grillage. In practice, location of piles on a regular grid with a given step can be observed very often. This choice is proper only at first sight. In fact, it has several drawbacks: peripheral piles are overloaded by 1.5-3 times as compared with the design load on a single pile, whereas the load on central piles, which share is about 50% in the foundation, is 50-60% of the design one on a single pile. It leads to a significant overspending of materials (concrete and reinforcement) in piles and, consequently, increases the cost of a construction. The approach for a nonlinear deformation of the soil base allows one to simulate a redistribution of efforts between piles, which are working up to the limit of their bearing capacity. The task of a designer is to find the optimal position of piles in the foundation. The typical zones of a foundation are angular, contour, and middle ones, where its piles work differently. The angular and contour zones together constitute the peripheral zone, which includes piles with the same name. In the middle zone, the middle piles and piles of the rigidity core are located (Fig. 3). It is noted that the lateral surface of peripheral piles works most efficiently. As for the middle piles, soil is clamped between the lateral surfaces of piles, which reduces or eliminates the lateral friction. Therefore, the middle piles are underloaded, and the peripheral piles are overloaded. In this case, the question arises about the efficient use of the bearing capacity of piles. It can be achieved by rational geometric arrangement of piles in the foundation (Fig.3,b). The criterion of rational location of piles is more uniform redistribution of efforts between the piles, providing the efficient use of piles material. Therefore, it is advisable to move the piles from the middle zone to the contour of the building and to dispose them under the load-bearing structures. This approach requires to increase the number of piles in peripheral zones and to reduce it in the middle zones. It increases the average distance between the piles, which leads to a more complete work of their lateral surface. An example of the problem and the main indicators are presented in Table 1.

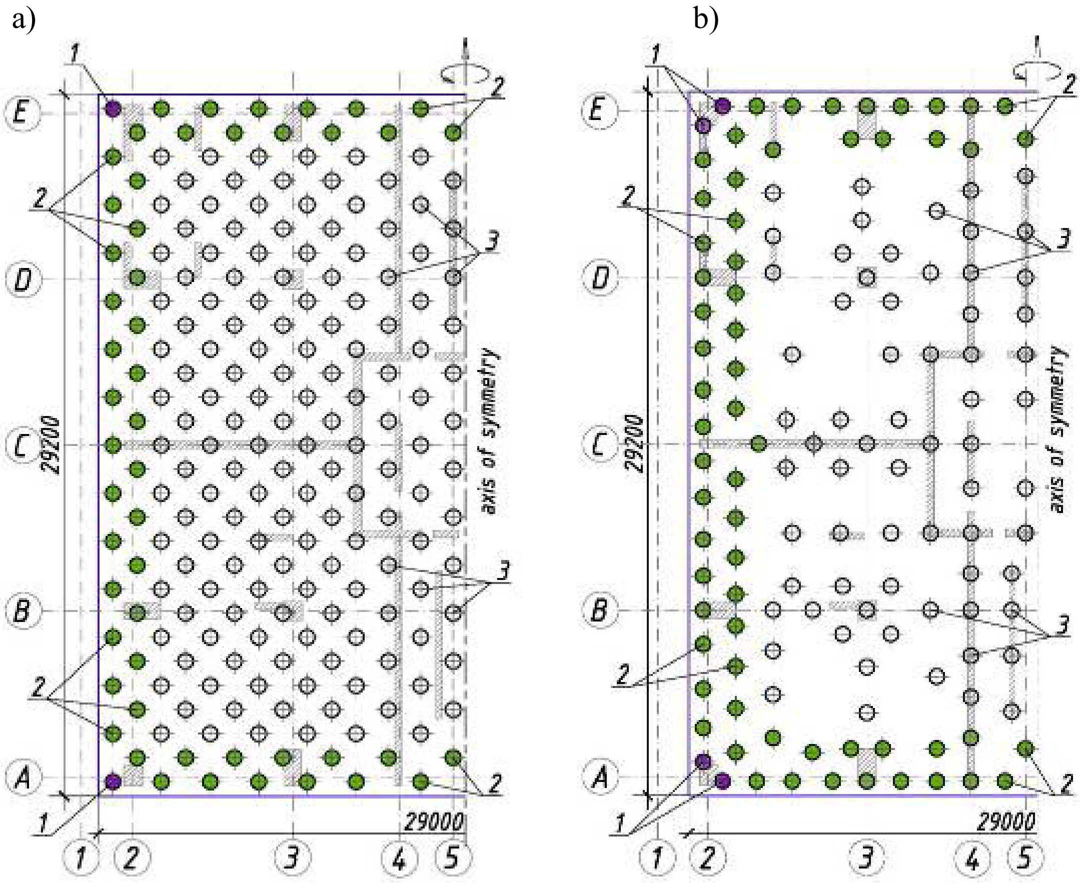

Figure 3 – Location of piles on a regular grid (a) and rationally (b) according to the zones: 1 – angular; 2 – contour; 3 – middle; (1+2) – peripheral

The executed studies showed that the angular piles and some contour ones, which are located on a regular grid, get loads that exceed their bearing capacity by ground and by material, which is unacceptable. The optimal arrangement of piles in plan was resulted in a more uniform redistribution of the efforts over foundation structures. In this case, the maximum effort in a pile decreases by 10-20%, and the minimum effort increases more than twice.

Table 1 – Comparison of the calculations results with different locations of piles

№	Indicator	Location of piles		Com-parison
		on a regular grid with given step	rational	
1	Total load of the building on the foundation, kN (%)	456 200 (100)		-
2	Load on the grillage, kN (%)	63 282.7 (13.9)	60 163.9 (13.2)	<1 %
3	Load on the piles, kN (%)	392 917.3 (86.1)	396 036.1 (86.8)	<1 %
4	Total number of piles, pcs (%)	392	252	↓ 140 pcs (↓ 35%)
5	Maximum effort in a pile, kN	3 729.6	3 470.0	↓ by 1.08 times
6	Average effort in a pile, kN	1 002.5	1 571.6	↑ by 1.57 times
7	Settlement of the grillage, cm	51.2	52.8	~3 %

Therefore, the efficiency of each pile bearing capacity use increases. The total number of piles can be reduced by 15-30% at their rational location, settlement of the foundation slab varies within 5%, and bending moments are changed within 10%. With such changes, the total bearing capacity of the foundation is not reduced.

Thus, the rational arrangement of piles makes it possible to efficiently distribute the load between the piles, to detect the zones of extreme internal efforts, and to make decision aimed at their reduction. It allows to design the reliable foundation constructions with optimum number of piles.

The investigation of stress-strain states of the «base – foundation – superstructure» system shows that the rational location of piles within the scope of a grillage gives the desired redistribution of efforts in piles not in all cases. The peripheral piles (especially, angular ones) remain problematic. In this connection, it is proposed to change the length of problematic piles for the regulation of efforts in them. It allows to use the bearing capacity of piles efficiently and to get optimal internal efforts in foundation structures. Indeed, the smaller the length of piles is, the less their bearing capacity is.

For this purpose, it is considered two variants with the variable lengths of piles: a foundation with shorter peripheral piles and a foundation with shorter middle piles (Fig. 4). These variants of foundations are compared with the foundation on piles with identical length. The results of calculations show a decrease in efforts in shorter piles comparatively with piles with identical length, that is logical. Therefore, short piles in the middle zone of a grillage are not recommended, because the peripheral piles are already overloaded. It is more efficient to reduce the length of piles in the peripheral zone. An example of such pile foundation variant is given by the high-rise building in Frankfurt-am-Main proposed by Professor R. Katzenbach et al. [3]. In the homogeneous basis, it was offered to arrange a pile field with significant increase in the length of piles to the building center.

a) b) c)

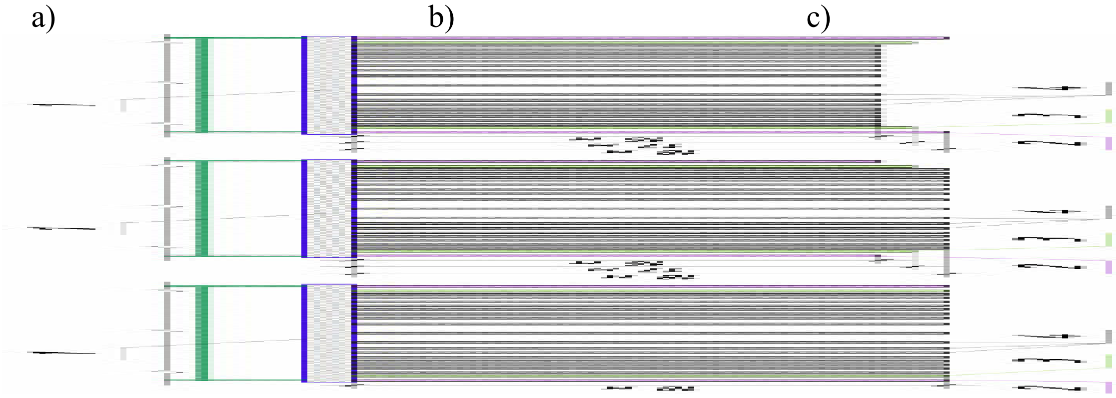

Figure 4 – Changing the length of piles within the typical zones of the foundation:
a) piles with identical length, b) shorter peripheral piles, c) shorter middle piles;
1 – grillage, 2 –peripheral piles of the 1st line, 3 – peripheral piles of the 2nd line, 4 –middle piles

The pile foundation with shorter peripheral piles allows reducing the effort in them up to 15% as compared with the foundation with piles with identical length (Fig. 5). The redistribution of efforts between the grillage and piles in two variants of foundations with shorter piles is almost unchanged on the whole or increases by about 1%. The change in the length of piles causes the increase in the settlement of a foundation plate within 5-6%. In general, the bearing capacity of the pile foundation remains unchanged. It is suggested that the rational choice of piles location in plan and of their lengths makes it possible to decrease the extreme values of internal efforts in the foundation structures and to reduce the total number of piles.

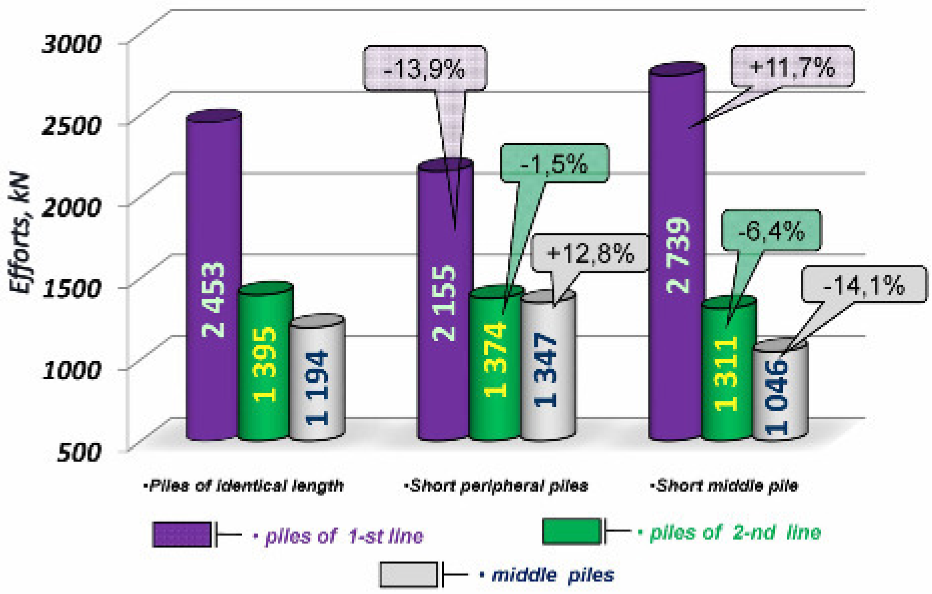

Figure 5 – Redistribution averaged efforts between piles depending on their length

After constructions calculations and their design, monitoring of the structures state in the process of buildings erection and at the stage of their operation is an important task. The results of experimental studies should be compared with the calculated values for the accumulation of design experience in difficult soil conditions, using rational location of piles (Fig. 6). At this facility, monitoring of bearing structures behavior at the construction stage was organized. The results are presented in Fig. 7.

The nature of settlements growth was confirmed by the actual observation, predicted in the simulation. The deviation between the experimental and calculated values of settlement is within the range of 6-25%, which satisfies the accepted accuracy. The difference between experimental and numerical deformations is explained by the fact that the actual load was not fully consistent with the design load at the time of the last measurement.

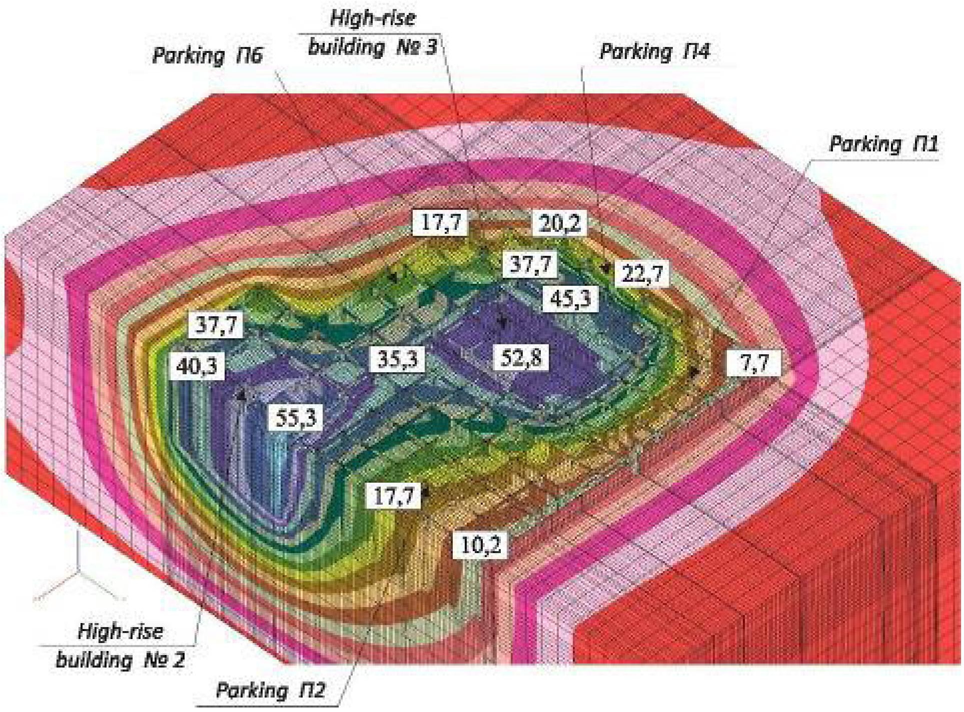

Figure 6 – The deformed scheme of the soil mass and basement part of the complex

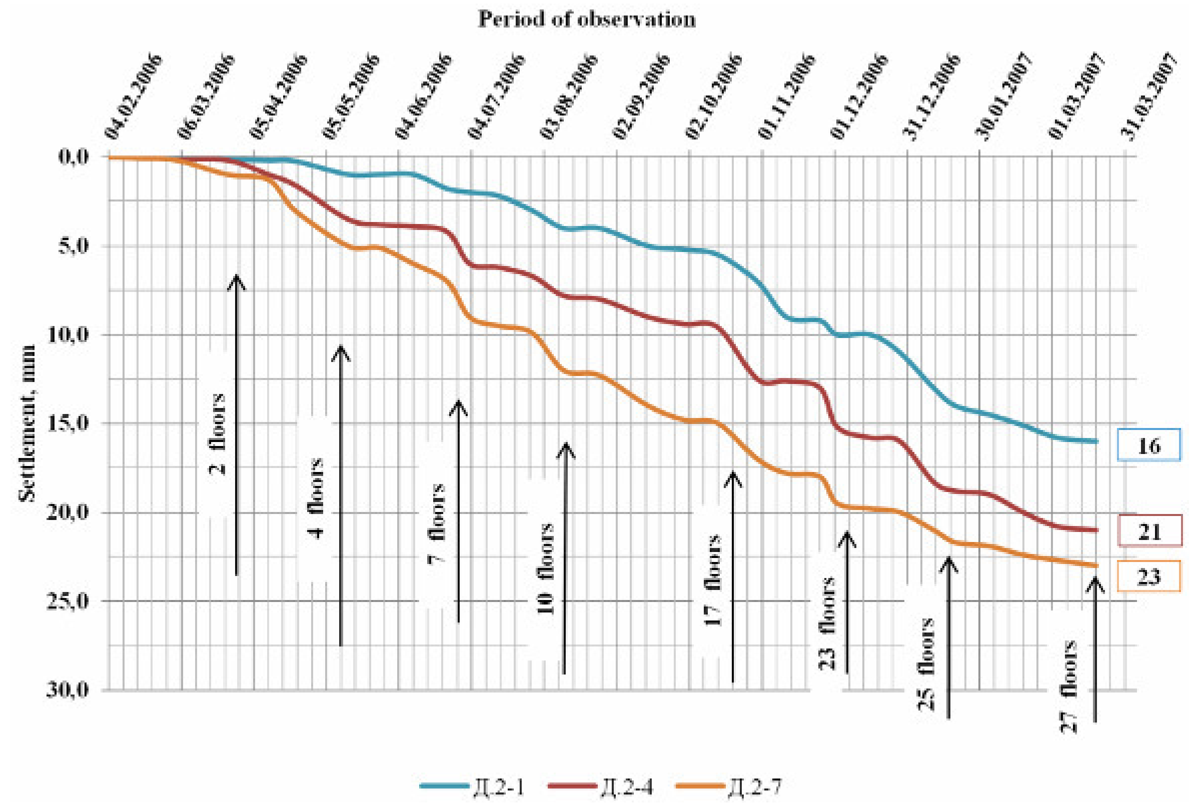

Figure 7 – Graphs of high-rise buildings bearing structures settlement

Conclusions. The reliable results of «base – foundation – superstructure» system numerical modeling can be obtained with regard for the identified parameters of a soil base model.

The redistribution of efforts in piles between typical zones such as the angular, contour, and middle ones should be considered. It has been established that the values of efforts in piles differ by three times in the angular and middle zones. Therefore, it is advisable to carry out complex calculations aimed at the determination of piles rational location in the pile foundation. It gives the economic effect concerning the consumption of materials.

It is found that the rational location of piles in a foundation allows load the piles uniformly and to decrease their total number by 15-30% as compared with the case of piles on a grid with regular step.

To adjust the efforts in the angular piles, it is proposed to change their lengths depending on the combination of loads. It is established that the pile foundation with shorter peripheral piles is more uniformly loaded, more efficiently uses its bearing capacity, and is more reliable as compared to the foundation with shorter middle piles.

It is shown that the instrumental observations of bearing structures behaviour are important in the design of high-rise buildings. It has been established that monitoring of constructions should be carried out from the beginning of foundation pit development until the completion of the construction, and after full load of the building with useful loads.

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COMPARISON OF THE SOIL DEFORMATION MODULUS VALUES RECEIVED BY THE LABORATORY AND IN SITU TESTS RESULTS

The soil deformation modulus largely depends on the obtaining method. There are various methods of determining E. In compression soil test a ring of small size is used. It causes a number of factors affecting the test results. The comparison of deformation modulus values obtained by in situ and laboratory methods is shown in article. It was believed for a long time that the oedometer deformation modulus of sands practically does not differ from the plate deformation test modulus, and therefore no transitional coefficients for this soil are given. However, it has been experimentally established that the results of oedometer tests of sands need to be corrected.

Key words: *oedometer deformation modulus, plate loading test modulus, in situ and laboratory soil tests.*

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ПОРІВНЯННЯ ВЕЛИЧИН МОДУЛЯ ДЕФОРМАЦІЇ ГРУНТУ, ОДЕРЖАНОГО ЗА РЕЗУЛЬТАТАМИ ЛАБОРАТОРНИХ ТА ПОЛЬОВИХ ВИПРОБУВАНЬ

Модуль деформації ґрунту значною мірою залежить від методу його отримання. Існують різні методи визначення Е. При компресійному стискуванні ґрунту використовується кільце невеликого розміру. Це обумовлює ряд факторів, що впливають на результати тесту. Наведено модуль деформації ґрунту, отриманий польовим і лабораторним методами, і порівняння їх величин. Довгий час вважалося, що одометричний модуль деформації пісків практично не відрізняється від штампового, а тому не наводяться перехідні коефіцієнти для цього ґрунту. Однак експериментально встановлено, що результати одометричних випробувань пісків потребують коригування.

Ключові слова: *одометричний модуль деформації, штамповий модуль деформації, польові та лабораторні випробування ґрунту.*

Introduction. The oedometer soil testing method is most common in our country to determine the deformation characteristics of soils due to its simplicity and accessibility, but it has a number of drawbacks. Therefore, its results should be monitored using other methods.

One of the most reliable methods for determining the dispersed soils deformation modulus (sandy, clay, organomineral and organic soils) is currently in situ stamp loading test metod. Usually, stamps of 5000, 2500 and 600 cm^2 are used, respectively, with diameters of 79,81, 56,43 and 27,65 cm. The first two sizes are used for testing soils in pits, pits and other mine workings, the smallest stamp is used for testing in boreholes. In the testing process a graph $S = f(p)$ is constructed. Then, in the initial section of the graph, the averaging straight line is drawn on the basis of the least-squares method by at least four test points. The deformation modulus are calculated on the basis of the Schleicher formula by the increments measured values of pressure on the stamp and the precipitation at a constant value of the coefficient ω = 0,79 for a rigid stamp and the values of the additional coefficient Kp = 1 ... 0,70, depending on the relative burial of the stamp and decreasing with its increase. [1]

Analysis of recent research sources and publications. In classical literature works, the basic ways of determining the deformation modulus are clearly established: plate loading, oedometer, pressuremeter, cone penetration and other tests. The carried out researches and analyzes of the obtained results can be found in the works of Agishev I.A., Ignatova A.I., Kornienko N.V., Zotsenko M.L., Vinnikova Y.L., Tertzagi and Pek, Joseph E. Bowles and many others.

These experiments were carried out on various soils, using various methods, but the question of the exact determination of the deformation modulus is still open.

Identification of general problem parts unsolved before. At the end of the 19-th century, it was began to assess the deformability of soils with the help of the plate loading tests. And only Terzagi transferred these tests to laboratory conditions, so like that oedometers appeared, and still occupy a leading position in deformation testing in our country. However, already in the sixties of the last century it was noted that the deformation modulus obtained by the results of odometer testing are underestimated and require correction [2 – 6].

These tests were widely conducted in KNUBC, SG SRIBC, PNTU, and others.

The widespread introduction of cone penetration simplified the evaluation of ground conditions, especially at the depth, but did not improve the accuracy of the deformation modulus as a result of the fact that the derivation of these values is based on correlation.

Today, great importance has been gained in obtaining the deformation modulus E by counting its value in terms of the real sediments values established by instrumental deformations observations of the building base in time.

The definition of E is a complex task that requires the complexity of research and the necessary analysis of data for soils obtained by different methods.

Problem definition. The aim of the study is to present a comprehensive evaluation of sandy soil based on the results of in situ plate loading tests and the oedometer soil tests in laboratory conditions.

Basic material and results. To date, plate loading tests are extremely rare and in quantities insufficient to obtain reliable soil characteristics. Even more rare are control tests by other methods. So to improve the foregoing, on the construction site in the Kiev region, complex tests were conducted by three different methods.

The laboratory tests

The soil specimens of natural structure were sampled at the in situ tests points of sections «a» and «b»to perform laboratory work.

The oedometer tests were performed with samples of fine sand broken structure samples having a specified (determined in the field conditions) density and humidity in standard odometers at loads up to 0.4 MPa at natural humidity (6 determinations). A typical compression curve is shown in figure 1.

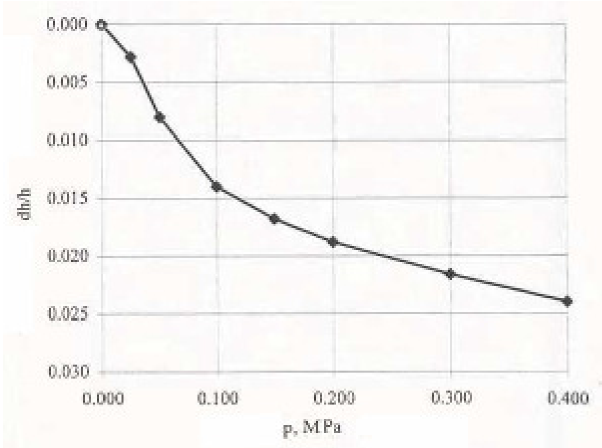

Figure 1 – Typical compression test graphic

Table 1 – Physical and mechanical soil test results

Stamp number	Soil sampling depth, m	Soil name	Mechanical characteristics		
			φ_0	c, MPa	E, MPa
53	0,2-0,3	dusty sand	26	0,010	20,9
54	0,2-0,3	fine sand	29	0,002	11,95
57	0,2-0,3	dusty sand	26	0,008	21,6
58	0,2-0,3	fine sand	28	0,004	22,25
61	0,2-0,3	fine sand	28	0,003	18,1
64	0,2-0,3	dusty sand	25	0,012	17,0
Average values			27	0,007	18,6

Stamp test

The static load tests with a round stamp of type I with an area of 5000 cm^2 are made in accordance with the Ukrainian normative documents [7]. On the central axis of foundation pits with a 10 m step on the surface of the foundation pits bottom at 24 points in two sections, to obtain a deformation modulus of bulk soils. Soil deformation modulus calculation according to the results of situ tests with load stages from 50 to 300 ... 350 kPa.

According to the data of plate loading test, the graphs of the dependence of the stamp residue on the pressure $S = f(p)$ are plotted, the values of Δp and ΔS are calculated on the averaging line.

The soil deformation modulus E, MPa is calculated for the linear part of this graph by the formula: $E = (1 - \nu^2) \times K_p \times K1 \times D \times \Delta p / \Delta S$.

The results of the work performed on sections «a» and «b».

The density of the skeleton average values of sandy soils are: $\rho_d = 1{,}79$ g/cm^3 (section «a»), $\rho_d = 1{,}78$ g/cm^3 (section «b»).

The average deformation modulus values of bulk soils, determined during in situ testing, are: E_{PLT} = 28 ... 31 MPa, the oedometer deformation modulus of bulk soils E_{oed} = 18,6 MPa. Thus, the correlation coefficient m_k = 1,6.

Typical dependence graphs of the soil deformation under the stamp from the load are shown in Figure 2.

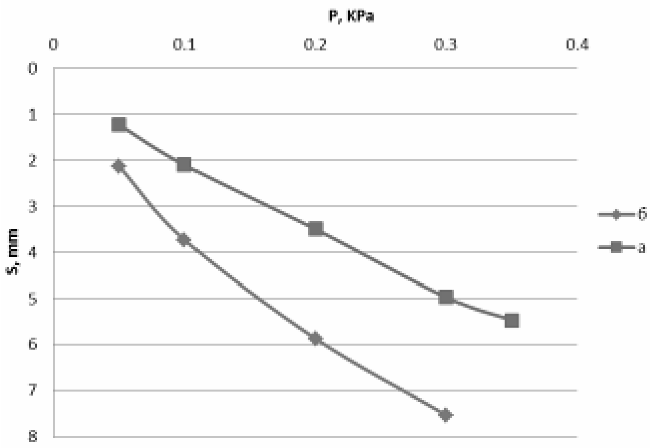

Figure 2 – Typical plate loading test graphics on sections «a» and «b»

Tests in sections «c» and «d»

In 2011, work was carried out on engineering-geological (geotechnical and geophysical) studies of foundation pits bottom deformations.

The following types of work were performed: seismic exploration by 2 profiles of 240 m in length (along the western and eastern branches of tracks). The results of seismic surveys are given below. And eight plate loading tests № 1 - 8. The test site is located between the sections «a» and «b», divided into two bands sections «c» and «d». Also it is given 11 additional tests of the same ground by plate loading.

Planning and construction work is carried out at the survey site. At the time of writing the report, the absolute marks of section «c» are 113.97 – 114.87 m, in section «d» 113.93 – 115.72 m.

The upper part of the geological section is composed of sands with a thickness of 0.5-2.4 m, under them lie upper and middle quaternary alluvial deposits lined at a depth of 31-32 m by the rocks of the Kiev suite of the middle paleogene (Eocene). At the explored depth of 5.4 m, the depositions of the flood plain facies ($a^2{}_3pr$) are involved – these are predominantly sands of different sizes with rare interbeds of sandy loam.

In order to clarify the determination of the soil deformation modulus values along the sections «c» and «d», seismic surveys were carried out on two profiles.

On the basis of the obtained velocity characteristics and density of soils, the value of Young's modulus (E_{dyn}) was determined and the modulus of deformation (E_{def}) of soils was calculated from the correlation dependence proposed by V.I. Bondarev: $E_{def} = 0.088 \times E_{\mathrm{dyn}} + 10.25$. The correlation equation is refined in the process of performing works on numerous objects as a result of comparison with the data of experimental works (plate loading).

The obtained average values of the deformation modulus were: «c» $E_{def} = 24.6$; and at the section «d» $E_{def} = 27.8$.

Plate loading testing. Plate loading tests with a stamp of type I – 5000 cm^2 are carried out according to [7] to obtain a deformation modulus of bulk soils. Test points from 1 to 8.

The soil deformation modulus calculation by the results of in situ tests was carried out with load stages from 0.05 to 0.3 MPa. 8 stamp tests of sandy soils were performed. The deformation modulus in section «d» amounted up to 22.1 - 29.4 MPa, in section «c» amounted to 18.1 - 21.7 MPa.

All the results of the in situ deformation modulus E definitions are summarized in Table 2.

Table 2 – Determination of the deformation modulus by in situ tests results

№ b/p	Test site	Deformation modulus E_{PLT}, MPa	№ b/p	Test site	Deformation modulus E_{PLT}, MPa
1	Section «a»	38,4	25	Additional	32,8
2	Section «a»	27,1	26	Additional	31,9
3	Section «a»	22,0	27	Additional	22,7
4	Section «a»	27,6	28	Additional	28,2
5	Section «a»	26,6	29	Additional	18,9
6	Section «a»	31,9	30	Additional	28,0
7	Section «a»	35,9	31	Additional	29,2
8	Section «a»	33,1	32	Additional	28,7
9	Section «a»	33,1	33	Additional	32,1
10	Section «a»	33,9	34	Additional	31,3
11	Section «a»	39,1	35	Additional	31,5
12	Section «a»	31,0	Av. val.		29,4
13	Section «b»	26,9	36	Section «c»	18,1
14	Section «b»	27,4	37	Section «c»	18,9
15	Section «b»	28,9	38	Section «c»	21,7
16	Section «b»	24,9	39	Section «c»	18,2
17	Section «b»	25,8	Av. val.		19,2
18	Section «b»	22,6	40	Section «d»	22,1
19	Section «b»	28,5	41	Section «d»	26,5
20	Section «b»	27,8	42	Section «d»	27,2
21	Section «b»	28,3	43	Section «d»	29,4
22	Section «b»	27,8	Av. val.		26,3
23	Section «b»	31,2	Av. val.	(Common)	28,1
24	Section «b»	35,0			

It should be noted that according to seismic data and the results of in situ tests, the deformation modulus along section «c» is somewhat lower than in section «d».

Based on the results of previous determinations and taking into account the deformation modulus values obtained in the present investigations, Tables 3 and 4 and histograms (Fig. 3 and 4) are constructed for the distribution of deformation modulus values determined by the in situ plate loading tests.

Table 3 – Deformation modulus determination by in situ plate loading tests results

Deformation modulus E_{PLT}, MPa	Number of values
18-20	4
20-25	6
25-30	18
30-35	13
38,4	1
39,1	1

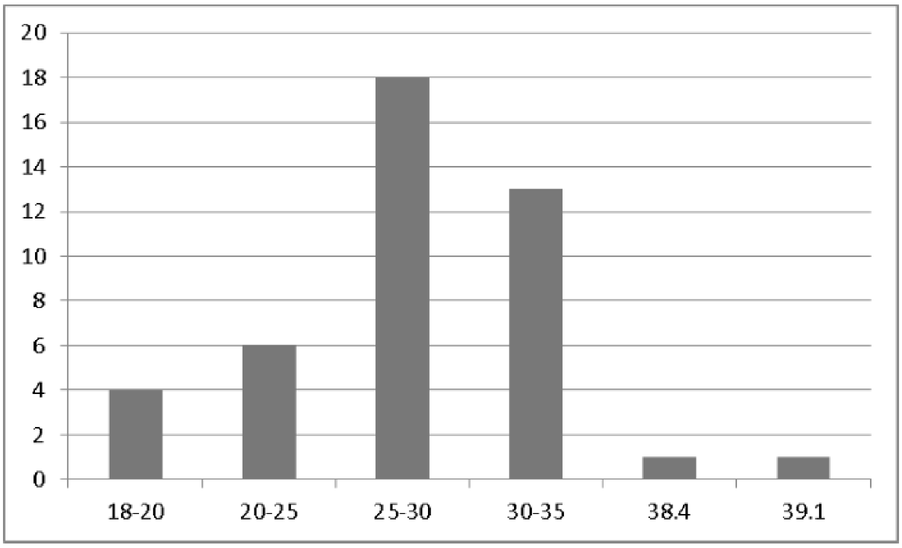

Figure 3 – The diagram of the E_{PLT} definition is quantified

Table 4 – Determination of the deformation modulus obtained by the in situ plate loading tests results (in the percentage)

Deformation modulus E_{PLT}, MPa	Number of values
18-20	9,30%
20-25	13,95%
25-30	41,86%
30-35	30,23%
38,4	2,33%
39,1	2,33%

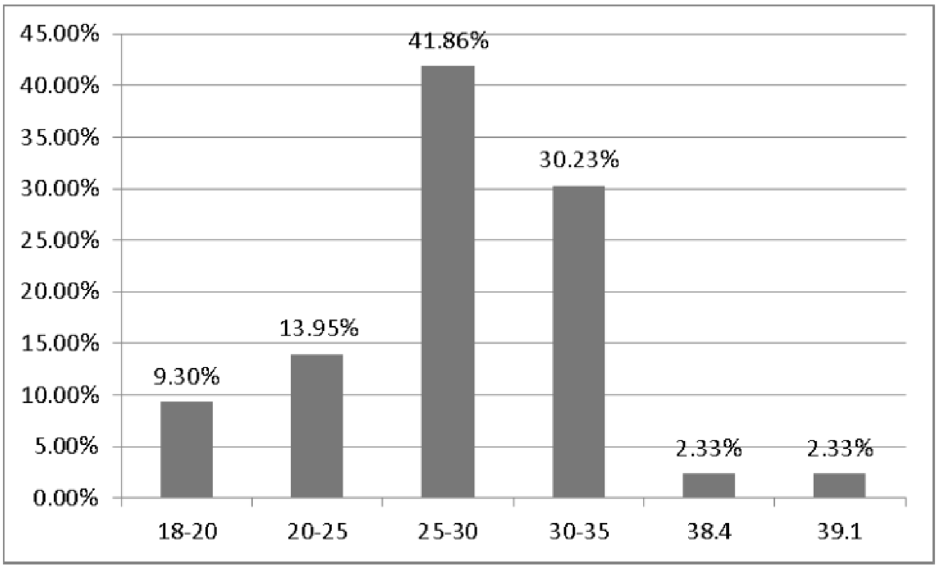

Figure 4 – Diagram of the E_{PLT} definition as a percentage definitions of the deformation modulus by plate loading tests

As a result of the above, it can be concluded that the soil testing are sufficiently well-converged by different methods. Histograms show that according to two methods of investigation, the values of the deformation modulus in the range 25-30 MPa constitute the largest percentage of all the obtained values. It indicates the correctness and possible application in future the seismic survey method in combination with other methods for the soil investigations, as more timely, which is advisable under conditions of radiation pollution.

Conclusions.

1. For a long time it was believed that the oedometer deformation modulus of sands practically does not differ from the modulus by the results of plate loading tests, and therefore no transitional coefficients for this soil are given. However, it has been experimentally established that the results of oedometer tests of sands need to be corrected. The difference between the plate loading and the oedometer moduluses was 1.5 times. The average values were: E_{PLT} = 28.1 MPa, E_{oed} = 18.6 MPa.

2. The problem of obtaining a reliable, E should be solved considering the soil structural strength and the stress-deformation state formation of the base within the thickness of the layer.

3. The deformation of the soil sample is uneven in height of the sample. The deformation of the crumbling significantly affects the value of the deformation modulus. The crumbling zone also occurs when testing the soil with a plate load, it is commensurate with the crumbling zone in the odometer. When calculating the sediment of the test base, the crumbling of the ground at its base can be neglected, but in the case of compression tests because of the small sample dimensions, the deformation of the crumbling zone significantly underestimates the values of the deformation modulus, overstates the draft of the building and, as a consequence, increases the cost of construction.

4. In the foreign literature [8, 9], in order to avoid inaccuracy in the determination of the deformation modulus, a differentiated definition is adopted, and in oedometer tests, it is often used with some approximation along the unloading compression curve branch.

5. The correlation coefficients m_k given in [10] are established in Ukraine as general average values for clays, loam and sandy loam, do not take into account the structural strength, loading range and depend on the type, density and condition of the soil.

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IRON ORE MINING GEOMECHANICAL PROBLEMS IN UKRAINE

The paper deals with problems in Ukrainian iron ore mining industry resulted from enterprises reaching deep levels of mining. There are also described main causes of hazardous situations due to geomechanical factors at such depths, dynamic forms of excessive rock pressure manifestations, and types of external factors affecting the state and behavior of load-bearing elements of mining and technological objects below the surface to enable mining operations. The paper provides recommendations on creating a specialized geomecanics support system for mining enterprises which is based on labour safety risk-management principles as well as it presents geomechanics tasks to be solved by the system.
Keywords: *geomechanics, problems, iron ores, mining industry.*

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ГЕОМЕХАНІЧНІ ПРОБЛЕМИ ЗАЛІЗОРУДНОЇ ГІРНИЧОДОБУВНОЇ ПРОМИСЛОВОСТІ УКРАЇНИ

Розглянуто проблеми, що виникають у залізорудній гірничодобувній промисловості України у зв'язку з досягненням підприємствами з видобутку залізних руд великих глибин здійснення добувних робіт. Описано основні причини виникнення небезпечних ситуацій, обумовлених геомеханічними факторами, які діють на великих глибинах; динамічні форми прояву надвисокого гірського тиску на таких глибинах; види зовнішніх геомеханічних факторів, що впливають на стан і поведінку несучих елементів гірничотехнологічних об'єктів, які зводяться у надрах для забезпечення можливості виконання гірничих робіт. Надано рекомендації щодо створення спеціалізованої системи геомеханічного забезпечення гірничодобувних підприємств, заснованої на принципах ризик-менеджменту у сфері безпеки праці. Наведено основні задачі з геомеханіки, які повинна розв'язувати така система.
Ключові слова: *геомеханіка, проблеми, залізні руди, гірничодобувна промисловість.*

Introduction. Mining is one of Ukrainian key industries. It is characterized by the fact that large amounts of the Earth interior are involved into commercial production, and thus its geomechanics is disturbed by mining. The scope of the disturbance is very great; its effects are irreversible and they are hazardous for both mining enterprises and the environment.

Negative effects become significant at great depths of mining characterized by considerable changes of geomechanics and excessive rock pressure. Thus, the risk of dynamic rock pressure manifestations and emergencies rises substantially.

Such situation requires development and implementation of efficient and reliable measures to decrease emergency risks and creation of reasonable safe conditions of mining.

Analysis of latest research and publications. Review and analysis of home and foreign publications on the problem show the ultra-low number of studies on the problem solution.

Recent developments in creating safe mining conditions focus on the only, yet very important, direction – building a computerized system of continuous monitoring of rock mass behavior affected by changes in natural geomechanics due to mining [0]. This monitoring enables prevention of emergencies and accidents due to prompt detection of zones with observed intensive mechanical stress concentration and rock deformations that may soon go beyond the critical boundary while mining. The monitoring results enable timely evacuation of miners from dangerous zones or suspension of operations before emergencies.

Such developments are currently under way in the USA, Canada, South Africa, Germany, Australia, Poland, and Czechia [2, 3]. The mentioned countries have mining enterprises operating at considerable depths (open pits - up to 1.5 km, underground mines - up to 3.0 – 5.0 km).

However, in practice rock mass behavior monitoring alone is not a sufficient means of preventing emergencies and accidents caused geomechanics. Even if the monitoring is applied, there are still accidents due to geomechanics that result in the staff injuries and fatalities [4].

Highlighting previously non-settled issues of the general problem. Development of efficient measures to create safe conditions of deep iron ore mining requires, first of all, a thorough study of the whole range of dangerous geomecanical factors and reasons of their occurrence. It should be noted that in Ukraine this aspect of iron ore mining geomechanics is underinvestigated. This fact complicates further mining at great depths and, as a result, development of Ukraine's iron ore mining industry in general.

Problem statement. Considering mentioned above, the authors were determined on reviewing and analyzing the situation in the Ukrainian mining industry in terms of deep iron ore mining geomechanics focusing on the following: determining current geomechanics conditions at great depths; determining types of geomechanics hazards at the depths, determining the tasks to be solved to decrease emergency risks.

Material presentation and work results. Iron ore is one of basic production resources of the contemporary industry. Ukraine is a large-scale producer of commercial products of this type. The development strategy and policy of Ukrainian economy are directly connected with further functioning and development of the iron ore mining industry [5].

At present Ukraine houses one of the largest iron ore mining industries in the world. The country mining capacity ranks 7th out of 52 iron ore mining countries, in terms of iron ore raw materials commercial types production. This industry considers 8 – 10% of the country GDP and is one of the major foreign exchange earners as nearly 40% of its produce is exported [6]. Besides, the iron ore mining industry is a major supplier of the basic raw materials (iron ore) for the national metallurgy. Iron ore mining and metallurgical industries form Ukraine's integrated mining and metallurgical complex (MMC of Ukraine).

Such opportunities are ensured, first of all, by the country significant mineral raw material base. The reserves of various iron ore kinds make Ukraine one of the world leaders in this field. 80 large proved deposits on the country territory contain 14% of the world iron ore reserves.

In Ukraine there are 10 large iron ore mining enterprises operating in surface and underground modes (11 open pits and 9 underground mines).

In Ukraine, iron ore has been mined for 120 years. The largest scale mining started in the 1950 s. It should be noted that since the very beginning of this period, iron ore had been extracted very actively accompanied by the depth increase. As a result, by 2000 main enterprises of Ukraine had reached great depths of mining (350 – 460 m in open pits, 1400 – 1500 m in underground mines). These depths are planned to reach 500 – 600 m and 1600-1800 m respectively. According to this, all iron ore enterprises of Ukraine are rated as «deep» [7].

Development at such depths is accompanied by serious geomechanics challenges: quick durability loss in structural units of mining and technological facilities caused by excessive rock pressure; specific behavior of rocks under developing mechanical stresses close to their strength limits; complicated or even impossible forecasting of behavior of mining and technological facilities and rocks at great depths under changes in their natural geomechanics.

These conditions lead to significant rise of risks of hazardous rock pressure manifestations (their rate increasing with depth) and the hazard level rises as the depth increases.

All the dangerous phenomena occur in mining and technological facilities that are built for mining purposes: open pit walls, benches, trenches, mining blocks and panels, mine workings, chambers. They house workplaces, mechanical means and equipment.

The mentioned facilities are intended for various activities, have various structures and building technology and operate in different ways. However, they all have an important common feature – load-bearing structural elements. These elements are parts of facility constructions (open pit wall faces, inter-block/inter-panel pillars, underground chamber ceilings, undercut massifs, arch roofs of workings) that carry the main rock pressure load and prevent the whole facility construction from failure. The «bearing capacity» feature [8] of the elements must meet the strictest requirements as the function they fulfil in terms of safety of their operation is very essential for preventing possible catastrophic consequences in case of their functioning failure.

It is given more detailed consideration to geomechanics hazards at great depths.

In their natural state (before human intervention) rocks occur in the condition of triaxial compression. This does not lead to dangerous consequences as the rock cannot be deformed even under high loads. However, at great depths the rock massif accumulates considerable mechanical stresses.

If the monolithic rock massif is distorted due to the human impact (ore mining), there appear voids and free surfaces. These disturbances are concentrators of mechanical stresses. On the disturbance interfaces there develop stresses that can be 10 – 20 times greater than those in natural conditions. The developed rock deformations result in high risks of hazardous dynamic manifestations of rock pressure. Rock failure forms may be as follows: rock bumps (the most dangerous form similar to the earthquake), large scale rock slides, rock outbursts, rock displacement, sheeting, ultra high local deformations [9]. Places, time and scale of such manifestations are extremely hard to forecast. The above mentioned phenomena form particularly dangerous conditions of mining as workers may possibly be trapped in a rock failure zone.

In underground mining beginning with 200 – 400 m levels the rock fall trough changes its shape from a cone to a fissure above which there is a hanging wall crystalline rock console.

This also results in redistribution of loads in rocks around the trough. The growing depth of mining leads to mechanical stress accumulation in the console as it is underlain by unconsolidated rocks and rests upon them. Loads on unconsolidated rocks result in their deformation and the console slide similar to deformation of a wall rested on the foundation of non-coherent materials. The scale of this process contributes to generation of «a rock bump» – one of the most hazardous forms of rock pressure manifestations [10].

At great depths, when technogenic voids appear, fragile crystalline rocks that have accumulated great stress begin to discharge and undergo deformation (similarly to a compressed spring released), i.e. «stress release» is generated [11]. This phenomenon may cause such great deformations that rocks lose strength, the bearing capacity of large rock massifs decreases and thus jeopardizes bearing elements of mining and technological facilities that cannot resist external forces any more.

Under actual mining conditions there is another hazardous aspect consisting in the fact that bearing elements of mining and technological facilities are influenced not only by rock pressure (which is static in nature) but also by a number of dynamic factors such as shock and seismic waves of explosions and earthquakes.

Bulk blasting shock waves result from blasting operations during ore breaking and run to relatively small distances (several scores of meters) but in terms of force impact they are one of the most powerful destructive factors for both ore and mining block elements.

The blasting shock wave is about several centimeters long, it propagates within a massif and has a frontal forward pressure zone (the first wave half-period) and a rear direct stress zone (the second inverse half-period). The wave is dangerous due to the fact that on hitting an exposed surface it bounces back and its direct stress zone causes breaking stresses in rocks. This results in ore breaking in a stope and may destroy bearing elements of mining blocks if there are initial stresses in them.

The bulk blasting seismic wave has a specific character. It is generated by explosive gases impact on the rock mass when there are factors stretching it in time (short-delay charge initiation, shock wave reflection from massif disturbances). This transforms a shock wave into a seismic one and changes its frequency. The seismic wave is hundreds of meters long and its danger consists in the fact that bearing elements of mining and technological facilities possess certain elasticity and may resonate with seismic wave oscillations and fail.

The phenomena resulted from bulk blasting require thorough and reliable determination of optimal parameters of drilling and blasting operations with simultaneous provision of ore mining safety and efficiency. The critical importance of the correct solution of this problem has generated a new research direction aimed at developing methods and ways of enhancing mining parameters [12].

The physical action of seismic earthquake waves is similar to seismic bulk blasting waves but their power and duration are considerably greater. These waves are generated by earthquakes in zones of great tectonic faults and local earthquakes.

Ukraine lies within several dangerous natural seismic zones. «The Vrancea zone» located at the junction of the South (Romania) and East (Ukraine) Carpathians is the most hazardous of them. Ukraine's territory is located within M 4 – 6 seismic district of this zone. «The Vrancea zone» is extremely active, 30 earthquakes magnitude 6.6 – 7.0 were registered in the XX century, and some of them had catastrophic consequences. Bukovyna and the Crimean-Black sea zone are also considered an earthquake endangered area with magnitude 5 – 6 and 8 – 9 respectively. There are also several seismic zones (earthquake magnitude up to 4.0 – 5.5) in the platform part of Ukraine «the Ukrainian shield» to which all its iron ore deposits are limited. One of them is Kryvyi Rih iron ore basin with 80% of all Ukraine's iron ore deposits and 90% of the country's mining industry.

The following events testify to the geomechanical danger of earthquakes. On November 29, 2016 the M 4.4 earthquake caused a collapse of the «Rudna» mine in Poland that left 8 miners dead and 16 miners were trapped 1100 m underground. On March 9, 2005 the M 5 earthquake near Stilfontein (South Africa) killed 5 «DRD Gold» miners and 42 people were trapped 2000 m underground. On August 5, 2017 the M 5.3 earthquake in Orkney killed 1 miner and caused mass destruction.

Another dangerous phenomenon related to the seismic activity is so called «induced seismicity» [0]. It is caused by the coupling of the seismic action of technogenic factors (bulk blasting) and local earthquakes.

The coupling is seen in seismic areas within which mining is carried out and mechanical stresses can be accumulated naturally. If the accumulation reaches a certain level, bulk blasting seismic waves may trigger a destructive earthquake.

The complicated great depth geomechanics problem demands creating «The Geomechanics Support System for iron ore deposit mining». Way back in the sixties – seventies of the XX century leading specialists in iron ore mining geomechanics forecast reaching great depths and emphasized the urgency of the system. In late 90s, the problem became a burning issue in terms of further iron ore mining [14, 15]. However, no effective measures to develop this direction have been taken so far.

«The Geomechanics Support System for iron ore deposit mining» means solution of problems of ensuring durability of mining and technological facilities, forecasting and monitoring impacts of mining on stressed and distorted rocks and engineering structures during the building period, operating and dismantling mining and technological facilities. The main objective of the geomechanics support is prevention of accidents and emergencies in mining.

National mining enterprises do not currently have the system of the kind. Thus, its development becomes one of the prime tasks of ensuring accident-free iron ore mining in Ukraine.

The scale and complexity of tasks to be solved in creating the system require relevant institutional, regulatory, methodological, engineering, tooling and organizational support.

The system creation should be based on principles of risk-management in useful mineral mining safety [16] which has become a central one in advanced countries [17].

Creation of the system requires the following steps: development of the legislative and regulatory framework of the risk-management functioning in the field of geomechanics support of mining enterprises; development of the methodological principles of solving geomechanics tasks based on modern approaches and means; creation of the automated system for monitoring rock geomechanics; development of information support for the risk-management system to provide the latest research data in the field and increase efficiency and reliability of its functioning.

The system of this kind should function according to the following chart: monitoring the situation in the deposit area planned for mining and collecting relevant data; modeling real situations; determining possible hazardous geomechanical factors on the basis of the modeling results; forecasting potential negative risks and consequences of the mentioned factors; developing measures to prevent hazardous processes and risks of accidents and determining necessary parameters of these measures; solving organizational, engineering, technical, and economic tasks in implementing the measures; monitoring the situation after the measures are implemented and assessing their actual effectiveness; creating the analytical database of hazardous situations and their types, taken decisions and their results; forming the data-based decision support system for hazard forecasting, preventive methods development and selection of relevant decisions on accident consequences elimination, development of emergency elimination plans.

Conclusions. The review and analysis of publications and practical data on iron ore mining geomechanics performed in this article enable the following conclusions:

1. The current situation in Ukrainian mining industry is considered dangerous due to the enterprises reaching great mining depths.

2. The hazardous character of the situation results from the specific character of the natural geomechanical disturbances and increased risks of mass dynamic excessive pressure manifestations.

3. Forms and reasons of manifestations are various and depend on specificity of great depth geomechanics, character and parameters of mining operations and processes and external factors.

4. The great number of accidents caused by geomechanics factors testifies to the fact that current methods of solving geomechanics tasks and securing safety of mining operations do not provide the sufficient durability of mining and technological facilities or prevent geomechanics hazards.

5. The situation is also conditioned by obsolete approaches and theoretical principles on the basis of which methods and means of securing labour safety.

6. The most efficient method of settling the problem is creating a specialized geomechanics support system for iron ore deposit mining based on risk-management in the field of iron ore deposit mining geomechanics.

7. The paper recommends on the order of the mentioned system creation and tasks to be solved for this purpose.

8. The paper presents the order and reveals the essence of tasks in the geomechanics support of national mining enterprises with the view of ensuring safe working conditions in mining at great depths.

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INFLUENCE OF DISTANCE BETWEEN A RETAINING WALL AND THE EXISTING BUILDING ON STRESS-STRAIN STATE OF THE SYSTEM «RETAINING STRUCTURES – SOIL MASSIF»

The results of work research of engineering protective structures in a densely built–up area with difficult engineering–geological conditions have been presented. The modeling of the geotechnical problem of deep excavation protection using a three-tier retaining wall has been performed. The task of mutual influence of existing building and deep excavation with the change of distance between them is solved. The grafs of displacement's dependence several tiers of retaining walls from the distance to an existing building have been presented. According to these data, a plot of the dependence of displacements of separate tiers of retaining walls from the distance to an existing building is constructed. The problem is solved by the finite element method using a nonlinear model of a solid soil environment. The character of the formation of zones of potential slip surface slope is revealed. The dependence of bending moments of the retaining walls from the distance to the existing building is shown. A safe location of an existing building to a deep excavation has been substinated.

Keywords: *numerical modeling, engineering protective structures, finite element method, horizontal displacements, bending moments.*

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ВПЛИВ ВІДСТАНІ МІЖ ПІДПІРНОЮ СТІНОЮ ТА ІСНУЮЧОЮ ЗАБУДОВОЮ НА НАПРУЖЕНО-ДЕФОРМОВАНИЙ СТАН СИСТЕМИ «УТРИМУЮЧІ КОНСТРУКЦІЇ – ҐРУНТОВИЙ МАСИВ»

Наведено результати дослідження роботи інженерних захисних конструкцій на щільно забудованій території зі складними інженерно-геологічними умовами. Виконано моделювання геотехнічної задачі захисту глибокого котловану за допомогою триярусної підпірної стіни. Розв'язано задачу взаємовпливу існуючої забудови та глибокого котловану при зміні відстані між ними. Подано графіки залежності переміщень окремих ярусів підпірних стін від відстані до існуючого будинку. Задачу розв'язано методом скінченних елементів із використанням нелінійної моделі суцільного ґрунтового середовища. Виявлено характер утворення зон потенційної поверхні ковзання схилу. Показано залежність згинальних моментів підпірних стін від відстані до існуючого будинку. Обґрунтовано безпечне розташування існуючого будинку до глибокого котловану.

Ключові слова: *числове моделювання, інженерні захисні конструкції, метод скінченних елементів, горизонтальні переміщення, згинальні моменти.*

Introduction. At the present stage of housing development, there is a prevalence of tendencies to increase the building density of historically formed districts of cities. Considering the density, the most rational in these conditions is the exploration of underground space, in which you can arrange engineering communications, parkings, warehouses, shopping and entertainment complexes, thus freeing the ground area. Thereby a number of urgent engineering problems arise due to the deepening the levels of basements of existing buildings and the excavation of pits to a depth of 10 – 15 m and more, and as a consequence, changes in the stress-strain state of neighboring buildings. In many cases, existing buildings undergo significant deformations (cracks in walls and foundations, structural distortions, etc.) that are caused by uneven subsidence that develops after the start of construction works of the zero cycle of new buildings and continue to evolve during the exploitation phase. Especially increases the risk of such deformations during construction on the bases, composed of weak soils. In these circumstances, the task of reducing the impact of new construction on the stress-strain state of the system «ground base – foundation – overground constructions» of existing buildings becomes of great importance.

Analysis of recent studies and publications. To date, a large number of methods for calculating the protective structures of deep excavation have been developed, but their use often leads to the receipt of significantly different, contradictory results to each other. The presence of surrounding buildings greatly complicates this process. Inaccurate calculation of the enclosure structure leads to unpredictable impact on surrounding construction. For solving the problem of predicting the impact of new construction on existing structures in densely built up area, the scientific works of M.L. Zotsenko [4], [5], Yu.L. Vynnykov [3], I.P. Boyko [7], P.I. Yakovlev [6], V.S. Nosenko [8], C. Capraru [9].

Definition unsolved aspects of the problem which the article covers. Construction of objects with underground facilities in a densely built up area requires the installation of a deep excavation, which should be from the conditions of preservation in the initial state of surrounding construction. To do this, it is necessary to fulfill the forecast of the impact of digging the foundation pit on the stress-strain state of this building. The difficulty in implementing such forecast lies in the fact that many initial data needs to be taken into account: the configuration and condition of the surrounding building, the parameters of the pit, the load from existing buildings, the uneven layering of soils, and the phased construction work. Simulation of such complex geotechnical processes requires simultaneous consideration of many factors, which can only be achieved by using the finite element method.

Formulation of problem. The purpose of this paper is to investigate the mutual influence of the installation of a deep excavation high-rise residential complex and existing buildings with varying the distance between them. To achieve the goal with the help of the finite element method, the modeling of the system of «retaining walls – a soil massive – existing building» was performed. The calculation was performed taking into account the formation of a strained-deformed state of the soil mass during the installation of retaining walls. At the same time, the soil array was considered as a solid nonlinear medium. The analysis of the formation of the strained-deformed state of the soil massif, protective fencing and foundation structures of the building is carried out. The horizontal movements of the top of the bearings of the retaining walls, the bending moments, as well as the vertical displacement of the foundation of the existing house were compared. The patterns of formation of the potential zone of sliding surface of slope and role in its formation of existing building with decreasing distance to the protective structure of the foundation pit are revealed. The rational arrangement of an existing building in relation to the deep excavation is established.

The main material. The object of construction is a multi–storey residential complex consisting of five sections located on a slope along the contour of Lake Glinka (Fig. 2), at the corner of the streets Filatov and Boulevard Druzhby Narodiv in Kyiv.

The surrounding building is represented by two five-section buildings located at a distance of 20 m from the fence of the pit (Fig. 1).

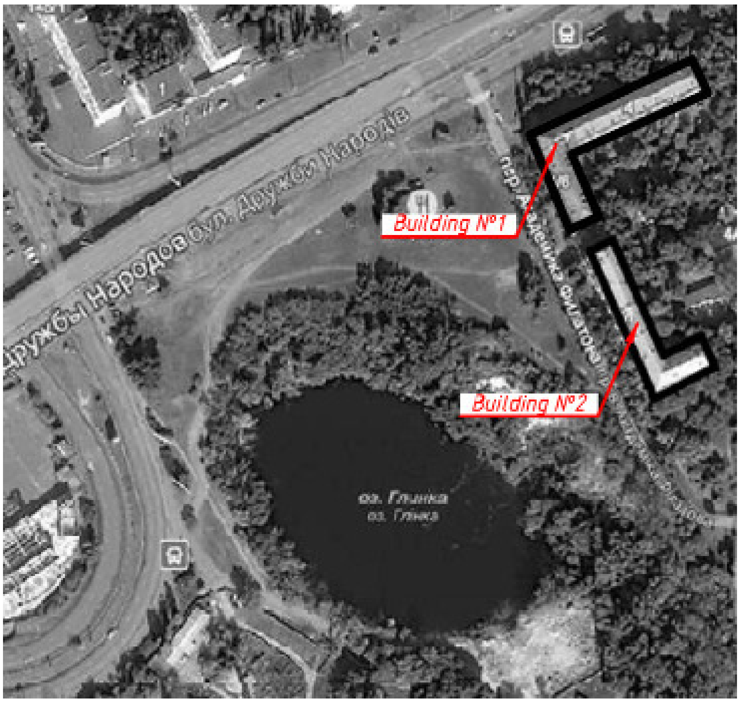

Figure 1 – Scheme of construction site

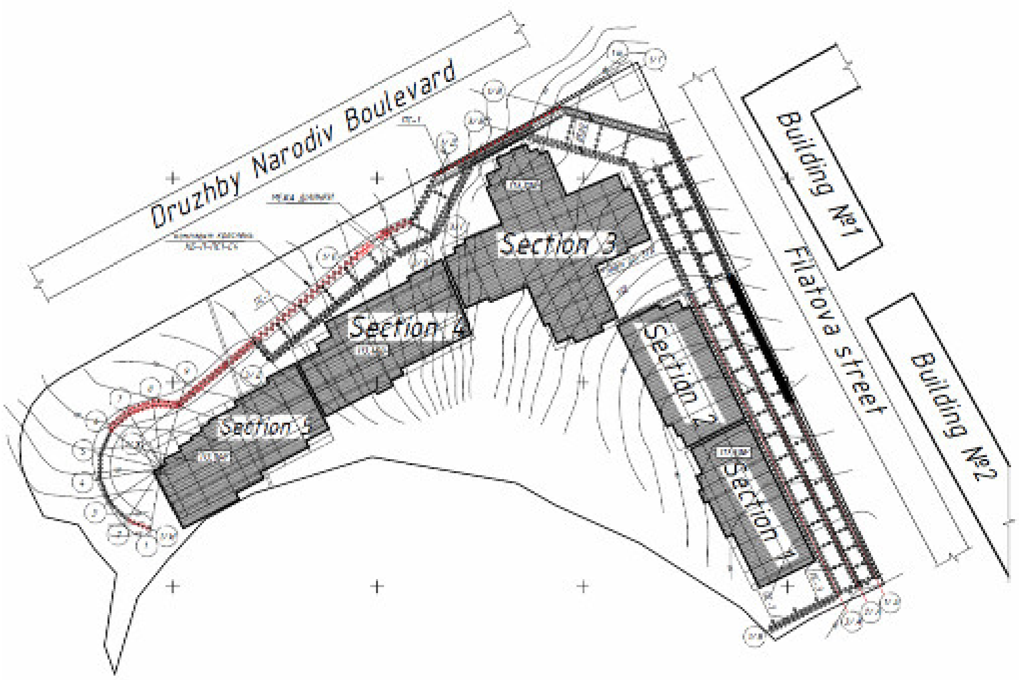

Figure 2 – Location plan of sections, retaining structures and surrounding buildings

Buildings are frameless with brick bearing longitudinal external and internal walls, on foundations of reinforced concrete basements, partially rubble, with basement, with prefabricated reinforced concrete overlap, enclosed on the longitudinal and inner walls. The stability of buildings in the longitudinal and transverse directions is ensured by the joint work of the external and internal walls, as well as prefabricated concrete slabs.

Before the construction was completed, a survey of the technical condition of the buildings was performed. The technical condition, established at the same time, is recognized as satisfactory. According to the report on the survey of buildings, a recommendation was made: when designing a deep excavation near the existing buildings, measures for protecting existing structures should be provided, by arranging the retain structures in the form of walls from bored piles [1]. Methods of these structures arrangement should exclude additional influences on existing objects (vibration, soaking, removal of soil from the basis of existing foundations, etc.).

As a complex of measures for engineering preparation and protection of the territory to ensure the stability of the slope on the site allocated for the construction of the project provides for the installation of a cascade of three levels of retaining walls (in separate sections of two and one row). Piles of all retaining walls are executed with a drilling diameter of 820 mm of variable length from 23 to 28 m with the reinforcement of round spatial reinforcement frames to the full depth. The class of piles concrete is C20 / 25.

The piles of the retaining wall RW–1 are arranged in a chess order with a distance between the piles in the row of 1,8 m and between the rows of piles 0,9 m. This provides for increasing the spatial rigidity of the design of the retaining wall and ensures the passage of groundwater between piles without raising the level of ground water behind the wall and the passage of groundwater between piles without raising the level of ground water behind the wall and the possibility of implementing measures for drainage of groundwater in the area of the retaining wall. In the upper part of the pile of the retaining wall are combined monolithic reinforced concrete grillage height of 1200 mm, which provides a compatible work of piles.

Between piles arranged monolithic reinforced concrete bracing. The maximum mark for the excavation of the pit is provided by the project after the installation of piles and grillages of RW–1 along the Druzhby Narodiv Boulevard is 117,50, and along the Filatov street 127,5.

After the installation of the retaining wall RW–1 on the site along the Filatov street, the supporting wall RW–2 is executed with an absolute mark of 127.5.

Piles of the supporting wall RW–2 are arranged in a chess order with a distance between the piles in the row of 1,8 m and between the rows of piles of 1 m. In addition, with a step of 6...7 m between the piles of retaining walls RW–1 and RW–2 arranged on 3 piles perpendicular to the main piles of retaining walls. In these piles, in the future, a wall–counterfort with a thickness of 500 mm, rests on the wall of the supporting wall RW–1, is arranged.

In the upper part of the pile of the retaining wall RW–2 are combined with reinforced concrete grillage in the thickness of 1200 mm, which provides a compatible work of piles. Between piles of RW–2 is a monolithic reinforced concrete wall–counerfort. Maximum mark for the excavation of the pit is provided by the project after the installation of piles and grillers RW–2 – 118.2.

After the excavation of the soil near the RW–1 and RW–2 along the Filatov street, the supporting wall RW–3 is executed with an absolute mark of 122.8, and along the Druzhby Narodiv Boulevard with a mark of 118.8. The walls of the supporting wall RW–3 are placed in two rows of the distance between the piles in the row is 1,2 m and between the rows of piles 1,2 m. The length of the piles is 23 m.

In addition, with a step of 6...7 m between the piles of retaining walls RW–2 and RW–3 piles arranged perpendicular to the main piles of retaining walls. In these piles, in the future, a wall-counterfort with a thickness of 500 mm is arranged.

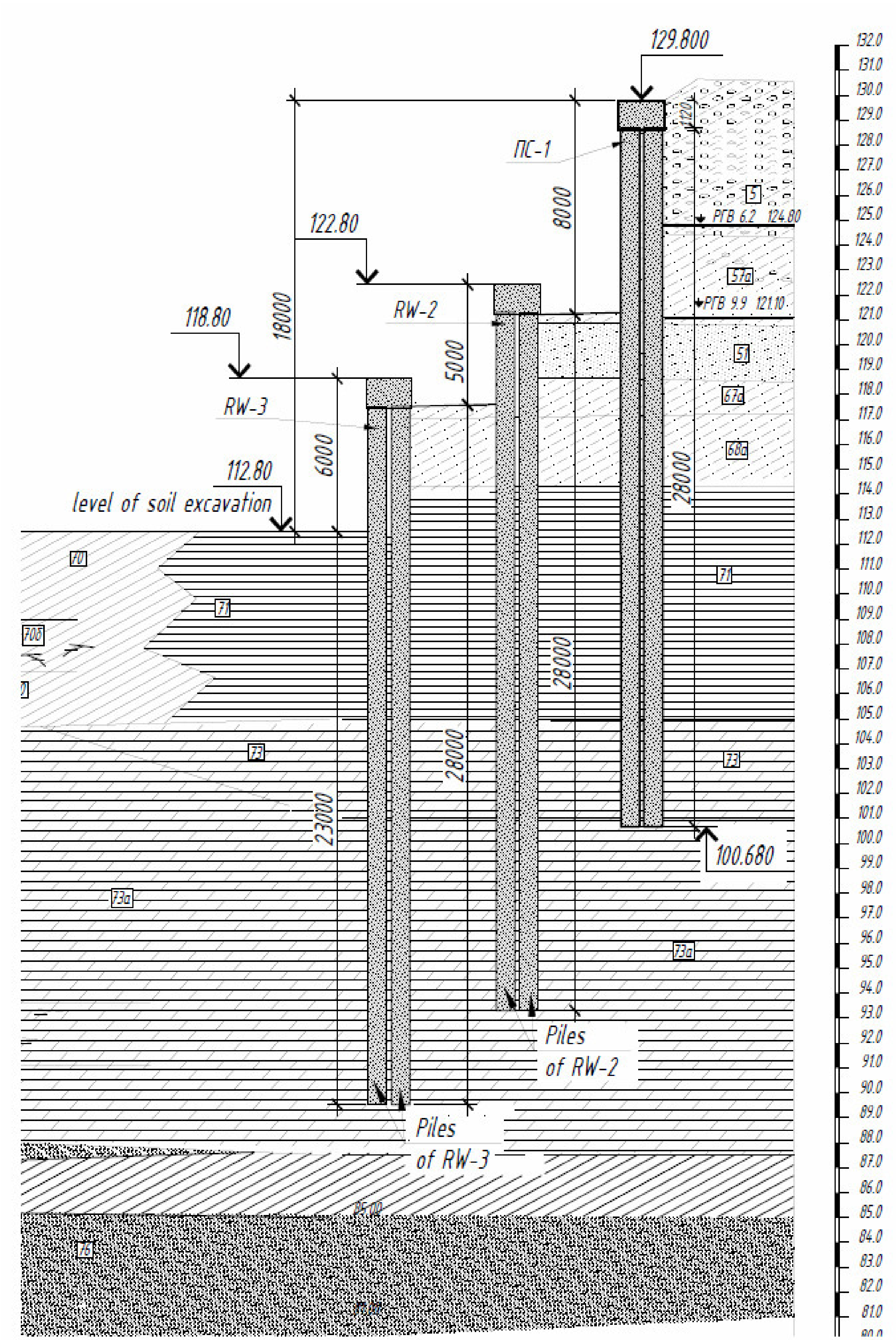

Figure 3 – Characteristic section on retaining walls

In the upper part of the pile of the retaining wall RW–3 are combined monolithic reinforced concrete grillage with a thickness of 1200 mm. Between piles of PS–3 is a monolithic reinforced concrete wall– counterfort.

The maximum mark for the excavation of the pit is provided by the project after the installation of piles and grills of RW–3 along the Filatov street of 117.5, and along the Druzhby Narodiv Boulevard is 113.70. The total depth of the pit makes 18m.

Engineering–geological site conditions are difficult (Fig. 4). According to the topographical plan and the report on engineering geological surveys, the maximum difference between the markings of the surface of the hilly part of the construction site is 12m. In the geological structure, from the surface of the territory prevail sandy soils of sandstone, loams with admixture of construction waste to 35 %, a significant soil–vegetation layer and peat. Below the bulk soil are sandy and sandy clay soils, marl clay and sand. Hydrogeological conditions of the construction site are characterized by the presence of several aquifers.

Within the construction site, there are active landslide processes that arise as a result of the suposonic washout of shallow rock particles when the first aquifer is inclined on the slopes of groundwater.

Before the beginning of work on planning of the territory, transfer of engineering communications and the arrangement of retaining walls, it is planned to organize monitoring of the stability of the slope, hydrological regime and the state of the surrounding building under a special program developed in the framework of scientific and technical support. On supporting elements (walls) of buildings is envisaged to fix geodetic observation marks and to arrange a system of rappers outside the building and its zone of influence.

Also, place the marks along the external retaining wall (RW–1) and on the slope with a step of 10 – 20 m and later on the grilliage of the retaining walls. It is also planned to organize a piezometric borehole system with a step of 20 – 25 m to monitor the level of groundwater.

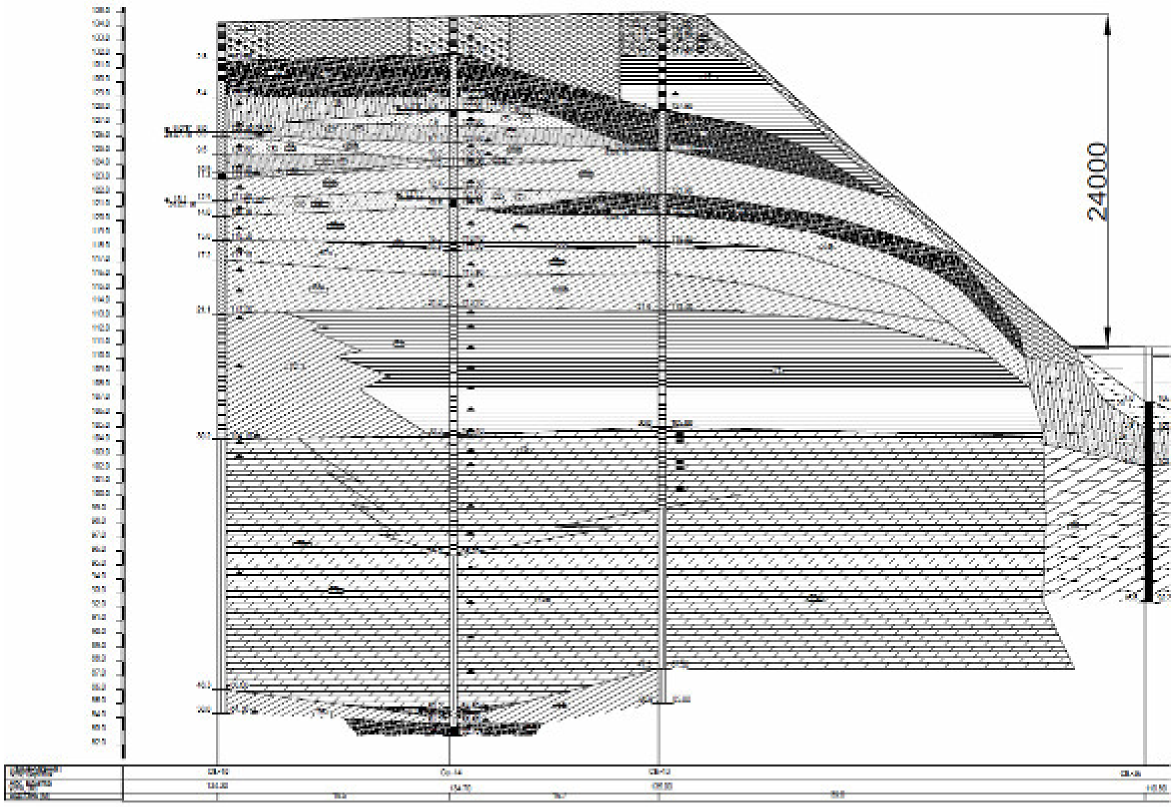

Figure 4 – Engineering-geological section

Table 1 – Physical and mechanical characteristics of soils

№ IGE	Soil description	Density, g/sm^3	Deformation module, MPa	Angle of internal friction, grad	Cohesion intercept, кPa	Filtration coefficient., m/day
14	Soil and vegetation layer	1.9	5	9	10	–
18	Peat of medium degree of decomposition	1.14	6	10	20	0.5
52	Sands of medium size	1.84	45	36	2	6
53	Large sand	1.84	40	37	1	7
55	Solid clay sand	2.0	19	28	17	0.5
56	Plastic clay sand	1.86	20	26	16	0.6
57a	Plastic clay sand	2.02	17	24	13	0.8
67a	Plastic clay sand	2.04	21	25	14	0.2
68a	Plastic clay sand	2.06	19	26	15	0.3
70	Semisolid loam	2.0	19	24	23	0.2
71	Semisolid clay	1.95	30	22	60	0.01
73	Semisolid clay	1.97	31	19	60	0.01

The numerical simulation of the stress-strain state of the protective structures of the deep excavation together with the soil mass was carried out using the finite element method, which enabled to take into calculation the complex properties of the soil, as well as to determine the stresses and displacements in all elements of the system at all stages of soil excavation. At the same time, the soil array was considered as a solid nonlinear medium. The task was solved in a two–dimensional setting. The calculation was carried out in 8 stages.

Fig. 6 shows the design scheme, which includes a ground base with a capacity of 50 m, three tiers of retaining walls (RW) with piles of step 1.2 m, length 28 m in RW–1 and PS–2, and 23 in PS–3, as well as the foundation of the existing buildings with reduced load from the superstructure. Characteristics of the rigidity of the piles of the retaining walls were calculated on the basis of 1m.p. The dimensions of the calculated area are 50×80 m. The lower part of the design circuit, at a distance of 20 m from the sole of the pile of the retaining wall, is limited by a plane that is fixed from the vertical displacements. On the lateral planes, on the basis is imposed ligature, which prevent only normal to the plane of displacement. For the retaining walls, the calculated stiffnesses are taken: $EI = 5.54 \cdot 10^5$ kN·m^2/m, $EA = 1.73$ kN/m.

4 variants of the task were solved:

V1 – retaining walls without surrounding building;

V2 – retaining walls with an existing building at a distance of 20m;

V3 – retaining walls with an existing building at a distance of 10m;

V4 – retaining walls with an existing building at a distance of 5m.

The calculation was made with taking into account the formation of the strained-deformed state of the soil mass during the installation of the retaining wall.

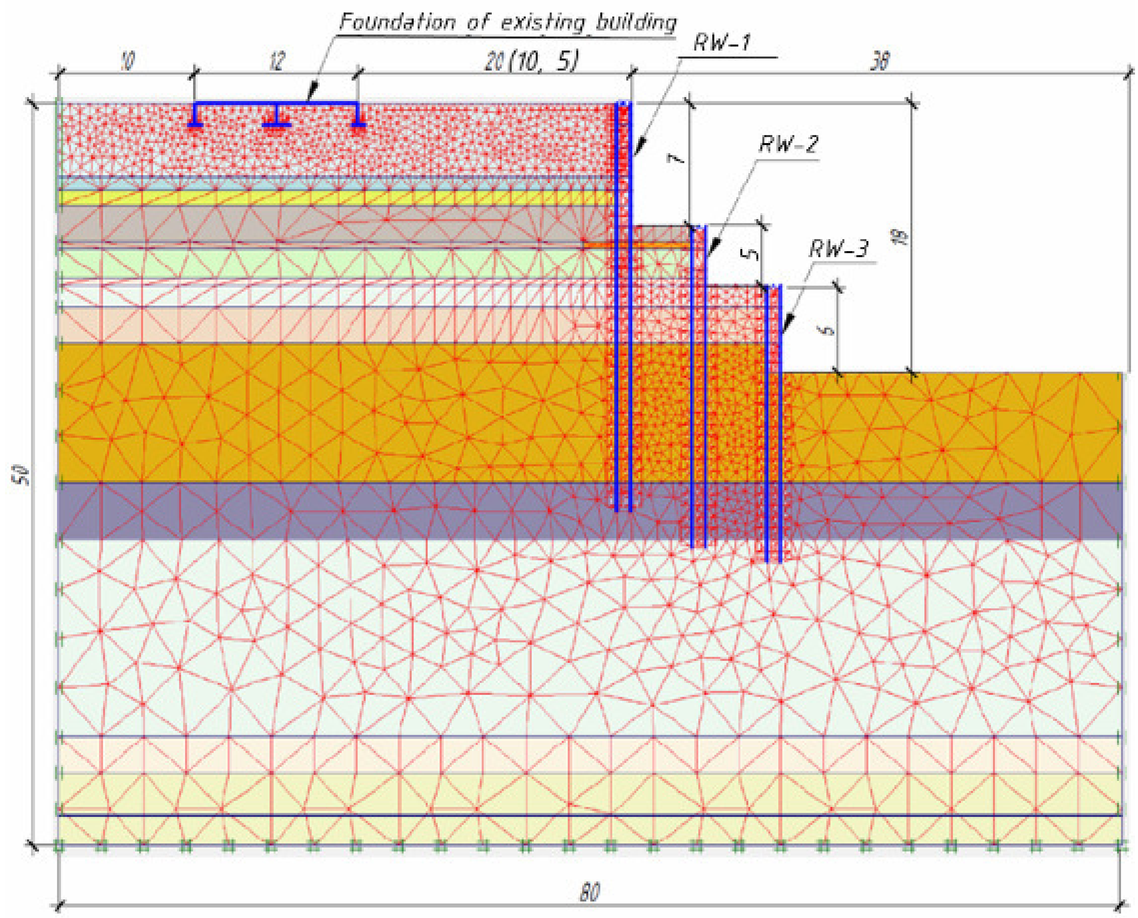

Figure 6 – Analytical scheme

In this case, horizontal displacements of the retaining wall's top, bending moments, and vertical displacement of the existing building's foundation were compared.

Fig. 7 indicates values of the horizontal displacements of the retaining walls top without building, in Fig. 8 – with building at a distance of 20, 10 and 5 m, respectively. Reducing the distance between the building and the retaining wall causes a significant change in horizontal displacement. In Fig. is 9 given a graph that showing changes in the displacement of the retaining wall tiers, depending on the distance to the building. The increase in displacement ranges from 12 to 70 %. The maximum value is observed in variant V4 at a distance of 5 m.

Analysis of the bending moments change indicates that the reduction of distance leads to both quantitative and qualitative change in the diagram of moments. This change is shown in fig. 10, which shows the diagrams of moments in the first pile to building at different variants for the location of the building. The same situation is observed in the piles of the other two tiers of retaining walls. So, with variants V3 and V4, bending moments increase by 2 and 60 % compared with V1 and V2. Insignificant change in the moments in variants V1 and V2 indicates that the existing building at a distance of 20 m from the pit is not in the zone of formation of the slope slip surface. With a decrease in the distance to 10 and 5 m, another picture is observed: the character of the diagram changes with the quantitative values of the moments, as well as their maximum values are formed on other marks. This is due to the fact that the close location of the building near the pit causes its entry into the zone of formation slide surface. This process is clearly observed in Fig. 8.

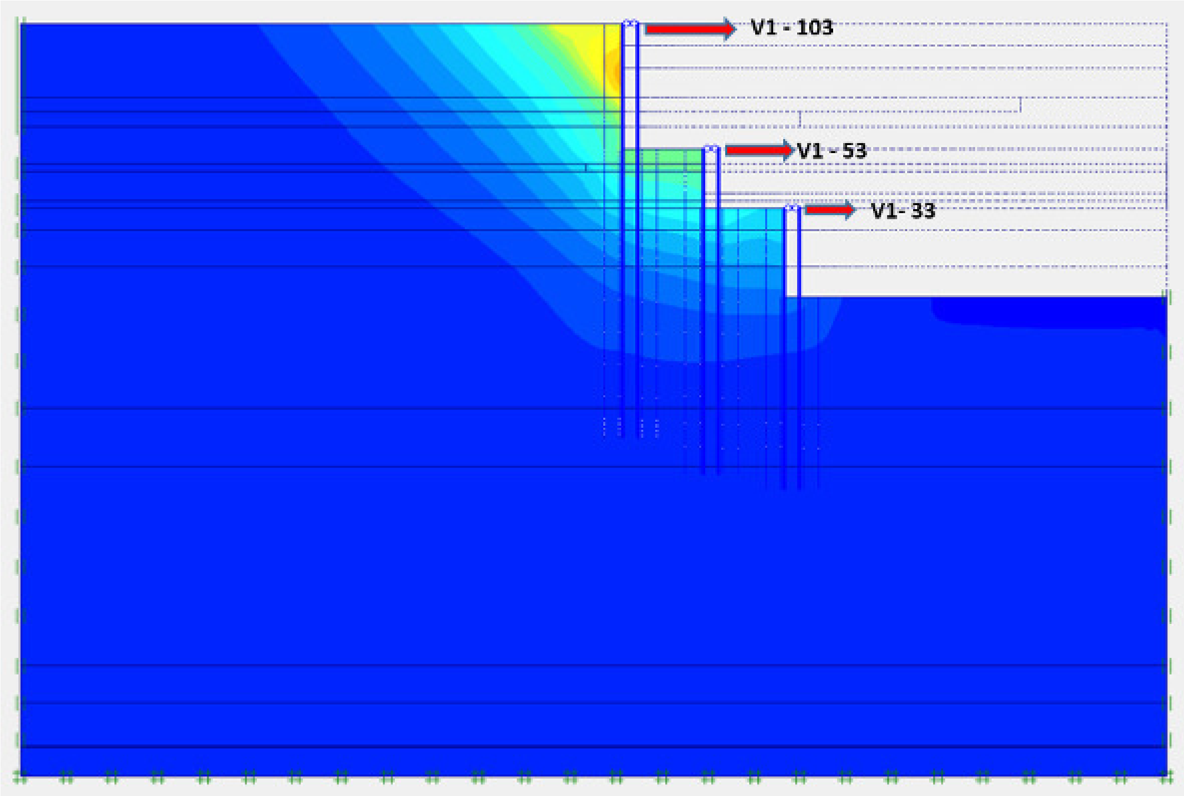

Figure 7 – Horizontal deformations of the retaining structure without building (mm)

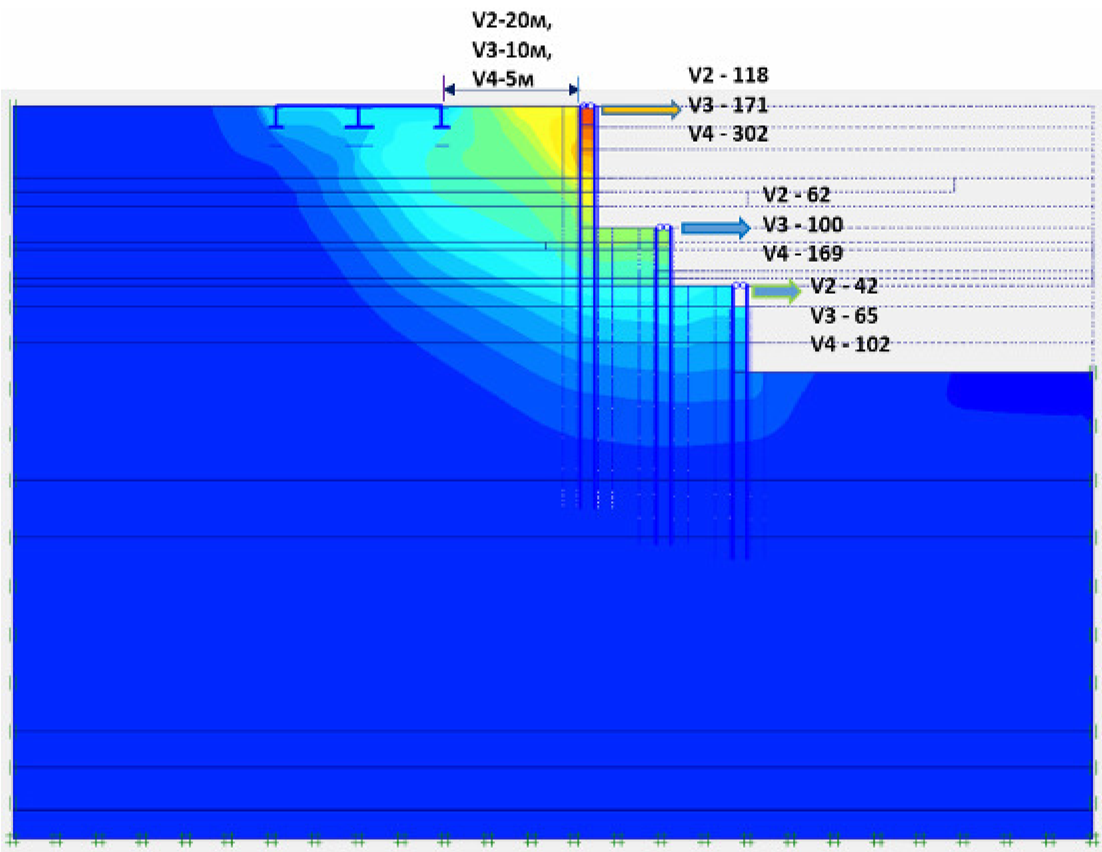

Figure 8 – Horizontal deformations of the retaining structure with building (mm)

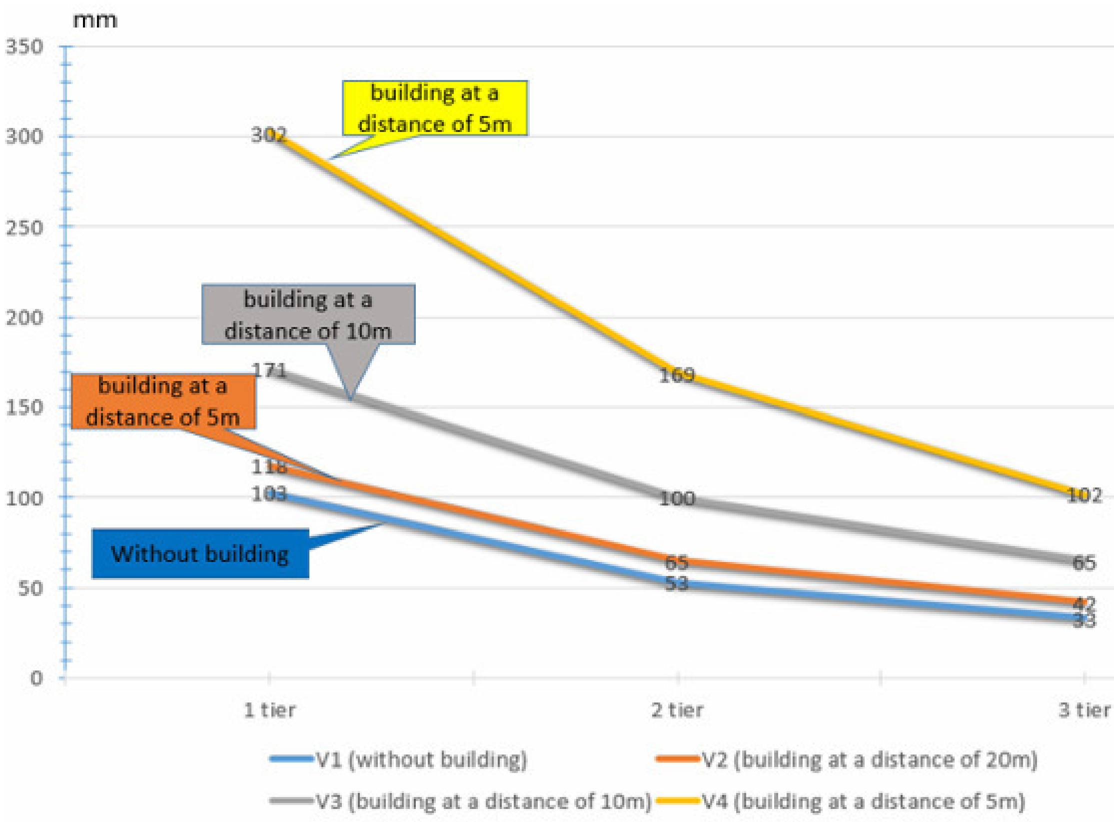

Figure 9 – Dependency graph of the retaining wall tiers displacement from the distance to the building

Analysis of the existing building's stress-strain state has shown that in the foundation structures, with a decrease the distance to the pit, the growth of additional sediments is fixed. There is an increase in vertical deformations by 30 – 40%. This situation requires additional measures to reduce the impact of the installation of the deep excavation.

Conclusions. It was revealed that taking into calculation the influence of existing building upon the installation of the deep excavation significantly changes the character of the soil massif and retaining structures stress-strain state, increasing the displacement of fence structures by 12 – 70%, depending on the distance to the building.

Discovered that bending moments in the fence structure with a decrease in the distance between the pit and existing buildings undergo quantitative and qualitative changes. Thus, the value of bending moments in piles with a decrease of the distance from 20 m to 10 m increases by 26 %, and in the case of a decrease of the distance from 10 to 5 m – by 45 %.

It is shown that reducing the distance between the existing building and deep excavation leads to additional sedimentations in the foundation structures of the existing building. Thus, vertical deformations at the location of the building at a distance of 5 m increase by 35 %, compared with the building at a distance of 20 m, which is explained by the fall of the building into the zone of slip surface slope.

It was revealed that the most rational location of the existing building is when the depth of the pit is equal to the distance between the building and the edge of the retaining wall, as the building does not fall into the zone of the new construction influence.

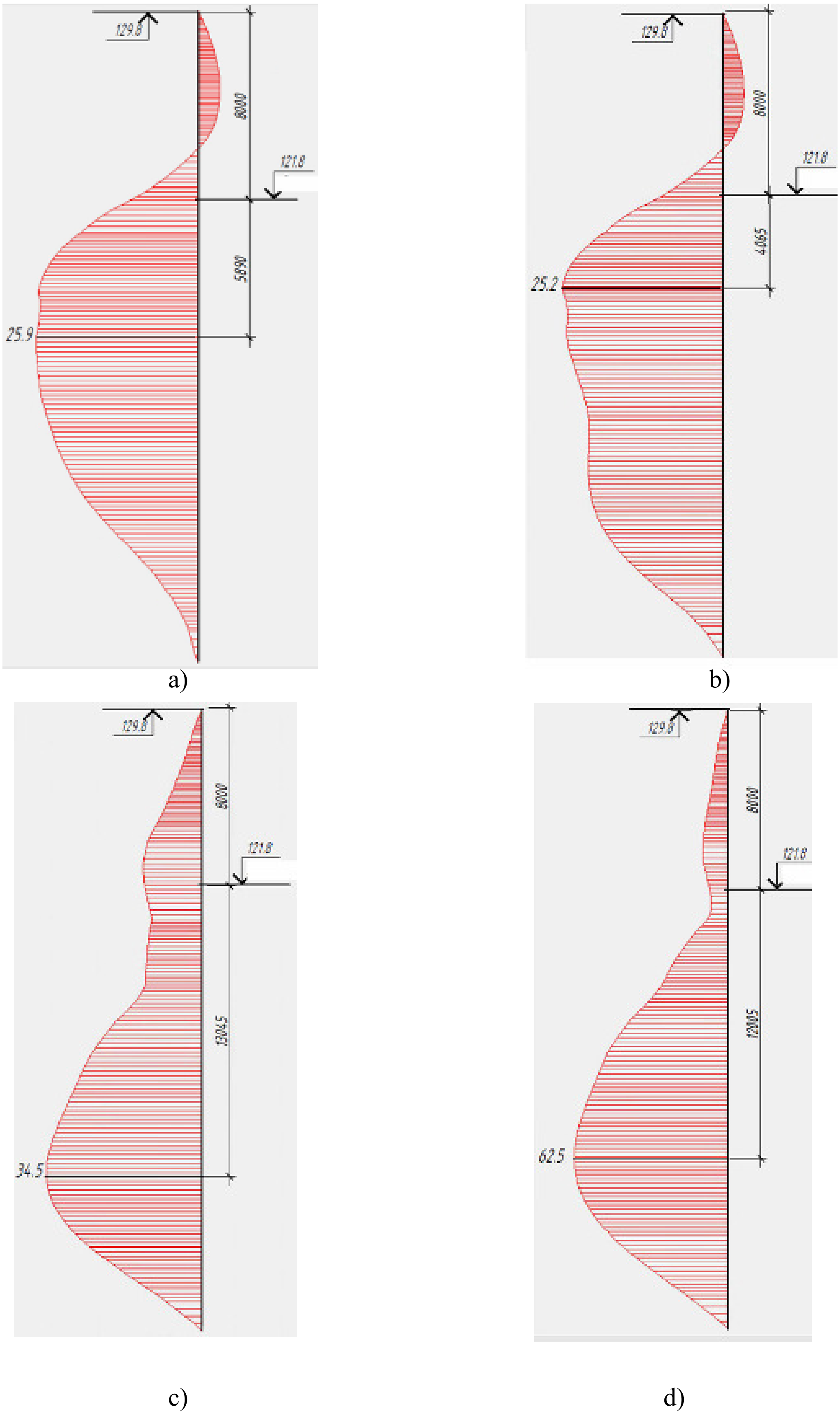

Figure 10 – Curves of bending moments in the first to building pile (t*m):
a) without building; b) building at a distance of 20m; c)building at a distance of 10m; d)building at a distance of 5m

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SINGLE PILES SETTLEMENT UNDER THE ACTION OF NEGATIVE FRICTION FORCES

Nowadays, the use of pile-plate foundations has received extensive development, which takes into account the soil inclusion in the work under its plate part. We proposed a new design of a plate-pile foundation and an engineering method for determining the main parameters of such a foundation, where one of the parameters is the piles settlement under the action of the negative friction forces P_n. As far as is known, field testing of production piles using pressing-in loads is the most reliable method for determining their actual load-bearing capacity. We carried out full-scale tests of a multi-section pile with the loading at depth, on the basis of which a comparative analysis of the settlement of single piles from the action of the negative friction forces, obtained with the help of adapted and existing methods and experimental data

***Keywords:** ground base, pile-plate foundation, clearance, single pile, settlement, method*

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ОСІДАННЯ ОДИНОЧНИХ ПАЛЬ ВІД ДІЇ ДОВАНТАЖУВАЛЬНИХ СИЛ ТЕРТЯ

На сьогоднішній день широкого розвитку отримало застосування пальово-плитних фундаментів, де враховується включення в роботу ґрунту під його плитною частиною. Нами було запропоновано нову конструкцію плитно-пальового фундаменту та інженерну методику визначення основних параметрів такого фундаменту, де одним з параметрів є осідання паль від дії довантажу вальних сил тертя P_n. Як відомо, польові випробування паль на вдавлюючі навантаження є найбільш надійним способом визначення їх фактичної несучої здатності. Нами було проведено натурні випробування багатосекційної палі з прикладенням навантаження на глибині на основі чого проведено порівняльний аналіз осідань одиночних паль від дії довантажувальних сил тертя, отриманих за допомогою адаптованих існуючих методик та експериментальних даних.

***Ключові слова:** ґрунтова основа, плитно-пальовий фундамент, зазор, одиночна паля, осідання, методика*

Introduction. In the construction of multi-storey and high-rise buildings, where there are significant loads on a subsoil base consisting of not rocky soils, the so-called combined pile-plate foundation has being recently used to reduce the absolute and relative settlements, taking into account the inclusion of soil under its plate part [1].

Review of the latest sources of research and publications. For a real inclusion of the base under the plate in the monograph [2], a new construction of the plate-pile foundation was proposed (Figure 1), consisting of a plate 1 and piles 2 with diameter *d*, with a clearance 3 with height *Δ* (for example, in a concrete cap 4 below the plate), and the distance between the pile axes is *nd*. At the same time, the new design lacks the drawbacks of existing constructive solutions and methods for constructing such plate-pile foundations [3].

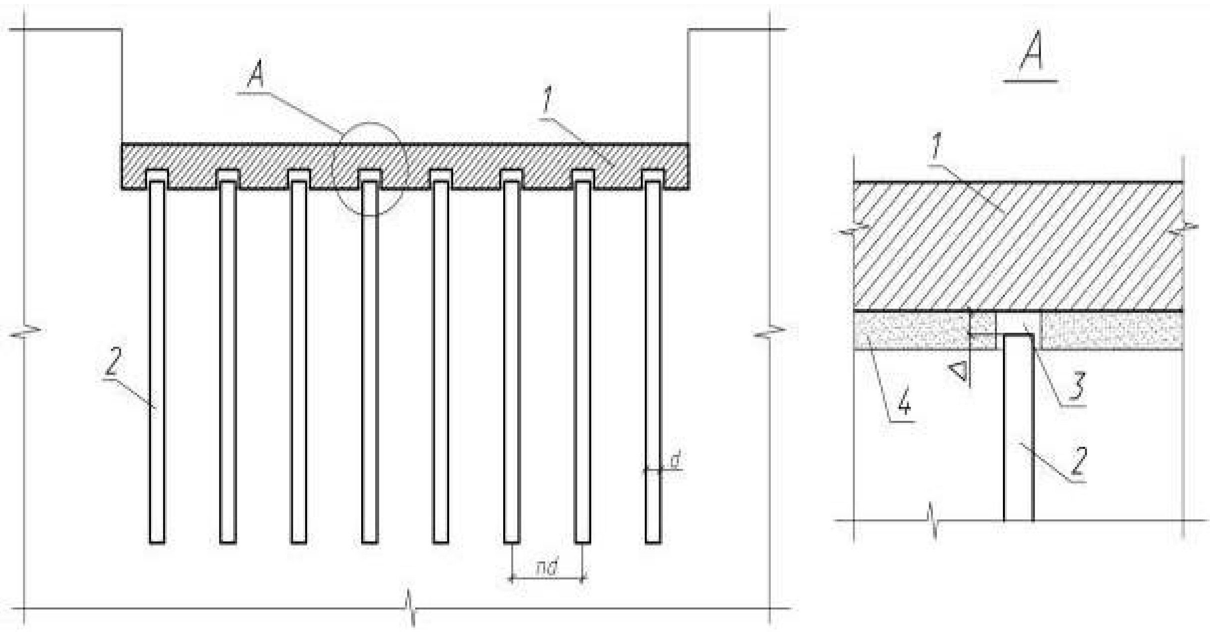

Figure 1 – Plate-pile foundation

The developed engineering method for determining the main parameters of such a plate-pile foundation based on the maximum allowable structure settlements [2] involves determining the height of the clearance *Δ* between piles and the plate part, which is equal to the value of additional pile settlement caused by the development of negative friction forces along their lateral surface within compressible thickness under the plate.

The selection of previously unresolved parts of the general problem to which the article is devoted. In the existing normative documents and scientific and technical literature, there are no methods for determining the settlements of piles from the action of negative friction forces P_n.

Formulation of the problem. The purpose of this paper is to perform a comparative analysis of the settlements of single piles from the action of negative friction forces obtained from various adapted methods with experimental data.

Main material and results. To determine the settlement of the pile $S_{pile}^{P_n}$ from the action of the negative friction force P_n, when the plate of the grillage works first, we present the calculation schemes in Figure 2, where the transition from the design scheme in Figure 2, a, but to the calculated scheme in Fig. 2, b, i.e. to the scheme of applying the load P_n to the pile at a certain depth. The lower part of the pile is located in a two-layer base with the corresponding deformation characteristics: E_1, v_1 and E_2, v_2.

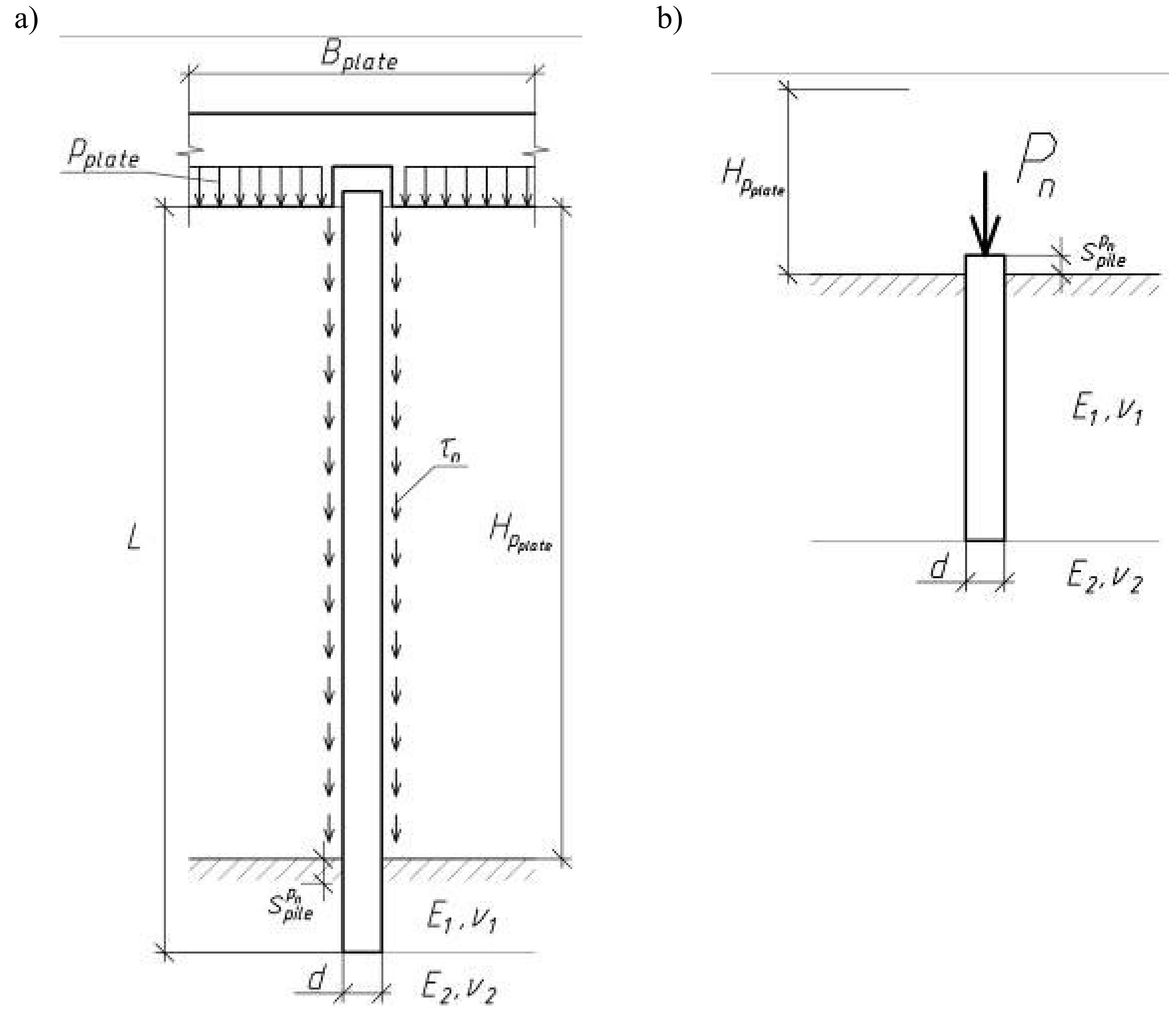

Figure 2 – Calculation schemes:
a) for the formation of friction forces τ_n on the lateral surface of the pile;
b) for determining the settlement of the pile from the action of negative friction forces

As far as is known, field testing of production piles using pressing-in loads is the most reliable method for determining their actual load-bearing capacity. In this field, the regulatory and the most common method is soil testing using piles, where a load is applied to the pile head by means of standard hydraulic equipment using counterweights or anchoring systems of various designs. While testing soils using piles, the stress-strain condition (SSC) of the «soil foundation – pile» system is being investigated using both conventional manometers and deflectometers, and modern transducers and equipment, whereby the load-bearing capacity is determined by means of different methods.

For the calculation scheme in Fig. 2, b we obtained the experimental values of the sediment for different loading levels by the force Pn when testing the soils by the full scale two-section pile by the «ONLY-DOWN» method [4, 5], namely the settlements of the lower part of the pile from the applied load at the depth (Figure 3). «ONLY-DOWN» method was proposed for testing production multi-section piles, which makes it possible to improve the reliability of the testing process and the accuracy of determining the value of the pile load-bearing capacity for pressing-in loads.

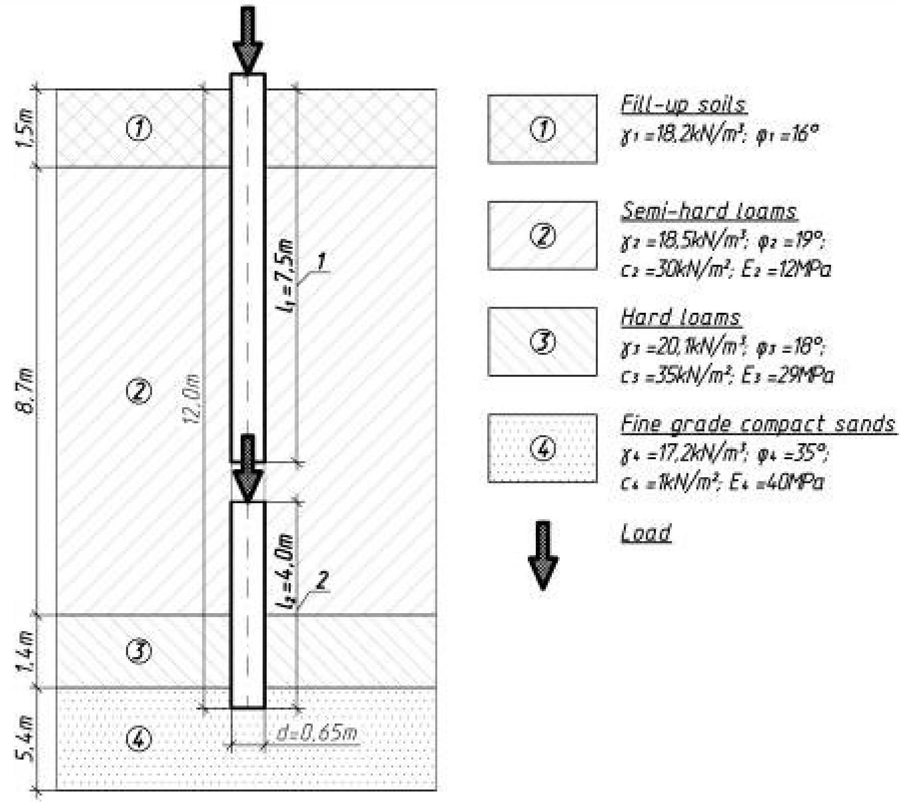

Figure 3 – Scheme of soil testing with a two-section natural pile by the «ONLY-DOWN» method:
1 – upper pile section; 2 – lower pile section

For the theoretical determination of the settlements of the lower section of the pile, the following adapted methods were considered:

– method for determining the settlement of a single pile in accordance with DBN [6];

– method for determining the settlement of the pile according to the design scheme of the conditional foundation in accordance with the DBN [6];

– method for calculating a single pile in a two-layer elastic environment, proposed by V.G. Fedorovskii [7, 8];

– method for calculating the settlement of a bored pile in a bilinear formulation, proposed by B.V. Bakholdin [10, 11].

The main factor of application of these methods is the correct determination of the proportionality limit or the boundary of the linear area of the dependence of the settlement on the load. In the case under consideration, the P_e load limiting the linear section of the pile settlement was determined in accordance with the recommendations of the DBN [6], namely:

$$P_e = 0{,}5P_u = 550{,}0 \text{ kN}, \tag{1}$$

where P_u – the value of the ultimate pile resistance based on the results of full-scale tests, equal to 1100,0 kN (Figure 4).

This approach was applied to all methods, except for the procedure for determining the settlement of the pile according to the design scheme of the conditional foundation. In this case, the conditional limit of proportionality was limited by the condition of point E.5 [8], when the average pressure under the base of the conditional foundation is less than or equal to the stress from the self-weight of the ground at the level of the base of the conditional foundation: $p \le \sigma_{zg,0}$.

The results of calculations using the adapted methods are shown in Figure 4 in the form of theoretical dependences of the settlement of the lower section of the pile on the load, and also the experimental dependence on the test results is presented.

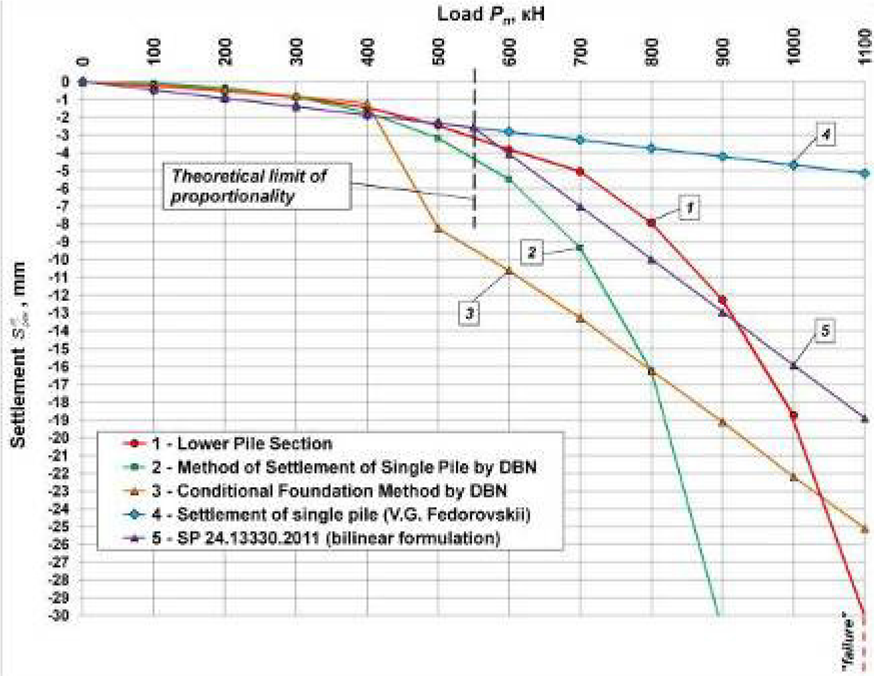

Figure 4 – Graphs of the theoretical and experimental dependencies of the pile settlements $S_{pile}^{P_n}$ on the negative friction force P_n

From the graphs obtained in Figure 4 it can be seen that all the adapted methods considered allow determining the settlement of the pile up to the limit of proportionality, i.e. up to the load limiting the linear dependence of the pile settlement on the load. The method for determining the single pile sediment, proposed in DBN [6], makes it possible to reveal the nonlinear nature of pile deformation after the proportionality limit, but overstates the real values. In this case, the most accurate method was used to calculate the settlement of a single pile from the action of additional load forces in a bilinear setting, proposed by B.V. Bakholdin [10, 11], which shows an inaccuracy of no more than 15,0% at any level of loading.

Conclusions. On the basis of carried out experimental and theoretical studies, the following conclusions can be drawn:

1. A comparative analysis of the settlement of single piles from the action of the negative friction forces obtained with the help of existing adapted calculation methods and on the basis of the results of testing the soils with a two-section in situ pile.

2. At this stage of the study, it is proposed to determine the settlement of the piles from the additional frictional forces using the B.V. Bakholdin method, which is represented in the Russian codes SP [11].

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INVESTIGATION OF GROUND AND FOUNDATION CONDITION INFLUENCE ON THE SPASO-PREOBROZHENSKAYA CHURCH CONSTRUCTIONS IN POLOTSK

In the article the problems of the XII century architecture monument preservation – Spaso-Preobrazhenskaya church of Spaso-Efrosinievskiy convent in Polotsk are considered. A brief reference is given on the technical condition and design features of the facility. Various parameters research results allows to estimating bearing constructions technical condition. The issues of bearing structure geodetic, visual, instrumental and electronic remote monitoring complex implementation and organization are considered. The issues of innovative technology introduction for this research area such as ground-based three-dimensional laser scanning are covered. There was made a conclusion church foundation conditions and there are given recommendations as for their strengthening.

Keywords: *foundation, church, architectural monument, laser scanning, monitoring of building structures.*

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ВИВЧЕННЯ ВПЛИВУ ҐРУНТУ ТА ФУНДАМЕНТУ НА БУДІВЛЮ СПАСО-ПРЕОБРАЖЕНСЬКОЇ ЦЕРКВИ В ПОЛОЦЬКУ

Розглянуто проблеми збереження пам'ятки архітектури XII ст. – Спасо-Преображенської церкви Спасо-Єфросиніївського жіночого православного монастиря в Полоцьку. Наведено коротку довідку про технічний стан і конструктивні особливості об'єкта, результати досліджень різних параметрів, які дозволяють оцінити технічний стан несучих конструкцій будівлі. Розглянуто питання впровадження та організації комплексу геодезичного, візуального, інструментального й електронного дистанційного моніторингу несучих конструкцій. Охоплено питання впрвадження інноваційних технологій у цій сфері досліджень, таких як наземне тривимірне лазерне сканування. Зроблено висновок про стан основ церкви та наведено рекомендації щодо укріплення.

Ключові слова: *фундамент, церква, пам'ятка архітектури, лазерне сканування, моніторинг будівельних конструкцій.*

Introduction. Since the end of the XX century, recovery and restoration work of buildings and structures having historical and cultural value are held. Ancient buildings and structures church appointments significantly differ from other structures in view of their purpose specific nature, actual operating conditions, long life time, building materials features. Deformations and damage of old buildings and structures is bound with natural processes, changes in soil conditions, water balance of the territory, etc. Therefore, the assessment of buildings technical condition is a complex task and requires specific research methods, in particular, the study of the system «structure – foundation».

The latest sources of research and publications analysis. When carrying out work to survey and study changes in the technical condition of load-bearing structures of architectural monuments, it is necessary to comply with the requirements of the current legislation [13 – 16]. The restoration of religious historic buildings and structures is always difficult due to lack of information. This requires studying the experience of domestic colleagues similar studies in foundations surveys and engineering-geological surveys [8, 9]; carrying out of measurement works of architectural heritage objects [11]; monitoring and development of measures to strengthen foundations [10, 12]. Also very valuable in the study of changes in the technical state of structures is the foreign experience [1 – 7].

Reconstruction of cultural heritage each object is a research work from the beginning to its completion.

Allocation of previously unresolved parts of a common problem. The determination of the building, structures and historical monuments actual technical condition was either not carried out or was not carried out completely. It is due to the fact that innovative technical tools were not available for engineers-restorers. Information on the dates and times of changes in the technical condition caused by changes in soil conditions, water balance of the territory, increased aggressiveness of natural factors and the reorganization of the temple, was not preserved, or was not sought in the course of special works. Reconstruction of full-fledged technical documentation of historical and cultural heritage objects with complex geometric parameters is impossible with the use of simple linear measuring instruments.

Formulation of the problem. The introduction of modern innovative software systems and equipment allows performing research at a high technical level. The development of a set of measures to investigate the stress-strain state of the church structures should be oriented towards comprehensive study of the factors affecting it, as well as preserving the old decorative layers that bear high cultural value. The analysis of the research results carried out on the architectural monument of the 12th century – the building of the Spaso-Preobrazhenskaya church of Spaso-Efrosinievskiy convent in the town Polotsk allows developing complex solutions for reinforcing or restoring existing structures and ensuring an acceptable level of facility safe operation.

Technical condition of the object. The Spaso-Preobrazhenskaya church is the oldest historical and cultural monument on the territory of Belarus that has survived till the present day. The history of the Spaso-Preobrazhenskaya church of the of Spaso-Efrosinievskiy convent in the town of Polotsk begins in the 12th century and is closely connected with the activities of the Reverend Efrosinia Polotskaya.

The temple is a cross-domed six-pillar structure with an adjacent porch and a semi-circular apse in the plan (Figure 1). The church has small dimensions: length – 14.4 m (with an apse – 18.2 m), width – 9.8 m, height to the roof ridge – 14.5 m, to the base of the cross on the drum – 22.7 m. The structural basis of the building is a powerful wall with pylons united in an integral system, columns-pillars and cross vaults. All the supporting structures of the XII century are made in the form of masonry on a lime mortar. On the internal plaster layer of the walls, as well as on the plaster surfaces of pillars and arches, mural painting is applied.

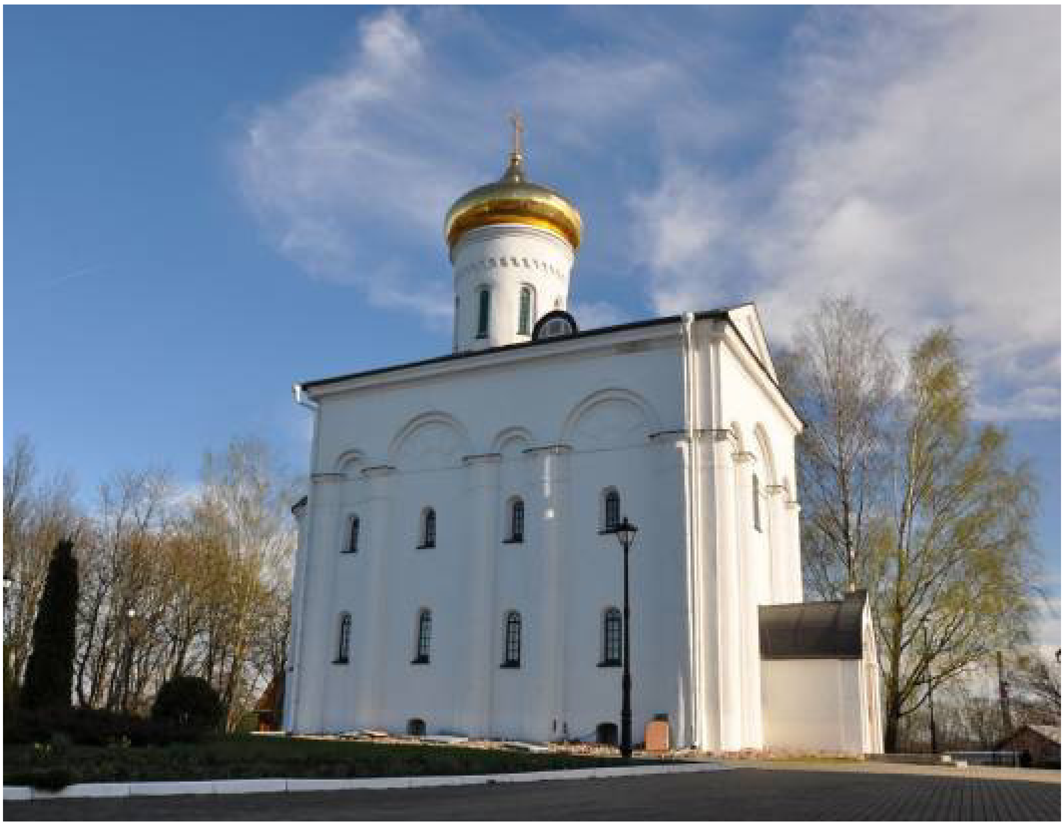

Figure 1 – General view of the Spaso-Preobrazhenskaya church in Polotsk

The appearance of the church has had significant changes in the original form. In the eighteenth century, the first restructuring was carried out by the Jesuits, who from 1580 were its owners. The facades and the roof were changed. The cascade of kokoshniks was hidden by a parapet and pitched roof. For ritual burial grounds, a crypt was built-a basement room with a width of 160 ... 180 cm in the form of a cross covered with brick vaults. At the device of a crypt, probably, foundations have been damaged, since. In the side walls, the openings through which the entrance was made were punched. Also, the level of the floor was raised by approximately 30 ... 40cm, the bottom of the walls of the crypt was arranged approximately at the bottom of the foundations of the main building, and the maximum height of the crypt is 150 ... 170cm. Currently, these three apertures (entrance-laz) are in closed pit covers in the walls of the southern and northern facades.

After the transfer of Polotsk to the Russian Empire, the temple was returned to the Orthodox. Repair work was carried out with a change in the appearance of the temple. The dome over the drum became bulbous. The works on the construction of the bell tower were stopped because of the cracks that appeared in the walls and vaults. The clutch of the bell tower was not dismantled. Work was done to strengthen the walls – the archa-shells were laid, steel straps were installed in the level of the parapet.

The planned restoration of the fresco painting began in the early 90s of the last century and has not been completed yet (Figure 2).

The material for the foundation of the church is rubble stone, laid with incomplete filling of seams. In archaeological excavations there are foundations with the laying of stones «dry» and the presence of residues (decay) of oak stands (Figure 3). The total height of the basement foundation is on the average 90 cm, and the height of the walls masonry is at a certain level below the level of the existing planning mark – about 100 cm. In the masonry wall solution contains a large amount of cement.

Figure 2 – General view of the church structure.
On the walls, arches, pillars, there is a fresco painting of the 12th century

Figure 3 – Results of the structure opening to determine the technical state of foundations under the pillars

Crypta (an underground arched structure for the burial of Catholic priests) was arranged during the period of Jesuits possession by the temple. Partly under the walls, the crypts were made of quarry stone on a limestone mortar with a base height of 300 mm, a width of 320 mm, partly no foundations. The walls of the crypt are made of thick “brick”, the vaults are «half-brick» on the lime mortar.

Within the walls of the church, cracks with different opening widths (from 0.05 to 75 mm) were identified (Figure 4). Cracks have sedimentary character. At the junction of the cell wall with the outer wall there is no bandage between the rows of wall masonry, which allows exchanging between the outer walls and inner pillars. In the pillars longitudinal and transverse cracks are revealed. The pillars have general tendency towards the exterior walls of the building. Longitudinal and inclined cracks are revealed in the laying of the kokoshniks.

According to the totality of moral and physical deterioration signs, the technical condition of the building constructions is generally characterized as unsatisfactory, requiring measures for repair and reinforcement. Physical wear of structures is 40...80%.

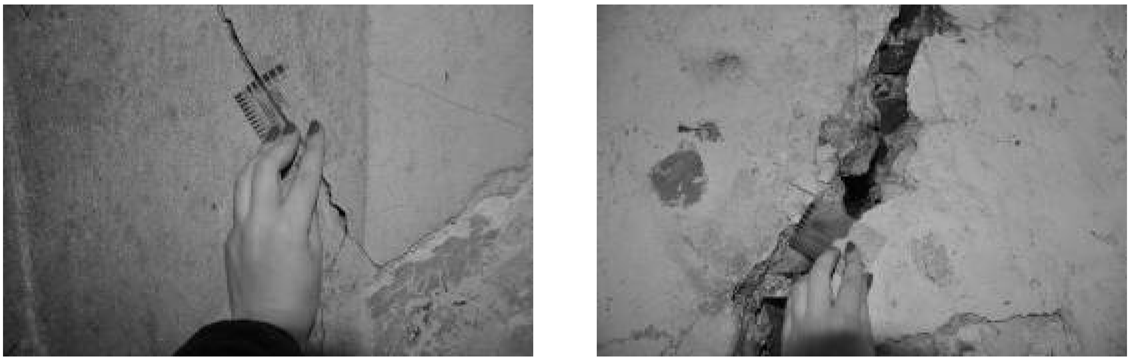

Figure 4 – Measurement of crack opening width in church structures

Monitoring of church buildings bearing structures stress-strain state. The results obtained during the survey showed the need to carry out work to strengthen the church designs and foundations. At the same time, in order to assess the actual stress-strain state of the structures more accurately and to develop measures to strengthen them, additional studies are needed (determining the properties of the foundation soils under the foundations soles, characteristics, considering the nonlinear stage of work and the actual pressures under the soles, clarifying the depth and width soles). In this situation, the most expedient solution was to monitor the stress-strain state of the building and the dynamics of crack opening in the above-ground structures.

Researches were done to monitor the stress-strain state of construction works to carry out joint projects of the research and production unitary enterprise «Stroyrekonstruktsiya» and the «Bridges and Tunnels» Department of the Belarusian National Technical University.

Monitoring provides for continuous monitoring in full-scale conditions for deformations and forces in the supporting building structures of the church and verification of their compliance with design values ensuring accident-free operation of structures, and includes: geodetic; visual; instrumental; electronic remote monitoring and scientific support of monitoring activities [17].

For geodetic monitoring inside and outside the temple geodetic marks were installed, with the help of which observations were made of the draft and deformations of the temple structures. In addition, a laser scan was performed, which will install deflections and rolls of load-bearing structures for a more complete analysis of the processes occurred during the time of existence.

The obtained scan results were subsequently transferred to correct the calculation model, performed in accordance with the previously existing approximate dimensional drawings. It should be noted that the application of the above technology allows making the amount of work that various specialists have tried to do for many decades (Figure 5).

During the visual monitoring, all the defects and damages were photographed, the initial geometric dimensions of the defects were recorded, maps of the cracks and the route were compiled for their control [18]. The deformations and damage to the building were caused by natural processes, modern technologies related to physical and material resources revealed during the control process, significantly accelerating the aging of building materials.

When instrumental monitoring, stationary beacons were installed to control the width of the crack opening by «Concretes and building materials» of the Belarusian National Technical University. The study of all the computational information was incorporated into the design model of the structures. In addition, a qualitative chemical analysis of a brick sample and masonry mortar was performed to determine the presence of soluble salts of carbonates in the masonry of the temple.

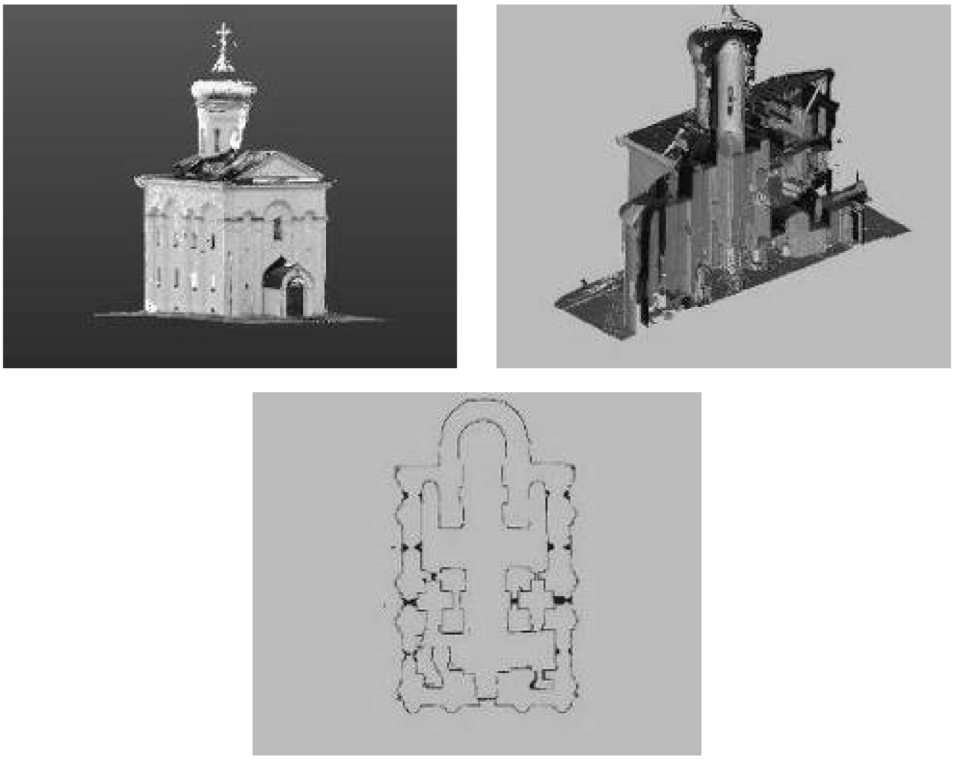

Figure 5 – Some results of a three-dimensional ground-based laser scanning of a church building. General view of the points cloud, a cut, a plan in the level of the choir

In the church it was controlled the temperature-humidity regime [19], in particular, using thermal imaging. As a result of the study, thermal bridges with obvious heat losses were identified; the actual irregularities in the distribution of the field temperature on the wall surface were revealed, indicating a degradation of the thermal protection characteristics of the finishing materials; places with high humidity were identified.

In the course of electronic monitoring, the state of structures was determined, the appearance and rise of groundwater level in the crypt and its fixation in quantitative terms, the angles of of the building parapets walls, the width of crack opening at characteristic points, and the temperature change in the church in height and area. For convenient analysis, the data was provided in the form of graphs (Figure 6).

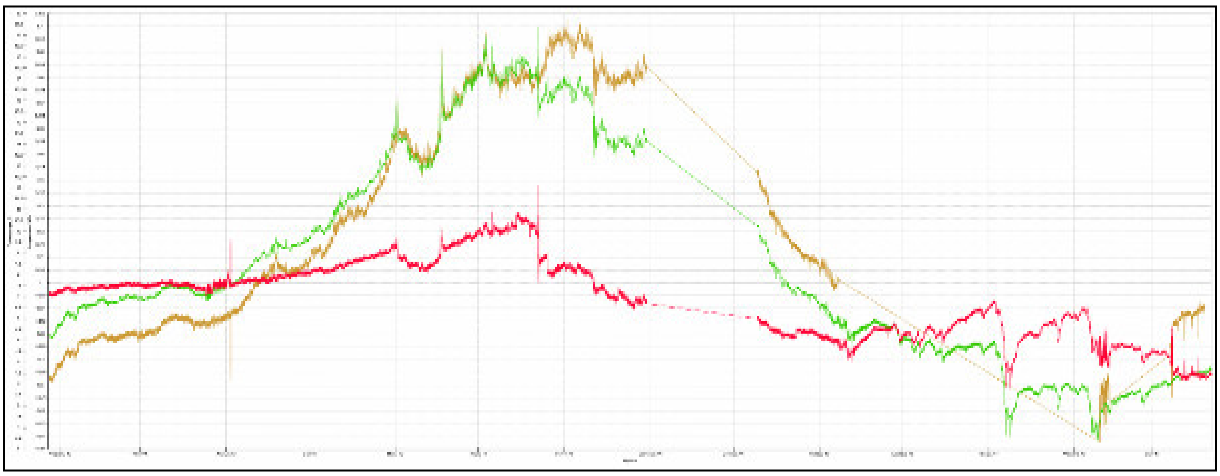

Figure 6 – Graph of changes in temperature and stresses occurring in pillar structures from December 2015 to March 2017

When analyzing the data of load-carrying structures electronic monitoring, it is established that not all structural elements work in the same way with temperature changes and the effect of temporary loads. The work of two pillars and the southern wall does not correspond to elastic work. The change in their stress-strain state cannot be simulated and, in some cases, works separately from the basic structures of the church. This is due to the base sediment and the insufficient bearing capacity of the foundations. The work of the crypts pillars is complicated, toward which the foundations of the pillars are shifted. It is necessary to take urgent measures to strengthen the foundation and foundations under the pillars. The crypt walls do not work in the elastic stage, deforming the stent inside the crypt. There is no unification of the crypt walls with the arch. The ability of walls to perceive Effective loads from ground pressure and loads transmitted from the pillar can be lost if the necessary measures are not fully implemented.

Engineering and geological surveys using static and dynamic sounding. The work was carried out by the employees of Vitgeostroy LLC and OOO Ecotechcontrol LLC. During engineering and geological surveys and at the same time there were carried out archaeological research, the main purpose of which is the most complete study of the cultural layer for the reconstruction of history, the architectural appearance of the temple surrounding their territory, and the collection of documentary data to justify restoration solutions.

When performing the refined, 7 pits were made (Figure 7).

Figure 7 – General view of the pit on the south side of the temple

Geomorphologically, the area belongs to the lake-glacial plain formed during the period of the otzyzer glacier retreat. The relief is in a state of stable equilibrium. Modern active physico-geological processes and phenomena are not observed.

The climate of the region is transitional from marine to continental, characterized by warm, humid winters and a cool rainy summer. According to SNiP 2.01.01-82, the survey area refers to the II «B» climatic zone.

The hydrogeological conditions of the site are characterized by the presence of groundwater. Groundwater is represented by perch, formed on the roof of lake-lacustrine loams in bulk soil.

Ground substrates under the foundations of the church show heterogeneity in their properties. There are 2 types of soil in underground structures: in the southeastern part of the temple, under the foundations are tug-loamy loams and (type I), in the northwestern part - soft-clastic loams (type II)

The values of the strain modulus for Type I soils are 32-35 MPa, for type II soils it ranges from 17 to 24 MPa. The amount of adhesion is 45 – 66 kPa for type I soils and 49 – 58 kPa for type II soils. The value of the angle of internal friction for soils of the type I type varies from 12 ° to 28 °, for soils of the base of type II – 13 – 14 °.

The values of the strength and deformation properties obtained in the framework of the present studies on the basis of the results of static and dynamic sounding for soils lying under the foundations or near the foundations of the temple are generally higher than the values of similar indicators found earlier for soils not compacted under the influence of the structure. This conclusion is supported by static and dynamic sounding data. For loam type II loam, the difference in the modulus of deformation of soils compacted by the building (E = 24 & 25 MPa) from uncompacted (E = 18 MPa) is well traced. For soils of type I loam this difference is much less: under the foundations E = 34 MPa, outside the foundations E = 32 MPa.

Engineering and geological survey results revealed that under the foundations of the Spaso-Preobrazhenskaya church there are no wooden elements to strengthen the soil base. The main influence on the hydrogeological conditions of soils occurrence of the church foundation is closely related to atmospheric precipitation. According to the data of electronic piezometers and stationary observations of the groundwater level, a regularity in the increase of groundwater level in observation wells was revealed, depending on the amount of precipitated precipitation. Underflooding of the temple basement part causes regular humidification of wall structures due to capillary water uplift, deterioration of strength and deformation properties of materials of structural elements, formation of high temperatures on their surface and traces of biopores.

Based on the analysis of the study results, changes in the technical condition of the church supporting structures are recommended to strengthen the existing foundations.

Strengthening of foundations must be performed with the use of restoration materials or materials prepared on the basis of spectral analysis of ancient material selected samples. The aforementioned condition was agreed with the restorers and it is necessary to preserve the authenticity of the object.

In the places of absence or rubble foundations boulders selection, a bookmark of new ones is necessary. Formed cavities between the boulders of foundations should be filled with a lime solution.

It is necessary to conduct all the works to strengthen foundations under the continuous supervision of specialists using the latest equipment to prevent the deterioration of the church structures state or emergency situations.

Conclusions:

1. It is necessary to develop normative documents regulating the conduct of survey works to study the stress-strain state of historical and cultural heritage sites that will ensure the maximum preservation of cultural heritage in terms of objects authenticity.

2. Introduction of innovative technologies in the above-described work allowed with a sufficient degree of quality to determine all the parameters necessary for study were the reasons for the deterioration of the object technical condition have been clarified.

3. It is needed a comprehensive systematic approach to the analysis of archaeological data, technical characteristics of the object and geological studies.

4. The solutions proposed by the authors for strengthening the foundations of the cultural heritage object make it possible to preserve the authenticity of the building.

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THE REDISTRIBUTION OF THE HEIGHT RETAINING WALLS LEVELS, ITS EFFECT AT THE STRESS-STRAIN STATE OF THE SYSTEM «RETAINING STRUCTURES – SOIL MASS»

The work of multi-level retaining walls in sandy loam soils is investigated. A numerical experiment was conducted for reveal the most rational choice of the level height at a constant total excavation depth. A three-level retaining wall is considered. A number of tasks have been solved. The depend values changing of displacement and internal effort on the redistribution of excavation levels is shown. Values are fixed in the characteristic points of the structural elements of retaining walls each level. Variables are different at level marks of retaining walls. The surfaces were created on bases of the obtained results. These surfaces are used to analyze the relationship between the heights of levels and the values of bending moments. Identified solutions lead to increased displacements in one or another level of retaining walls. The constitutive laws between the geometric parameters of the retaining walls and the stress-strain state of the system «retaining constructions – soil mass» are obtained.

***Keywords**: multi-level retaining wall, heights of the level, stress-strain state, horizontal displacements, bending moments.*

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ПЕРЕРОЗПОДІЛ ВИСОТ ЯРУСІВ ПІДПІРНИХ СТІН, ЙОГО ВПЛИВ НА НАПРУЖЕНО-ДЕФОРМОВАНИЙ СТАН СИСТЕМИ «УТРИМУЮЧІ КОНСТРУКЦІЇ – ҐРУНТОВИЙ МАСИВ»

Досліджено роботу багатоярусних підпірних стін у супіщаних ґрунтах. Числовий експеримент проведено для виявлення найбільш раціонального вибору висот ярусів при сталому загальному перепаді ґрунту. Розглянуто триярусну підпірну стінку, розв'язано ряд задач, де змінними є позначки окремих ярусів. Показано залежність зміни значень переміщень і внутрішніх зусиль від перерозподілу рівнів розробки ґрунту. Значення зафіксовано у характерних точках конструктивних елементів окремих ярусів підпірних стін. За отриманими результатами побудовано поверхні. За графіками проаналізовано залежності висот ярусів і згинальних моментів у цих ярусах. Визначено рішення, що призводять до збільшення переміщень у тому чи іншому рівні підпірних стін. Отримано закономірності між геометричними параметрами підпірної стіни і напружено-деформованим станом системи «утримуючі конструкції – ґрунтовий масив».

***Ключові слова:** багатоярусна підпірна стіна, висота ярусу, напружено-деформований стан, горизонтальні переміщення, згинальний момент.*

Introduction. The planning of areas with complex terrain in a modern city often needs the excavation pits with a depth of about 20 m. The construction of a retaining wall of this height provides for the mandatory use of ground anchors, node-to-node anchor, or any other limiters of horizontal displacements. Such structural elements are usually located in the neighboring territory. These structures can be cut by the following construction. Therefore, one of the solutions for protecting the territory can be the use of several levels of retaining walls.

The latest sources of research and publications analysis. Stability of slopes and methods of numerical modeling of landslide-prone areas were investigated by M. L. Zotsenko and Y. L. Vinnikov [1]. Influence of piles, which are located at the level of soil excavation, on retaining wall is considered in the article of Katzenbach G. [2]. This article presents the results of numerical simulation, which are based on a series of full-scale tests of a reduced model, and the corresponding dependencies are revealed. The methods for determining the parameters of retaining walls was considered in the A. L. Gotman`s paper [3]. Also, the issue of numerical modeling and comparison of the obtained results with geodetic monitoring data was considered by such scientists as I. Skrzypczak, J. Kogut, W. Kokoszka [4], C.V.S. Benjamim, B.S. Bueno, J. G. Zornberg [5] and others.

Distinction of unresolved parts of a common problem. The choice of the planned–high-altitude position of the levels of the retaining walls is an important factor, as well as an understanding of how the change in the position of levels affects to the stress-strain state of the system «retaining structures – soil mass». The results of the numerical experiment on the change in the redistribution of heights of the three-level retaining wall, and the effect of this redistribution on the operation of each of the tiers are described in this paper.

Formulation of the problem. The number of problems is solved to investigate the influence of the distribution of excavating heights of levels of the retaining walls on the stress-strain state of the system «retaining structures – soil mass». The calculations were performed using a model of a physically nonlinear elastic-plastic environment, based on the dilatancy theory of Professor V.M. Nikolaevsky [6]. This soil model is realized in the program complex ASSR «VESNA». Part of the dilatancy theory, which describes the dependence of the critical density level, ρ_{cr} on the hydrostatic pressure, σ_m, was supplemented by Professor I.P. Boyko [7, 8] by the following equations:

$$\begin{aligned}
&\rho_{cr_i} = \rho_{cr_0} \text{ at } \sigma_m > 0\\
&\rho_{cr_i} = -\frac{2(\rho_{cr_max} - \rho_{cr_0})}{P_0^2}\sigma_m^3 + \frac{3(\rho_{cr_max} - \rho_{cr_0})}{P_0^2}\sigma_m^2 + \rho_{cr_0} \text{ at } P_0 \le \sigma_m \le 0;\\
&\rho_{cr_i} = \rho_{cr_max} \text{ at } \sigma_m < 0,
\end{aligned} \tag{1}$$

where ρ_{cr_i} – currently critical density;

ρ_{cr_0} – critical density in the absence of all-round compression;

ρ_{cr_max} – the maximum critical density for a given soil;

P_0 – a parameter of the soil that determines the level of hydrostatic pressure at which the transition from the conical surface to the cylindrical surface. Equation (1) shows that the critical density ρ_{cr_i} increases with the hydrostatic pressure σ_m.

The proposed relationships also provide an opportunity for more complete description of soils elastic-plastic deformation processes. Because these relationships allow to consider elastic volume deformations of the soil, while they are in a critical state. The value of the critical density ρ_{cr_i} is variable due to the redistribution of the hydrostatic pressure σ_m in the investigated region.

The purpose of this work is to determine the regularities between the height of the levels of the retaining wall and the components of the retaining structure levels stress-strain state.

The main material and results. A three-level retaining wall with the following input constant geometric parameters for research has been selected. This retaining wall has some geometric parameters: excavation height of the pit, *H;* total distance between levels, *L=2/3H;* distance between each of the levels in the plan*, 1/2L.* Variable for these tasks were height of each level. The calculation scheme for this task set is shown in Fig. 1. The tasks are solved by numerical simulation in a flat formulation.

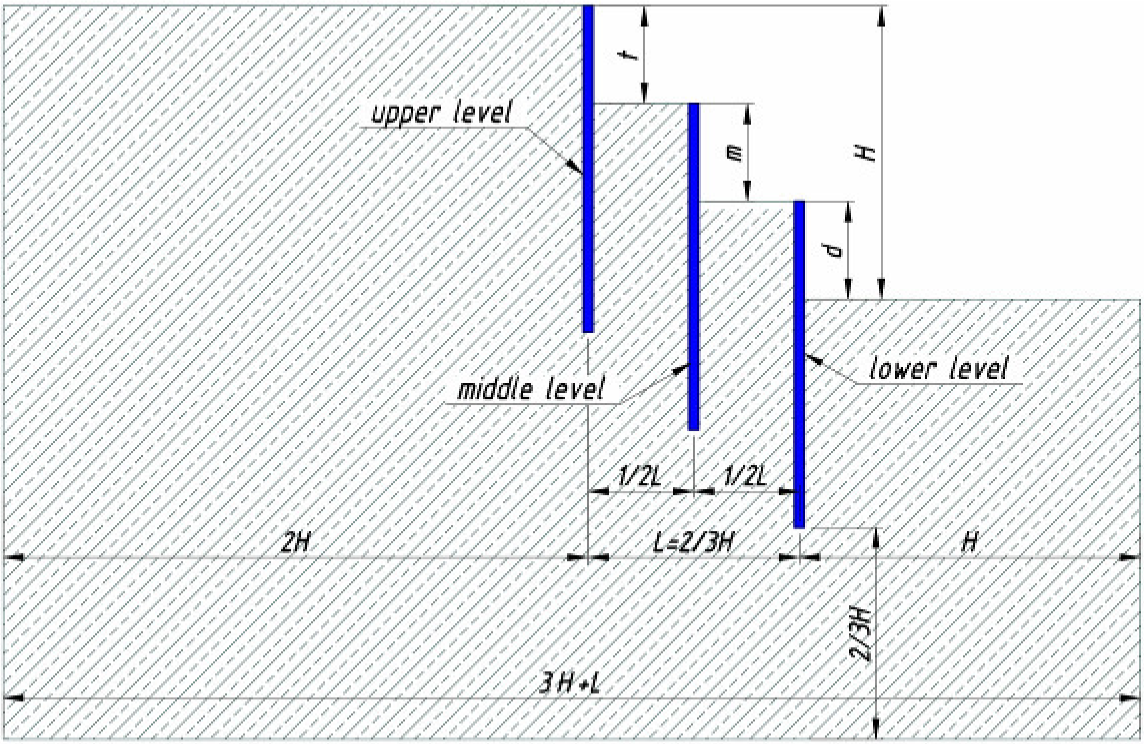


Figure 1 – Calculation scheme for determining the effect of the mutual position of the height of levels of retaining walls on the stress–strain state of the system «retaining structures – soil mass», for sandy loam soils, with the relation L = 2 / 3H

The height of the lower level is *d*, of the middle – *m*, of the upper – *t*. The regularities between these parameters and the components of the stress-strain state are given in the paper.

The calculations are performed for tasks with the following input parameters: *H* = 18 m, respectively *L* = 12 m. The retaining walls are in a homogeneous loam soils to exclude the effect of soil layers on the stress-strain state of the system «retaining structures – soil mass». Sandy loam has the following physical and mechanical characteristics: specific weight, 20,2 kN/m^3; Poisson`s ratio, ν = 0,3; cohesion, *C* = 15 kPa; friction angle, φ = 26; deformation module, *E* = 38 MPa. The stiffness of the retaining walls, *EI* are the same. All retaining walls are made of bored piles with a diameter of 820 mm, in a 1 m step.

The values of *t*, *m*, *d* are given in Table 1. Also, the results of a numerical experiment, such as the values of maximum bending moments and displacements of the retaining walls top of each level is shown in the table.

The dimensions of the finite element scheme, the sizes of the finite elements, their type and the grid thickening zone are the same for all solved tasks. It allowed to avoid the accumulation of errors in numerical modeling Also, during the modeling of pit excavation, it is important to consider the calculation stages of excavation and the technological sequence of retaining walls levels construction. The results of numerical simulation differ by 15%, when digging out the ground in stages 1 and 2 m. The lack of technological stages of construction in the calculation shows a difference between the results of horizontal deformations of more than 2 and almost 3 times. The results of the soil massif deformation distribution are incorrect without considering the stage of construction. Therefore, the step-by-step calculation and modeling with including phases of retaining walls erection and pit excavation is a necessary component of the numerical modeling of the system «retaining structures – soil mass». The solution of all tasks is performed in 13 stages, and the excavation of the pit is modeled by uniform steps.

Table 1 – Results of numerical modeling of a three-level retaining wall with different distribution of height levels

d, m	m, m	t, m	Displacement the top of piles, mm			Maximum bending moments, kN·m/r.m.		
			Upper level	Middle level	Lower level	Upper level	Middle level	Lower level
4	4	10	39.4	76.4	225	424.7	517	620
4	6	8	41.3	96.8	160.8	432	535	335
4	8	6	45.7	133	151	458.5	613.53	297
6	4	8	55.9	68	122.4	521.3	352	264.6
6	6	6	61.7	94	107	560	371	193
6	8	4	72.5	137.68	125	648	358	237
8	4	6	84.06	80.3	94	649	333.4	156
8	6	4	96.8	115.7	108.5	734	320	195.7
8	8	2	121.6	185.6	124	931.2	390.1	204

The obtained data must be visualized in the form of graphs surfaces. For convenience, it was denoted any investigated component of the stress-strain state S. The main task is to associate 4 parameters: d, m, t, and S in one scheme (one of the obtained data components: displacement the top of piles or maximum bending moments of the levels). This dependence requires the construction of a 4-dimensional surface. It is associated the variation parameters by the following logical equation (2) in order to go to three-dimensional space and visualize the results in the form of iso poles

$$d + m + t = H, \tag{2}$$

where H – excavation height of the pit.

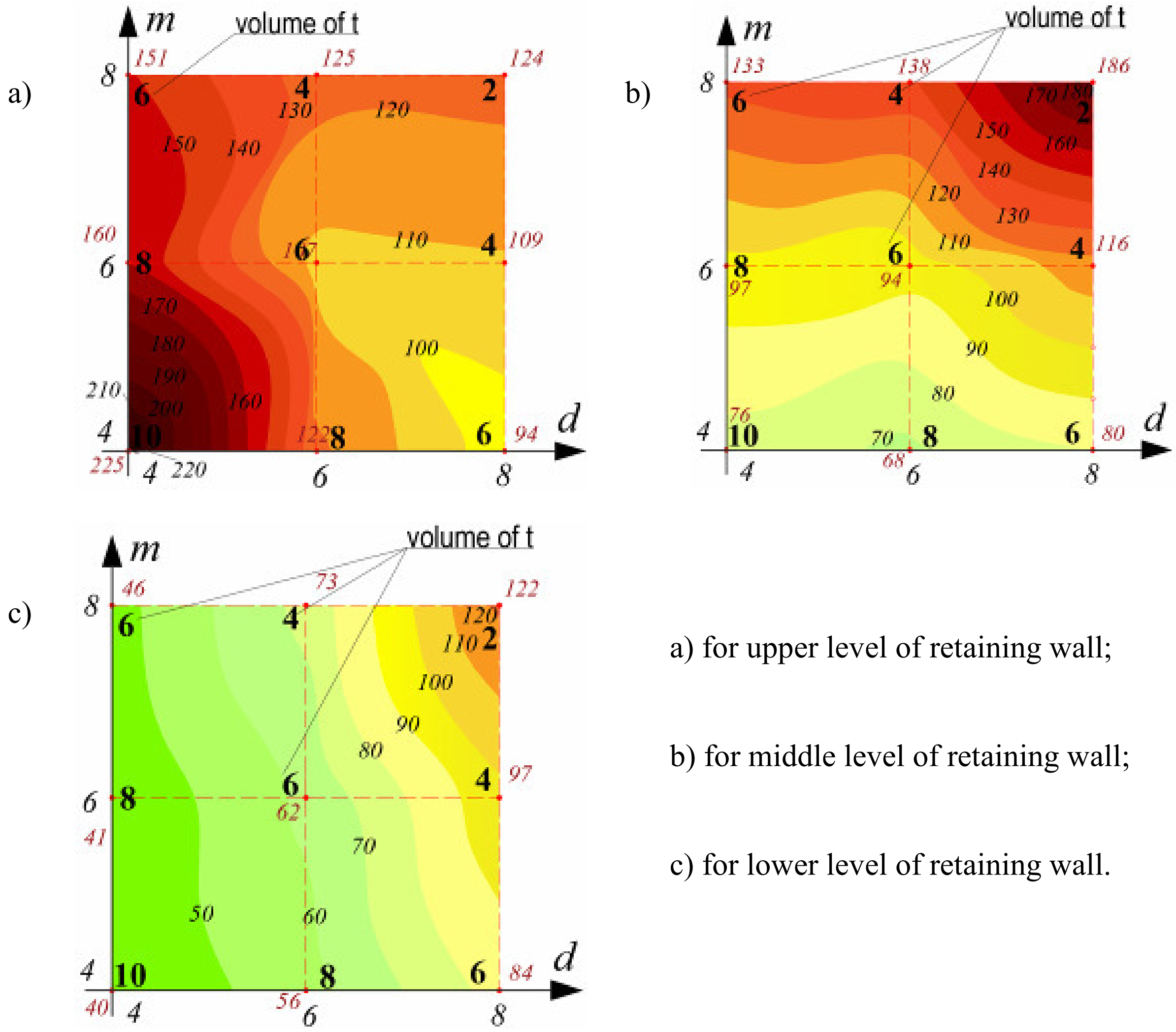
Using equation (2), it can always be expressed the height of the upper level, t, presetting the heights d and m:

$$t(d,m) = H - d - m. \tag{3}$$

Now, any physical quantity S can be found as a function of only two variation parameters (d and m):

$$S = S(d,m) = f(d,m). \tag{4}$$

Thus, it is got themethod for constructing dependencies between the components of the stress-strain state of the system «restraining structures – a soil mass» and the geometric parameters of a three-level retaining wall. The projections of the surfaces for horizontal displacements of retaining walls each level are shown in Fig. 2. The values of t are shown in the control points on the projections, for convenience of use.



a) for upper level of retaining wall;

b) for middle level of retaining wall;

c) for lower level of retaining wall.

Figure 2 – Horizontal displacements of the top of the retaining walls

The height of the upper and horizontal displacements of the upper level is increased and level of excavation of the lower level decreases respectively. In addition, the minimum values of horizontal displacements correspond to the values $d = 8$ m, $m = 4$ m, $t = 6$ m, thus, the height of the middle level should be less than all others.

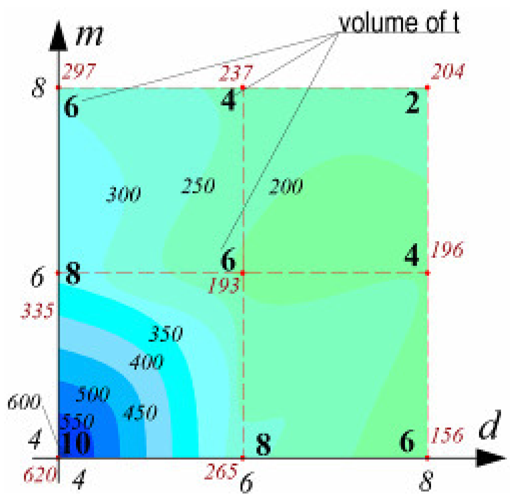
It is expected that an increase in the middle level height of the retaining walls causes an increase in the horizontal displacements of this level (Figure 2, b), but at the same time, an increase in the lower level from 6 to 8 meters also shows a significant increase in the deformations of the middle retaining wall. The minimum values of the horizontal displacements of the retaining walls middle level correspond to the values of the heights $d = 6$ m, $m = 4$ m, $t = 8$ m, the maximum displacements correspond to – $d = 8$ m, $m = 8$ m, $t = 2$ m. That is, increasing the height of the lower level more significantly affects the displacements of the middle retaining wall than the increase in the upper one. This is explained by the fact that the volume of soil, which retains the middle retaining wall, decreases with increasing height of the lower level.

The lower level of the retaining walls has received the smallest absolute horizontal displacements for almost all the considered variants of levels heights distribution. The horizontal displacements of the lower tier had only 4% higher values than for the middle tier only for the altitude relation $d = 6$ m, $m = 4$ m, $t = 8$ m. The obtained isopoles of horizontal displacements of the lower level (Fig. 2, c) showed that the increase in the deformations of this level is due to increase in its height in a greater degree. The increase in the heights of the middle and upper levels showed the least influence on the displacements of the lower level. It can be explained by the fact that the redistribution of the loads level to the lower tier in a lesser way affects the deformations of the retaining wall than the volume of the soil mass that retained this level.

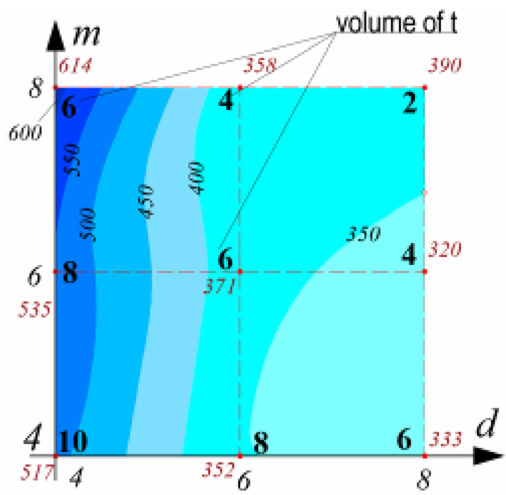
Analysis of the calculations results was limited to tasks with the values d, m and $t = 4...8$ m. It was done to compare the effect of mutual redistribution of the levels heights on the each horizontal displacements. Studies have shown that the difference between the smallest and the largest values of horizontal displacements in the given range of levels heights is 57 % for the lower level; 50,6 % for the middle level; 41 % for the upper level.

The next investigated component of the stress-strain state of the system «retaining structures – soil mass» is the maximum values of the bending moments in the piles of the retaining walls of retaining structure each level. The values of the bending moments are determined at the last stage of modeling pit excavation. The results obtained in the form of a surface projection are shown in Fig. 3.

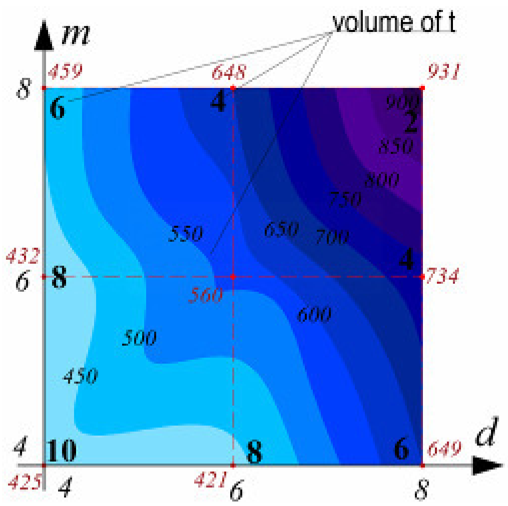
a)



b)



c)



a) for upper level of retaining wall;

b) for middle level of retaining wall;

c) for lower level of retaining wall.

Figure 3 – Maximum bending moments in the piles of retaining walls each tier

The minimum values of the bending moments in the piles of the upper retaining wall correspond to the values d = 6 m, m = 4 m, t = 8 m. For the upper level, such a position of the retaining walls, also corresponds to the lowest value of the horizontal displacements of this level within the studied values of heights redistribution. The maximum values of the bending moments of retaining walls upper level correspond to the values d=4 m, m = 4 m, t = 10 m and exceed the values of the bending moments for d = 8 m, m = 4 m, t = 6 m by almost 4 times. Our research is limited by the number of solved problems. Range of the solutions (Figure 3a) shows that the position of the middle level influences to the forces in the upper retaining wall in a large way than the position of the lower one. It is analyzed the solutions for a constant value of t = 6 m, in order to investigate the influence of the height of the lower levels on the bending moments of the upper one. It was found that the change in the heights of tiers d and m in the range from 4 to 8 m in different ratios showed the difference between the values of the bending moments of the upper level to 47 %, that is, almost 2 times.

The maximum values of the bending moments in the middle level of the retaining walls correspond to the maximum value of this level height and the minimum possible value of d in the given range of input data. The distribution of the heights d = 4 m, m = 8 m, t = 6 m corresponds to the maximum value of the bending moments in the piles of the middle level, which is almost 3 times greater than the value of the bending moments for d = 8 m, m = 6 m, t = 4 m. This redistribution of the levels heights corresponds to the minimum value of efforts in the constructions of the middle level. Analysis of the graphs showed that the values of the bending moments of the middle tier are significantly influenced by the geometric dimensions of the other tiers, since the minimum bending moments of this tier do not correspond to its minimum height. It is analyzed the value of the bending moments of the middle retaining wall at a constant of its own height, m = 6 m, and the values of d and t in the range 4...8 m, as well as for the upper level. The difference between the smallest and largest values of the bending moments is 40 %.

Most of the researched redistribution variants showed that the greatest bending moments occur in the lower retaining wall. The moments in the piles of lower retaining wall are not the largest among all the levels only when d = 4 m. It can be explained by the fact that the pressure on the lower retaining wall is always greater than on the middle or upper, but the volume of the retaining soil mass depends only on the soils own height and does not change in any way regardless of the other levels heights. The maximum values of the bending moments of the lower levels of the retaining walls correspond to the values d = 8 m, m = 8 m and t = 2 m; the minimum values of the bending moments arise at d = 4 m, m = 4 m and t = 10 m. The difference between the minimum and maximum values of bending moments within the range of values is 54%, that is, more than twice. However, it is important to consider that the researched range of changes in the heights of soil mass for each levels is different. It is limited the comparison of data as follows: d = 6 m; m and t can be variable in the range from 4 to 8 m, as well as for the upper two levels. The difference between the maximum and minimum values of the bending moments with a constant value of the height of the lower level is 24 %.

The maximum bending moments that appear in one or another level of retaining wall in all the researched cases correspond to such a mutual height arrangement of other levels where the own height of the wall level is greatest. The minimum bending moments of each levels do not always correspond to its lowest height. The dependence, at which the minimum own height corresponds to the minimum moment, is valid only for the lower level. For upper level, the redistribution of neighboring levels heights has a greater influence on the values of the bending moments than the own heights.

Conclusions. The construction of multi-level retaining walls allows retaining significant changes in the ground without horizontal limiters use, and at the same time, using the area of the construction site for building, by integrating the retaining structures in the architectural solutions of the new building.

It has been established that the maximum movements of the levels piles top depend largely on their own heights. The change in the height of the level leads to loads redistribution throughout the structure. The upper level is the most sensitive to the change in the geometric parameters of the retaining walls in percentage terms. Thus, the differences between the smallest and largest values of horizontal displacements are: 57 % – for the lower level; 50,6 % – for the middle level; 41 % – for the top level.

The researching of soil mass influence redistribution between levels showed that neighboring levels heights influences on the value of the bending moments are: for the lower level – 24 %, for the average level – 40 %, for the upper level – 47 %.

The visualization of the obtained results in the form of isofields makes it possible to evaluate the influence of the level heights on the components of the stress-strain state of the system «soil massif – retaining structures», which in turn allows to justify the rational dimensions of the retaining structures in each specific case.

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PROBLEMS OF SUBWAY CONSTRUCTION IN COMPLEX ENGINEERING AND GEOLOGICAL CONDITIONS BY THE EXAMPLE OF KHARKIV CITY

Scientific, technical and practical problems connected with the construction of subways, such as complex engineering and geological conditions, tunneling need in existing urban development conditions, unfavorable processes emergence during construction and operation are considered. A number of resonant factors that require special decisions when constructing the third subway line ine Kharkiv city have been identified. These factors are large dewatering influence radius threaten surface subsidence and damage of building structures in the area of open tunneling; barrage impact on the groundwater flow in the area of closed tunneling; threat to several buildings located above the tunnels; threat of weak soils dumping in the tunnel face; vibration impact on soils, buildings and structures. Seismic effect assessment on Kharkiv subway objects is discussed.

Key words: *subway, complex conditions, soft soils, accidents*

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ПРОБЛЕМИ БУДІВНИЦТВА МЕТРОПОЛІТЕНІВ У СКЛАДНИХ ІНЖЕНЕРНО-ГЕОЛОГІЧНИХ УМОВАХ НА ПРИКЛАДІ М. ХАРКОВА

Розглянуто науково-технічні та практичні проблеми, пов'язані з будівництвом метрополітенів, серед них: складність інженерно-геологічних умов, необхідність проходки тунелів в умовах існуючої міської забудови, виникнення несприятливих процесів при будівництві й експлуатації. Виявлено ряд резонансних факторів, що вимагають прийняття спеціальних рішень під час будівництва III лінії метро в місті Харкові: великий радіус впливу водозниження, що загрожує осіданням поверхні та пошкодженням будівельних конструкцій на ділянці відкритої проходки; баражний вплив на потік підземних вод на ділянці закритої проходки; загроза кільком будівлям, розташованим над тунелями; загроза вивалів слабких ґрунтів у забою тунелю; вібраційний вплив на ґрунти, будівлі й споруди. Обговорено оцінку сейсмічного впливу на об'єкти Харківського метрополітену.

Ключові слова: *метрополітен, складні умови, слабкі горнути, аварії.*

Introduction. As a rule, a number of scientific, technical and practical problems are connected with subway construction. Among the main challenges there are complex engineering and geological conditions, need for tunneling in the conditions of existing urban development, emergence of unfavorable processes during subways construction and operation.

For such cases the existing standards provide survey scientific and technical suppor, design and construction works [1]. Thus, at the «survey» stage, it is often necessary to study the filtration properties of various soils, soil ability to soften under vibration dynamic influences, to assess surface dehydration-gravitational subsidence emergence possibility during dewatering and frost heaving. It is necessary to evaluate the vibration impact on existing buildings and structures, stability of buildings and structures when undermining or close tunneling, excavation pits, tunnel crown and face walls stability etc.

In order to carry out the relevant studies, special support is required by development of technical task development and its implementation. The received survey information with scientific and technical support makes the basis for multi-variant design solutions. And in each case the problems of reliability, safety and economic efficiency must be solved. And if it is considered every point of the geological space to be unique, design solutions development is a rather complex and science-intensive task. For example, by construction of the subway station «Yuzhnyi Vokzal» in Kharkiv it was necessary to apply freezing of watered soils with weak structural bonds, instead of traditional construction dewatering, which would cause significant deformations of existing buildings and structures.

Analysis of recent research sources and publications. Examples of dangerous consequences arising from incorrect design solutions adoption in complex engineering and geological conditions by subway tunnels constrctions are given in a number of works [2 – 10].

Underground structures construction experience in complex engineering and geological conditions in Kharkiv, Kiev, Copenhagen, St. Petersburg and a number of other cities shows that the following negative and dangerous phenomena must considered in construction projects:

– possible inrush of thixotropic liquefied soils into tunnels and excavations, which leads to long breaks in construction and requires significant costs for accident elimination;

– dehydration-gravitational settling of the soil body and surface during long-term dewatering. In this case, the uneven settlement of closely located buildings with shallow foundations can make more than 15 cm (i.e. many times exceeding the regular value);

– suffosion sinkholes after a soil collapse at the tunnel crown.

Identification of general problem parts unsolved before. It should paid special attention to constructing subways problems in complex engineering and geological conditions. At the same time, it must be considered that no model (project) can fully correspond to real conditions. The reasons lie in both complex natural conditions and certain simplification of engineering solutions and sometimes their incomplete correspondence. For example, cutting the crown when tunneling in clay soils in the territory of Shevchenko garden in Kharkiv led to significant settlement of overlying soil body and even to sinkhole formation with the threat of movie and concert hall «Ukraina» destruction». It was needed to eliminate this suffosion sinkhole without stopping subway operation and short-term hall closedown. Other cases of emergency situations during construction are connected with water and soil inrush, as well as excessive buildings settlement which occurred near stations «Prospekt Haharina» and «Ploshchad Konstitutsii».

Problem definition. Nowadays during extension of the third subway line (from «Metrobudivnykiv» to «Odesskaia» station) it becomes necessary to solve a number of problems related to dewatering in the area of tunneling using cut-and-cover method, tunneling under existing buildings, tunneling using caisson method in soft thixotropic soils etc.

Obviously, complex of difficult tasks cannot be solved by single design organization, and scientific and technical support should be provided by skilled professionals from several specialized organizations. Many years of experience shows that scientific and technical support is necessary not only at the stage of survey and design, but also by performance of construction works. Experience shows negative tendencies in design, survey and construction works. First and foremost, this is performers professional level decrease and quality management system lack. Expertise is not able completely eliminate these shortcomings.

Basic material and results. Scientific and technical support of design and survey works on construction of the third subway line in Kharkiv was performed by the staff of the Geotechnics and Underground Structures Department of Kharkiv National University of Civil Engineering and Architecture.

As a result of available design and survey materials study, as well as results of -territory site survey, there was identified a number of resonant factors, which required special solutions:

– wide spreading of soft and specific soils that complicate construction conditions;

– high groundwater level in some areas; barrage effect of tunnels by the drainage absence;

– vibration impact on soils, buildings and structures;

– pressure on the constructions enclosing excavation pits.

In the engineering-geological section of the territory, 34 engineering-geological elements were identified. Soils are heterogeneous in terms of their lithological composition, genesis, textural and structural features and nomenclature. Soils significantly differ in their properties and mode of occurrence. Soil depth changes rapidly, soil lensing can be often observed (Fig. 1).

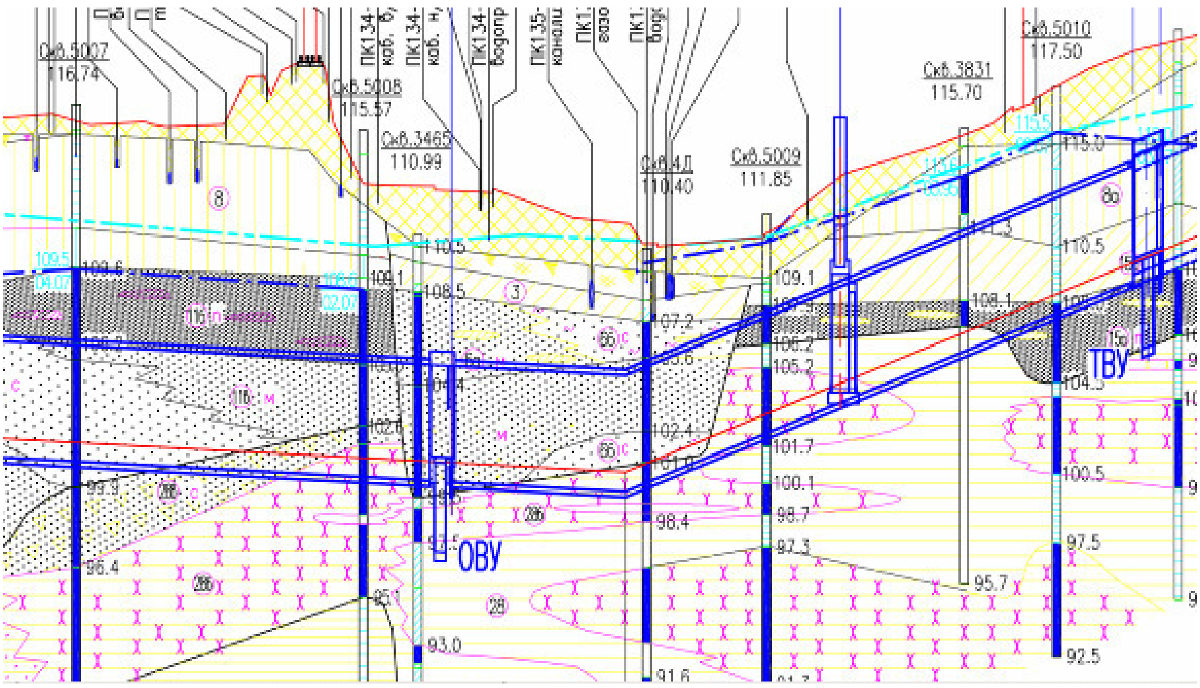

Figure 1 – The most complex section where the tunnels are crossing Hlybokii Yar ravine

Within the studied depth, there are two aquifers in impaired mode with low water pressure and possibility of negative geological and hydrogeological processes development, both in construction period and during operation. In accordance with DBN A.2.1-1-2014, ground conditions relate to the highest category of difficulty III a.

One of the major resonance factors in the area where tunnels are crossing Hlybokii Yar ravine is a high groundwater level (0.5 m beneath the ground surface), which means that tunneling under this level requires taking a number of special measures. Such measures include: construction dewatering, which provides possibility of constructing auxiliary tunnel facilities using cut-and-cover method; installation of a culvert or drainage system to prevent barrage effect and flooding of the upstream buildings.

Complex geotechnical conditions are connected with the fact that the subway route passes through several geomorphological elements having complex geological structure (three Lopan River terraces), crosses Hlybokii Yar ravine and filled up Sychevskii gully.

Complex construction conditions are connected with wide spreading of watered, semi-stable and unstable soils, as well as subsiding soil in some areas, presence of operated railway tracks and numerous utility lines. As tunnels are near to waterproof ground determined use of a powered tunneling aggregate for tunneling.

The radius dewatering system influence and possibility of dehydration and gravitation processes development under the footing of existing buildings and structures were not considered in the project of construction dewatering. In addition, the presence of dusty sands watered lenses makes tunnel flooding risk during construction works and by installation of excavation pits slops. Thus, in the presence of such lenses it is necessary to thicken a network of water-dropping wells or wellpoints. To exclude the negative impact on the environment due to the barrage effect of subway tunnels and stations, drainage is used in the flow of groundwater in the areas of embedding into the aquifer.

Vibration impact on soils during construction may cause thixotropic liquefaction of silty-clayed soils and fine sands and, correspondingly, flooding of excavation pits and tunnels with liquefied soils and sands. Due to possibility of the negative impact of vibration from underground trains passing closely to buildings and structures (Fig. 2), it is necessary to strictly comply with the normative remoteness of these objects or development and application of special protective measures.

In potentially flooded areas network of observation wells to control groundwater level should be equipped regardless of the construction work process. Organization of monitoring observations and analysis of their results allows to quickly assess impact of construction on the hydrogeological conditions.

In the area of tunneling using cut-and-cover method, reliability and safety of work is ensured by secure fixation of the pit walls. Installation of metal piles and filling in the space between the piles with wooden shields are most effective in this area under the condition of construction dewatering.

The issue of estimating seismic conditions is considered separately, because according to DBN B.1.1-12-2014 seismic intensity in the territory is estimated in 7 points. This evaluation requires special consideration, as the territory of Kharkiv is far from seismogenic areas. Formal application of regulatory requirements will lead to unwanted rise in construction costs.

Conclusions. Scientific and technical support the third subway line construction in Kharkiv contributed to identification and consideration of the following adverse processes and phenomena:

– barrage impact on groundwater flow in the area of tunneling using tunneling machines;

– threat to several buildings and structures in the areas where tunnels run close to the surface level (instability of the crown with regard to vibration impact);

– threat of soft thixotropic soils inrush into the tunnel face;

– soil conditions and intensification change of soil pressure on the enclosure structures of the excavation pits in the area of tunneling using cut-and-cover method.

a)

b)

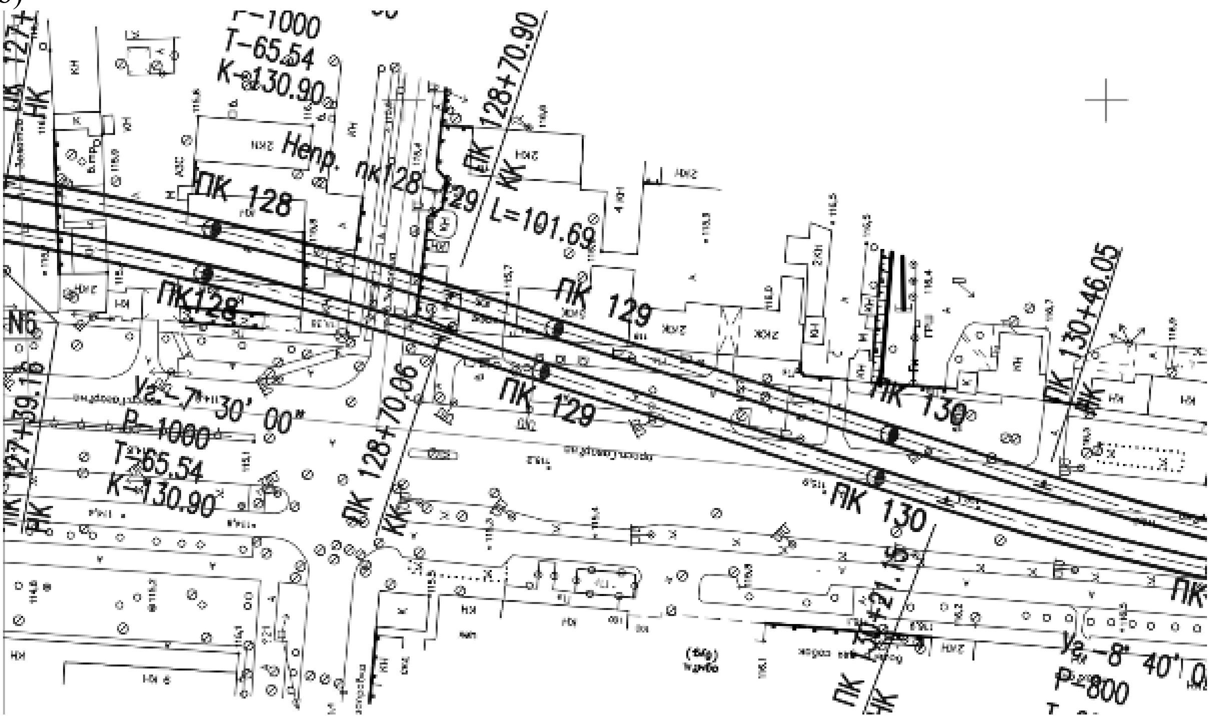

Figure 2 – A solid five-story building located directly above the tunnel been designed:
a – a photo (January 2017); b – a design plan of the tunnels

Considering the problems of subways construction in complex engineering and geological conditions, it should be noted that almost any construction in a large city requires solving many engineering, environmental and economic problems. To find solution of such problems it is necessary to involve leading scientific organizations and specialized departments of higher educational institutions. It is not within the framework of traditional tender procedures and requires special legislative regulation.

Unfortunately, it should be noted that the tendency to minimize costs for the complex of construction works (e.g. call for bids) contributes to quality reduction. As a result, small savings can lead to multimillion losses or emergency situations.

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RATIONAL PLANNING OF PILE FOUNDATIONS

The existing methods (design directory) recommend using nomograms depending on the eccentricity of bending moment and numbers of piles while acting only vertical loads. These methods have a number of shortcomings and are inconvenient, because of lack of adequate alternatives and they do not provide the greatest efficiency. The convenient method of pile foundation designing (grillages and fields), depending on the load values, is the foundation proposed by the authors in this article. The results of numerical investigations determined the optimal grillages dimensions and numbers of pile in "bush" of pile, showed that this method gives optimal options for pile foundation under single column with large bending moments and minimum material expenditures and also provides its bearing capacity

***Keywords**: the foundation, pile, bearing capacity calculation.*

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РАЦІОНАЛЬНЕ ПРОЕКТУВАННЯ ПАЛЬОВИХ ФУНДАМЕНТІВ

Існуючою методикою, яка наведена в довіднику проектувальника, передбачено проектування пальового фундаменту за допомогою номограми залежно від ексцентриситету дії згинального моменту та необхідної кількості паль при дії лише вертикального навантаження. З'ясовано, що вона досить громіздка, незручна і має ряд недоліків, а саме: не вказує на будь-який альтернативний варіант і не забезпечує одну з основних вимог щодо того, що запроектований варіант повинен бути найбільш економічним. Розроблено зручну та зрозумілу методику розрахунку й конструювання пальового поля та ростверків залежно від дії розрахункових зусиль на фундамент з використанням таблиці або графіка, які є зручними при проектуванні. Виконаними числовими дослідженнями й установленими оптимальними параметрами розмірів ростверків та кількості паль у кущі показано, що застосування запропонованої методики дозволяє запроектувати пальовий фундамент з оптимальними параметрами під окремо стоячу колону при дії значного згинального моменту з мінімальними витратами матеріалу з одночасним забезпеченням його несучої здатності.

***Ключові слова:** фундамент, паля, несуча здатність, розрахунок.*

Introduction. The main task of pile field and metal grills planning is to provide maximum allowable pile load capacity, to ensure the uniformity of foundation materials and pile-soil, to determine optimal sizes and unification of piles and grills, to calculate the lowest cost of installation and placement, as well as the minimum earthwork volumes at their installation.

As a rule, under bearing walls piles are placed in a single row and this does not cause any special complications for the designer. Frame structures determine usage of minimal number of piles in the bush with minimum-size grillages. The methods that are used for eccentric foundations and concentrically loaded foundation depending on the eccentricity of bending moment are simple and clear [2 – 4]. When stand-alone columnar foundation perceives significant bending moments, there are complications caused by the impossibility to ensure the fulfillment of all conditions.

These options are appropriate for industrial steel or reinforced concrete framed buildings with a heavy crane load, for high-rise buildings, which perceive significant wind loads, for pressure gravity retaining walls, etc.

Analysis of recent researches and publications. According to the design standards [5, 6], calculation and planning of pile concentrically loaded foundation with separately standing columns on the same plane defines carrying capacity of hanging piles F_d and allowable load P. Further, depending on size of the external load, the required number of piles is determined and the pile bush is constructed. Then the following conditions are checked:

$$\left.\begin{aligned} N &= G_{nI} + \frac{N_I + G_{pI}}{n} \le P \\ N_{max} &= G_{nI} + \frac{N_I + G_{pI}}{n} + \frac{(M_I + Q_I \cdot h)\cdot x_{max}}{\sum x_i^2} \le 1{,}2 \cdot P \\ N_{min} &= G_{nI} + \frac{N_I + G_{pI}}{n} - \frac{(M_I + Q_I \cdot h)\cdot x_{max}}{\sum x_i^2} \ge 0 \end{aligned}\right\}, \tag{1}$$

where N_{max}, N_{min} – maximum and minimum load on extreme piles;

G_{pI} – grillage and soil weight;

n – number of piles in the bush;

G_{nI}– the pile weight;

x_{max}– distance from the main axis to the extreme pile axis;

x_i – distance from the main axis to the axis of each pile;

M_I , Q_I – moment and horizontal component of external influences at the level of the foundation trimming to the corresponding axis;

h – distance from the grillage bottom to its top;

P – pile allowable loads.

Overloading of extreme piles in the bush for 20% (1,2P) from the allowable load is possible in the calculation of bases with regard to wind and crane loads (while the crane load should be more than 30% of the total load on the bases), i.e. $N_{max} \le 1{,}2P$.

If $N_{min}<0$, then it is necessary to calculate driving force for pulling loads.

In the design directory [1] the parameters of unified bush piles of square section for the single-storey and multi-storey buildings are listed, and the methods of using nomograms depending on the eccentricity of bending moment and numbers of piles while acting only vertical loads are characterized.

Identification of general problem parts unsolved before. The current methods of nomogram is used to determine the required number of piles and the distance between the centers of their axes. These methods have a number of shortcomings and are inconvenient, because of adequate alternatives lack and they do not provide the greatest efficiency. For example, in some cases if the first and third conditions of expression (1) are provided, the second condition is not ensured. In this case we need to increase the distance between the extreme driving axles, which will increase the size of grillage and therefore the construction costs; or it will increase the number of piles in the bush, leading to failure of the third condition as piles location will be changed, while bending moment influence will increase and the effect of vertical load will be reduced. So, there's no definite salvation of the problem: whether to develop methods within the frame "bush - pile", or determine the optimal number and size of piles.

Thus, there is a need to develop convenient method which clearly designs and plans the best option pile foundation without further recalculation because of one failure of the expression three conditions (1).

Setting objectives. A key point in the calculation of the foundation with large value of eccentricity is to provide third condition (1), it is true when driving bush takes considerable largest bending moments. The most rational option will be when the vertical load in the most dumped pile from the longitudinal force N_I and bending moment M_I are equal:

$$N_{min} = G_{nI} + \frac{N_I + G_{pI}}{n} - \frac{(M_I + Q_I \cdot h) \cdot x_{max}}{\sum x_i^2} \geq 0\,. \tag{2}$$

After some mathematical operations, it is got the following expression:

$$\frac{n \cdot G_{nI} + N_I + G_{pI}}{M_I + Q_I \cdot h} = \frac{n \cdot x_{max}}{\sum x_i^2}\,. \tag{3}$$

The left side of the expression (3) is inverse eccentricity of the bending moment ($M_I + Q_I\, h$) relative to the bottom of the grillages, and the right side of the expression depends on the number of piles and their location in the bush and is constant for an unaltered number of piles and the same distance between the axes of the adjacent piles x for unified bushes.

Numerical studies were conducted to determine the relationship $1/e = (n \cdot x_{max}) / \sum x_i^2$ for the unified bush piles, results of which are listed in the design directory [1] and in Table 1 or graphically in Figure 1.

Basic material and results. According to the above stated, calculation of concentrically loaded columnar foundation, depending on a bending moment, should be taking with the regard numbers of pile from the first part of the expression (1), and considering pile foundation has only 60% vertical load N_I, providing implementation of the second condition of the expression with the allowable load 1,2P, and providing the third part, which determines that the most dumped piles of bending moment and longitudinal forces are equal, i.e.

$$n = \frac{N_I + G_{pI}}{0{,}6 \cdot P - G_{nI}} \approx \frac{N_I}{0{,}6 \cdot P}\,. \tag{4}$$

Table 1 – Inverse eccentricity of the bending moment *1/e*

Number of piles in the bushes n	Distance between the axes of the adjacent piles x, м									
	0,3	0,6	0,9	1,2	1,5	1,8	2,1	2,4	2,7	3,0
2	6,667	3,333	2,222	1,667	1,333	1,111	0,952	0,837	0,741	0,667
3	11,560	5,780	3,853	2,890	2,312	1,927	1,651	1,445	1,284	1,156
4	6,667	3,333	2,222	1,667	1,333	1,111	0,952	0,837	0,741	0,667
5	4,811	2,406	1,604	1,203	0,962	0,802	0,687	0,601	0,535	0,481
6	5,000	2,500	1,667	1,250	1,000	0,833	0,714	0,625	0,556	0,500
7	4,490	2,245	1,497	1,123	0,898	0,748	0,641	0,561	0,499	0,464
8	3,421	1,711	1,140	0,855	0,684	0,570	0,489	0,428	0,380	0,342
9	5,000	2,500	1,667	1,250	1,000	0,833	0,714	0,625	0,556	0,500
10	3,733	1,867	1,244	0,933	0,747	0,622	0,533	0,467	0,415	0,373
11	3,023	1,512	1,008	0,756	0,605	0,504	0,432	0,378	0,336	0,302
12	4,015	2,011	1,352	1,014	0,831	0,673	0,569	0,505	0,451	0,401

Next step in the calculation is to determine the inverse eccentricity of bending moment in accordance with the expression

$$\frac{1}{e} = \frac{n \cdot G_{nI} + N_I + G_{pI}}{M_I + Q_I \cdot h} \approx \frac{N_I}{M_I + Q_I \cdot h} \qquad (5)$$

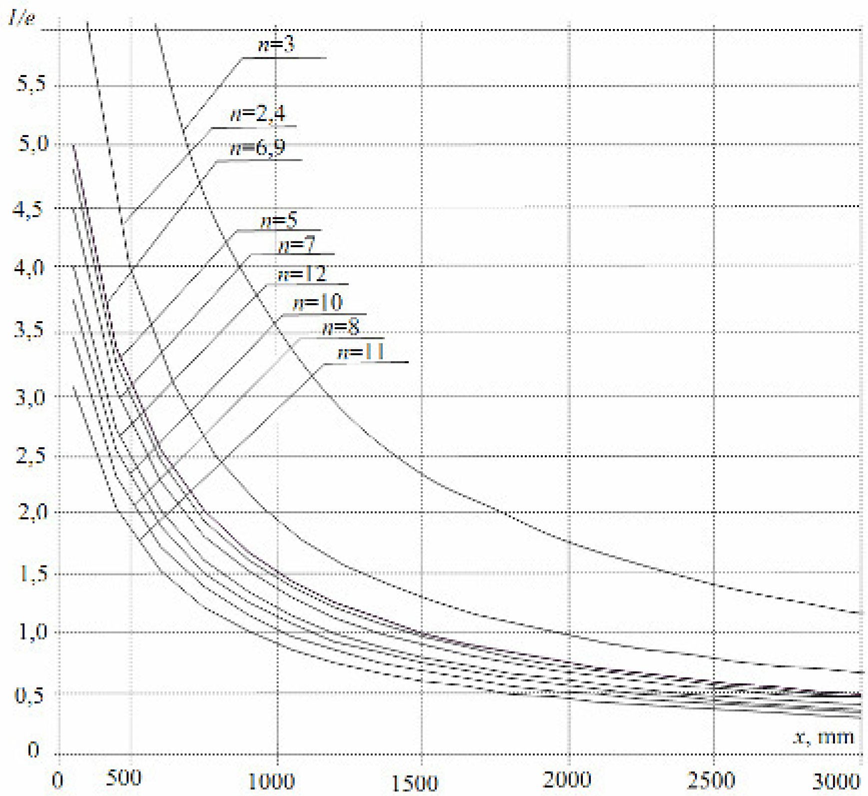

Figure 1 – Determination of the optimal parameters for bush piles planning

In the expressions (4) and (5) during the first stages of calculations the weight of the pile itself G_{nI} and the weight of the grillages G_{pI} can be neglected, because their exact values are unknown. In addition, planning of G_{nI}, G_{pI} load pile is slightly compared with vertical load N_I (≈5%), and while checking the third part of the expression (1), to neglet of their values in the beginning contributes to strength of construction.

Then taking into consideration the data from the Table 1 or Figure 1, it is determined the required distance between the axes of the neighboring piles depending on their number n and the inverse eccentricity $1/e$, or according to the eccentricity inverse $1/e$ it is accepted another number of piles n with the appropriate step x, but not less then in expression (4), as in this case first part of condition fails (1).

The graphic of the inverse eccentricity of bending moment $1/e$ shows that increasing of the piles number at the same distance between the axes does not always raise the reliability of the foundation, in particular, the foundations with three and four piles.

It is calculated the pile foundation (Figure 2) with such loads: vertical loads – N_I = 1800 kN; bending moment – M_I = 1200 kN·m transverse load – Q_I = 60 kN, h = 1,5 m; pile ПН 110.30; allowable pile load is P = 800 kN.

It is determined the required number of piles and inverse eccentricity

$$n = \frac{N_I}{0{,}6 \cdot P} = \frac{1800}{0{,}6 \cdot 800} = 3{,}75 \text{ , we take 4 piles;}$$

$$1/e = \frac{N_I}{M_I + Q_I \cdot h} = \frac{1800}{1000 + 60 \cdot 1{,}5} = 1{,}65.$$

In accordance with the Table 1 or Figure 1 distance between pile axes as a = 1,2 m is taken, alternatively it is possible to take 5 or 6 piles with the distance between them in 0,9 m and more.

Finally we check the implementation of conditions (1), determining the weight of piles and grillage:

G_{nI}=0,3·0,3·10,7·25·1,1=26,5 кN;

G_{pI}=1,8·1,8·1,65·20·1,12=119,75 кN,

$$\left.\begin{aligned}
&N = G_{nI} + \frac{N_I + G_{pI}}{n} = 26{,}5 + \frac{1800 + 119{,}75}{4} = 505{,}44\,\text{кН} < P = 800\,\text{кN};\\
&N_{max} = G_{nI} + \frac{N_I + G_{pI}}{n} + \frac{(M_I + Q_I \cdot h) \cdot x_{max}}{\sum x_i^2} = 505{,}44 + \\
&+ \frac{\left(1000 + 60 \cdot 1{,}5\right) \cdot 0{,}6}{4 \cdot 0{,}6^2} = 959{,}57 < 1{,}2 \cdot P = 1{,}2 \cdot 800 = 960\,\text{кN};\\
&N_{min} = G_{nI} + \frac{N_I + G_{pI}}{n} - \frac{(M_I + Q_I \cdot h) \cdot x_{max}}{\sum x_i^2} = 505{,}44 + 454{,}2 = 51{,}24\,\text{кN} > 0.
\end{aligned}\right\}.$$

The conditions are fulfilled, so, the pile foundation is designed correctly.

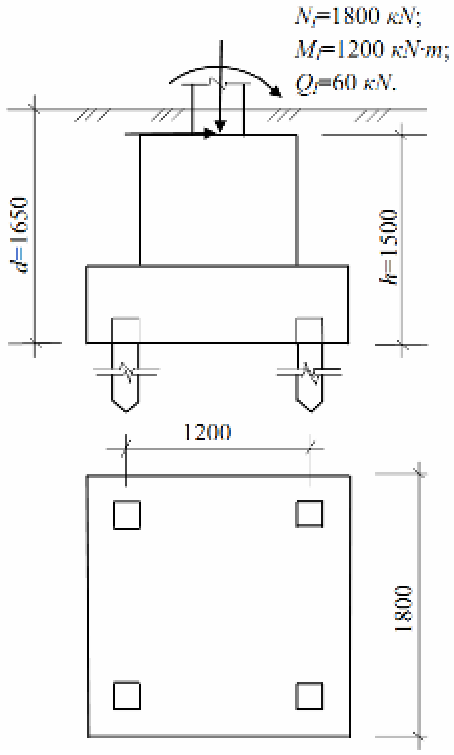

Figure 2 – Calculation of pile foundation

Conclusions. Applying of the proposed methods allows to design pile foundation with optimal options for single column with large bending moments and minimum material expenditures, while providing its bearing capacity.

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CONSTRUCTION SOLUTION OF FOLDED-PLATE SHELL FOUNDATION FOR POWER TRANSMISSION TOWERS

The article presents the practice of applying different types of shell foundations. The peculiarities of foundation design for power transmission towers under complex engineering and geological conditions are considered. It is defined that under uneven deformations of the base when applying folded-plate shell foundations on weak or water-saturated soils load from the foundation must be uniformly redistributed to the ground base. It is proposed to use an alternative construction solution of the folded-plate foundation for power transmission towers with a hinged system for fastening folds comprising supporting beams. The improved design of the folded-plate shell foundation for power transmission towers can be used on water-saturated, marshy, weak soils and under uneven deformations of the base.

***Keywords:** shell foundation, hinge system, power transmission towers, folded-plate foundation.*

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КОНСТРУКТИВНЕ РІШЕННЯ БАГАТОХВИЛЬОВОГО СКЛАДЧАСТОГО ФУНДАМЕНТУ ПІД ОПОРИ ЛЕП

Наведено досвід використання фундаментів-оболонок різного типу. Розглянуто особливості проектування фундаментів для опор ліній електропередач у складних інженерно-геологічних умовах. Установлено, що при нерівномірних деформаціях основи та використанні багатохвильових складчастих фундаментів на слабких або водонасичених ґрунтах важливим є рівномірне перерозподілення навантаження від фундаментної конструкції на ґрунт основи. Запропоновано альтернативне конструктивне рішення багатохвильового складчастого фундаменту під опори ЛЕП із використанням шарнірної системи закріплення складок з опорними балками. З'ясовано, що вдосконалену конструкцію багатохвильового складчастого фундаменту під опори ЛЕП можна використовувати на водонасичених, болотистих, слабких ґрунтах та при нерівномірних деформаціях основи.

***Ключові слова:** фундамент-оболонка, шарнірна система, опори ЛЕП, багатохвильові фундаменти.*

Introduction. While engineering power transmission towers particular attention should be paid to the choice and design of an optimal type of foundation considering a number of factors, such as soil and climatic conditions, dimensional parameters of the tower and cost efficiency. These foundations are exposed to significant weight and wind loads, wire tension and lightning protection cables loads transferring them to the ground. Power transmission towers located in specific ground conditions (weak water-saturated soils case study) require a more meticulous choice of the foundation type when the use of traditional umbrella or pile foundations is not always suitable and cost-effective. As an alternative, it is possible to use folded-plate shell foundations that can be diverse in shape, working conditions and application areas.

One of the factors slowing down the process of these foundations active use is the insufficient amount of experimental and theoretical studies on the compatible operation of folded shells with a base. However, a series of new construction solutions for such foundations has been developed recently expanding the scope of their effective use from weak soils and peats to permafrost soils.

Analysis of latest sources of research publications. Interaction between folded-plate foundation and a base and the character of their work in weak, water-saturated soils are studied by many scientists [1 – 3]. Vanyushkin S.G. investigated features of the interaction between folded-plate shell foundation and a base [1]. For the research the following types of foundations are chosen: slab and beam raft, multibarrel shell, folded-plate structure, folded-plate structure with voids filled with other than gravel and earth. The results obtained indicate that the depth of voltage attenuation is the same for all models regardless of the foundation design, and is determined by its dimension and the general level of stress and strain module of a base [1].

Using shells of different types as foundations arose considerable interest all over the world [4, 5]. Their higher load bearing capacity and smaller subsidence compared to solid slab foundation are proved. Various researches showed that shell foundations were more effective when it was necessary to transfer significant loads to weak ground bases with uneven deformations of the base, and for structures exposed to a large wind load, such as smoke pipes, silos, and power transmission towers. Operational analysis of different shell foundations, introduction of the newest construction solutions, and development of calculation methods of such foundations are carried out by foreign scientists of the industry [4 – 6].

Today there is a practice of using folded-plate shell foundations in different countries of the world – in the USA, China, Mexico, France, etc. Such foundations were used in the construction of both ordinary 4 – 5 storey residential buildings and high-rise civil buildings.

The shell foundation was developed in the Tyumen State Architectural and Construction Academy. It represents a reinforced concrete thin-walled shell laid on a ground base. The shell is of zero or positive Gaussian curvature with a cross-beam system; the reinforcement of the shell can be made of single-layer steel or synthetics [2].

The usage of folded-plate shell foundations on water-saturated soils and marshes is of particular interest, especially when arranging foundations for power transmission towers. In some cases these foundations are used on weak soils, this indicates an improvement of foundation work. If the soils at the base are characterized by high water absorption and compressibility, this leads to permanent settling of the earth's surface. In this case it is appropriate to use a floating foundation as a folded-plate shell. An example of such use is the construction of the USA embassy in Mexico City [7, 8]. There are also alternatives floating foundation for power transmissions towers; it consists of 12 floats (metal cylinders with the capacity of 6 m^3 each) joined by small frames into three groups of four floats. A large metal girder rests on groups of floaters, the tower body is installed in the middle of the girder, and the brace is attached to each of the three ends of the girder [8]. Less time-consuming and simpler is the foundation which consists of overturned trough-shaped reinforced concrete slabs, connected on top by two metal girders. Additional grip of the foundation with peat is achieved by "suction" of the trough-like elements; therefore, this relatively light foundation can withstand a significant overthrow. During the installation such foundations gradually descend under load peat compacting, however, after reaching a certain value, further subsidence stops. As brief experience shows, the foundations of such design are the most economical having peat of 5 or more metres [9].

Folded-plate shell foundations were used for the construction of 220 kV power transmission towers in the Middle Urals and Tyumen region [7, 8]. The foundations were built on marshes of 5 – 6 meters depth. Reinforced concrete folded foundations were constructed as separate thin-walled folds connected on top by steel or concrete beam or girder. Power transmission towers of about 40 meters height were put on foundations. A great economic effect was obtained during the construction. The construction cost was reduced by 37% compared with the traditional arrangement of foundations. Inadequate work of folds at uneven deformations of base under foundation can be considered as a shortcoming of this foundation structure.

Parts of general problem unsolved before. One of the main unsolved issues in the design of folded-plate foundations for the power transmission towers is foundation structure and towers project stability during their operation. With uneven deformations of the base, using folded-plate foundations on weak or water-saturated soils it is important to evenly redistribute the load from the foundation to the ground base.

Problem statement. The aim of the research is to expand the scope of folded-plate foundation in difficult geotechnical conditions and also to improve the operation of folded-plate foundation for transmission towers by using a hinged system of folds fastening with supporting beams, which enables the load and emerging forces to be redistributed evenly in every fold of the foundation. Thus, it ensures high efficiency of each element of folded-plate foundation and the system of folds in general.

Main part and results. The study was conducted and it was proposed an alternative design solution of the foundation for the power transmission towers [10, 11]. However, while studying work of the foundation and interaction with the base, the disadvantages of its structural parts were revealed. Therefore, considering all the disadvantages of the previous design, we have developed a new improved construction solution of the folded-plate foundation for power transmission towers.

Folded-plate foundation for transmission towers consists of thin-walled reinforced folds which are interconnected on top by steel or reinforced concrete beams. The first two folds are connected with two separate beams, while the third one is connected with them with the same beam on the hinged joints forming a system of bearing beams with fixed hinges. A system of folded-plate foundation consists of six prismatic folds, that is why other three folds are interconnected symmetrically and in a similar manner to the first three folds (Fig. 1).

An element of the fold has three horizontal plates and two sloping planes in cross-section. The upper horizontal fold is used to support steel or reinforced concrete beam. Two lower plates are required for fold installation during assembly and for load distribution, and also as bearing elements for the folds. Width of the upper plate is determined based on the calculation of concrete bending and on design reasons [10].

For the structural elements of folded-plate foundation to work jointly, all six folds are interconnected due to the combination of three separate folds through a system of fixed joints and bearing beams similar to other three folds.

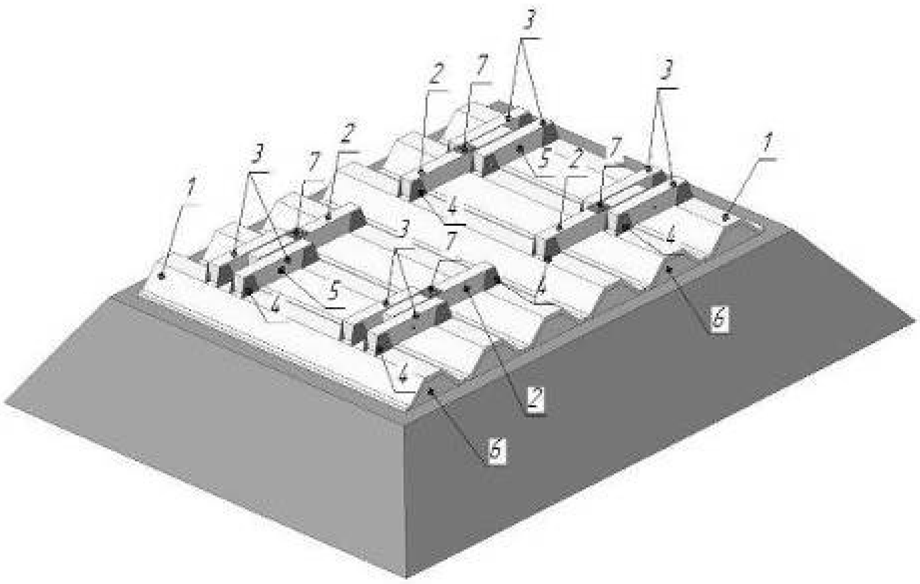

Figure 1 – Foundations for power transmission towers:
1 – thin prismatic reinforced concrete fold; 2 – bearing reinforced concrete beam;
3 – auxiliary reinforced concrete beam; 4 – metal hinge joints;
5 – bolt connection of the bearing beam and auxiliary beams;
6 – vertical reinforced concrete diaphragms;
7 – embedded plate with anchor bolts for fixing the base of the power transmission towers

The proposed foundation consists of six separate reinforced concrete thin-walled folds 1; three of them are arranged symmetrically to other three folds in respect to power transmission tower axis. Two folds on the edges are joined with two auxiliary reinforced concrete beams 3 on the top by metal hinge joint 4. The third prismatic fold is joined with the first two through the bearing reinforced concrete beam 2, on one side the bearing reinforced concrete beam 2 is connected to the fold 1 by the metal hinge joint 4, on the other side the bearing beam 2 is fixed directly to the auxiliary reinforced concrete beams 3 by the bolted connection. In the places of beams 2 and 3 bearing on the folds 1 vertical reinforced concrete diaphragms 6 are arranged. Vertical reinforced concrete diaphragms 6 are also installed in the center of the folds and at the edges. This type of connection is performed on both edges of folds 1 longitudinally. The other three folds 1 are joined similarly.

Presented foundation design is implemented through the joint work of structural elements of folded-plate foundation. More even distribution of external loads to the elements of the foundation system is achieved due to hinged joint of prismatic folds with bearing beams. Voids of the folds are of prismatic shape to provide the formation of a compact core of a certain value and to redistribute base pressure on the foundation.

According to the proposed design, prismatic folds 1 are joined with beams 2 and 3 on both edges longitudinally. Bearing beams 2 and 3 are installed in places designed for supporting the slabs of metal power transmission towers. In addition, folds 1 are designed with transverse diaphragms 6 located in spots of beams contacting 2 with 1 folded-plates, in the middle of the fold and across its edges. Diaphragms 6 are used for fixing fold 1 to the beam 2 with bolts installed in diaphragm. These diaphragms are supporting and designed considering shear forces in fold on a support.

Bolt connections 5, and bolts of the hinged element 4 are selected based on shear calculation to prevent from the action of all external loads.

The lower horizontal plates of prismatic fold 1, which are the end beams, reduce horizontal and vertical deformation of the fold’s edges [7].

The dimensions of each structural part of the foundation are selected individually for each power transmission tower; the main geometric parameter in selecting the geometry of the entire foundation is the base of the tower, while the geometry of prismatic folds, bearing and auxiliary beams should be constant. This relates to the dimensions ratio of the supporting parts of the fold, inclination angle of the sloping planes of the fold (it should be within $30 – 40\ ^{o}$), the ratio of height and width of the cross section of bearing and auxiliary beams.

Significant loss of foundation weight is reached due to application of shells which work as spatial structures having a curved shape and a large bearing capacity with minimal thickness. The shells are also characterized as having compressive and stretching tensions at relatively small bending moments [12].

The purpose of this construction is to limit (prevent) absolute and (or) relative foundation and superstructure displacements to the extent required for proper operation and durability of a structure [12].

Uneven vertical displacements under power transmission towers, under base, provoke load growth, but contact pressure cannot exceed its limit; as a result there is an intensive fold's voids filling 1 with subsoil in these areas [13]. Whereas the power load is redistributed again: it is reduced in areas with high values and increased displacements of the fold's voids; accordingly it increases in areas with lower values and insignificant displacements of the fold's voids. Thus, process of contact pressure self-regulation of fold's voids is realized. All this allows: smoothing out uneven deformation of soil base; smoothing the peaks of stress concentration in substructures and reducing the stress value in superstructures eliminating foundation tilt. Eventually when the effect of uneven vertical displacements stops, all components of folded-plate foundation gain new stable state of static equilibrium. At repeated uneven vertical displacements under power transmission towers character of fold's voids work 1 is repeated according to a new scheme of load redistribution. The process of self-regulation is possible as long as there is free capacity in fold's voids [14].

Foundation stiffness impacts the distribution of contact pressure significantly. Therefore, alteration of shell's stiffness may result in qualitative change in the shape of contact pressure diagrams. The research results state that for the reinforced concrete thin-walled shell foundations distribution of contact pressure can be taken as uniform in excess strength [12].

The improved design of foundation construction meets a number of requirements: the possibility to use weak structural capacity of the soil and water buoyancy force; minimum weight of prefabricated elements to be transported to remote zones; minimum materials consumption; uniformity of prefabricated elements; voids formation in the bottom to create osculum effect in case of foundation separation; creation of porous lower surface to increase foundation resistance to horizontal displacement.

According to the construction solution, under uneven base deformations the redistribution of contact pressures occurs and the system comes into equilibrium. The alternative construction design of folded prismatic foundation for transmission towers can evenly redistribute the loads to the structural elements of the foundation system.

Conclusions. The scientific novelty of the research is the folded-plate foundation new improved design solution development for the power transmission towers with the specific hinged system use of fastening bearing structural elements, and the formation of a special load calculation scheme. The further investigation presupposes the development of methods for calculating this type of foundation. Analysis of shell foundations showed that these foundations are in many cases more effective not only in terms of interaction conditions with the base, but also in economic aspects, as the material costs during the construction of such foundations are much lower compared to other foundations. The proposed folded-plate foundation can be used for transmission towers which are applied in water-saturated, marshy and weak soils and at uneven base deformation.

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CONSTRUCTIVE SOLUTION OF A TUNNEL UNDER THE EXISTING HIGHWAY

Urban underground structures (shallow tunnels), which are usually built in conditions of urban area with heavy traffic, are considered. It was clarified that the reinforced concrete lining are performed of concrete, reinforced concrete, natural and artificial stones in most cases. The types of tunnels road cutting are given. The geometrical dimensions and characteristics of using materials in the construction of tunnels are given. It was established that the existing methods of tunnel construction do not allow to solve the problem in cramped conditions and intense traffic. The new constructive solution of the transverse tunnel under the existing highway was proposed. It is found that the proposed design of the transverse tunnel may be performed in cramped urban environments with the use of modern technological equipment and the proposed method of tunnel meets the requirements of strength, reliability, durability.

***Keywords:** cross-tunnel, tunnel lining, installation of horizontal directional drilling.*

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КОНСТРУКТИВНЕ РІШЕННЯ ТУНЕЛЮ ПІД ДІЮЧОЮ ТРАНСПОРТНОЮ МАГІСТРАЛЛЮ

Розглянуто міські підземні споруди (транспортні тунелі дрібного закладення), які споруджують звичайно в умовах забудованої міської території при інтенсивному вуличному русі. З'ясовано, що монолітні оброблення виконуються в більшості випадків з бетону, залізобетону, природних і штучних каменів. Наведено типи оброблень автодорожніх тунелів. Надано геометричні розміри та характеристики використаних матеріалів при споруді тунелів. Установлено, що існуючі методи спорудження тунелів не дозволяють розв'язати завдання в стиснутих умовах і при інтенсивному транспортному русі. Запропоновано нове конструктивне рішення поперечного тунелю під діючою транспортною магістраллю. З'ясовано, що запропонована конструкція поперечного тунелю може бути виконана в стиснутих міських умовах із застосуванням сучасного технологічного устаткування, а запропонований спосіб прокладання тунелю задовольняє вимоги міцності, надійності, довговічності.

***Ключові слова:** поперечний тунель, тунельні оброблення, установка горизонтально направленого буріння.*

Introduction. World practice of urban planning shows that one of the most effective ways of territorial, transport, environmental and energy issues decision is comprehensive underground space development, which can accommodate structures of different purposes [1, 2].

In recent decades, the growth in the volume and extent of underground construction is observed in large cities of Ukraine. Large underground complexes for various purposes, transportation and communication tunnels, underground parking lots and garages, manufacturing and warehouse facilities are built, the length of subway lines is increased.

Review of the latest research sources and publications. The development of large cities and the continuous growth of traffic demand improvement of the urban transport system. To ensure the rational organization of traffic and improve urban transport linkages there are provided dof facilities of transport crossings in different levels on the busiest routes and transport hubs, construction of subways lines, underground parking lots and garages. In the general complex of urban underground structures traffic and pedestrian tunnels of shallow are occupied an important place, that are pitched usually in terms of a built up urban area with heavy traffic. In the construction of these tunnels it is necessary to ensure the safety of buildings and structures on the route of the tunnel and continuous passage of vehicles on highways, which intersection is constructed in tunnel. Urban shallow tunnels are erected mainly by the open method with the reverse surface. In this work they are consistent in certain areas, the length of which is prescribed under minimal compression of traffic [3 – 8].

Definition of unsolved aspects of the problem. One of the main issues is the design of the transverse tunnel, which can be performed in cramped urban environments with the use of modern technological equipment.

Problem statement. The task of the research is development of transverse tunnel new constructive solutions with the use of devices atroscine technology for large and industrial cities with high density and intense traffic.

Basic material and results. Underground structures were known in ancient times. They were first for habitation and burials, in the construction of temples, and then for the extraction of stone and ore, and later for water supply purposes and irrigation. Then transport, communications and other types of tunnels came [9].

Design of tunnel cutting can be applied as prefabricated from reinforced concrete, concrete or metal elements and monolithic concrete or reinforced concrete. At this timein prolect of technical conditions it is noted that the main materials of tunnel cutting construction should be precast concrete and reinforced concrete. The internal design of stations and other tunnel structures, as a rule, is provised of precast concrete.

Concrete and reinforced concrete tunnel structures shall be calculated by the method of limit states (bearing capacity, deformation, the formation and opening of cracks), iron and steel structures can be calculated and allowing for tension.

Calculation of tunnel constructions is conducted on the basic, additional and special combinations of loads, which are taken in the most unfavorable for the individual elements and the entire structure combinations.

Regulatory pressure on the lining depends on the specific geological and construction conditions, tunnel construction: considering work of the breed and unloading of the arch, or on the total weight visaliaca rocks. The horizontal confining pressure is taken depending on physical-mechanical properties of rocks, but not more than 70% of the vertical rock pressure. If the rocks are free (unbound) water is also considered hydrostatic pressure.

The tunnel lining is built by closed method, and calculated considering elastic resistance of rock in the tray and the side parts of the loop processing. When the founding of tunnels is below ground water level, tunnel stability against the ascent is tested.

Monolithic lining are made of concrete in most cases, and of reinforced concrete, natural and artificial stones.

The concrete class is assigned to the B15, B25 for monolithic concrete cutting and B15-B30 is for precast concrete cutting. Masonry cutting of natural or artificial stone is made from cement mortar grade not lower than 50 for structures with exterior waterproofing and not lower than 100 or higher for structures without exterior insulation.

Processing of mining tunnels is satisfied primarily of reinforced concrete or reinforced concrete. The thickness of cutting should be at least 20 cm.

In the construction of mountain tunnels in the rock fractured rocks, renewed ordinary reinforced concrete is used in cutting with sprayed concrete which is deposited on the surface of the generation layer of 5 – 20 cm under the pressure of a 4 – 4.5 ATM. Using this type of concrete is got the reducing of cement consumption, increasing the density, water resistance. It greatly simplifies the process of concreting in absence of formwork.

To increase the bearing capacity of such a lining sprayed concrete, it is often reinforced with steel meshes.

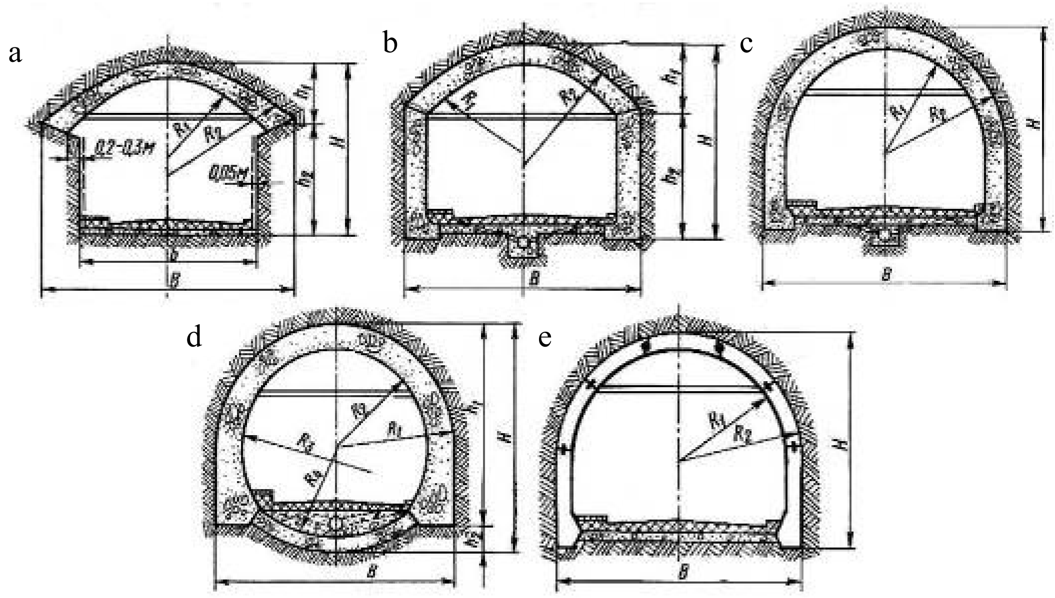

Figure 1 – Types of cutting in road tunnels:
a – cutting of normal cast in place concrete or sprayed concrete;
b – treatment consists of a vault based on a straight wall;
c – processing is in the form of not closed structures without a back arch;
d – treatment with massive walls of curved inner shape and a reverse arch;
e – collective processing in the form of a vault based on breed or monolithic walls

The construction of cutting mining tunnels of reinforced concrete is advisable primarily in complex engineering-geological conditions with increased loads on the structure and significant inflows of groundwater, when the device of massive concrete cutting becomes uneconomical. However, the construction process of reinforced concrete cutting is a difficult installation of rebar, pouring and compaction of concrete mix.

There are different types of monolithic tunnel cutting arched shape [10, 11].

In strong rocks which do not make lateral pressure on the mount, it is applied processing from conventional solid concrete or sprayed concrete in the form of a set of constant or variable stiffness, based on the species (Fig. 1, a). For greater stability the vault is suited the ledges of rocks berm – width of 0.2 – 0.3 m. Wall production can be vertical or with a slight slope and lined with the layer of shotcrete thickness of 5 cm. The ratio of arch height span should not be more than four, because five of shallow arch can obtain the horizontal offset, which will lead to sharp increase in bending moments in lock section.

In the less strong and fissured rocks it is necessary to arrange cutting not only ceiling but walls of the tunnel. With a slight side pressure, walls are suited of straight. If erection produces production parts, the processing consists of the arch, which rests on the rectilinear walls (Fig. 1, b and 1, a). The excavation of the tunnel on the full profile is in the form of open loop construction with no arch (Fig. 1, c 1, b). The roadway in such tunnels is laid directly on rock or concrete preparation.

In weak rocks, that exhibit significant vertical lateral pressure and pressure from below, the processing must have a massive wall to a curved internal shape and a reverse arch (Fig. 1, d). Walls need a bit of deepen in the breed for perception of lateral pressure.

To protect the tunnel against the penetration of groundwater a waterproofing treatment is arranged, and sometimes drainage of the surrounding mountain range is. In addition to waterproofing, to protect the tunnel from water injection, cement processing is used, which fills all voids and cracks that is the source of the leaks.

To drain the mountain range in some cases surface drainage is used, stulnev, gravity and sporovo drains and grouting curtain.

The disclosure of excavations in hard rock on the full profile makes possible the use of prefabricated cutting from pre-fabricated elements of concrete or concrete blocks solid or ribbed cross section.

Collective processing can be arranged in the form of a vault based on breed or monolithic walls and open crypt construction for the entire height of tunnel cross section (Fig. 1, e). To support the precast treatment on the species there should be provided special support units with an extended fifth.

When using prefabricated cutting, the arched shape achieves a high quality of design and reduces the consumption of concrete, however, deteriorates the water-resistant treatment due to the presence of seams between the blocks and the need to fill extra processed space.

Prefabricated MULTI PLATE constructions are well-provided [12].

Design of MULTI PLATE can be used to create pipes, auto and railway tunnels, pedestrian tunnels, tunnels for the distillation of cattle, communication reservoirs, hangars and warehouses, protective galleries, and so on.

Constructions are used as a molded shell, and as a self-supporting structure during the reconstruction of the bridges and water ducts.

All tunnel construction can be equipped with straight or sloping sides and inset light fittings. It should be selected the type and size of the structure depends on the dominant height, loads, and design capacity of the water flow or the movement. The thickness of the metal plates varies from 2.75 to 7.00 mm. Plate between OSA is screwed by M20 (SB8,8). For all designs it is possible to manufacture special parts, holes and the nozzle almost without any restrictions on the form.

All design solutions are considered in the analysis which do not help to resolve the problem in cramped conditions and intense traffic.

The distinctive feature of the new constructive solutions of the transverse tunnel is the placement of steel pipe in the workings with the subsequent concreting, while ensuring the continuity of traffic on the street network. The ridge array 1 laid a highway 2. There is made in the array 1 cross tunnel 3. It contains: wall 4 and ceiling 6 through holes (openings) and is filled by pipes 5 with their subsequent concreting, the bottom 7, a cross frame 8. Tunnel design is done using horizontal directional drilling (HDD) 9 (HDD Vermeer Navigator D24×40) (Fig. 2, a – d).

First there is formed skeleton of the tunnel by drilling of production installation HDD Vermeer Navigator D24×40. Before starting of work, the properties and composition of soil, location of existing underground utilities are carefully studied. There is sensing of soils and, if necessary, surfline particularly complex intersections route of drilling with existing communications. The results of these works have a certain significance for the choice of trajectory and tactics of the construction workings. Particular importance should be given to the optimal location of drilling equipment on the construction site and to ensure safe working conditions of the drilling crew and other people.

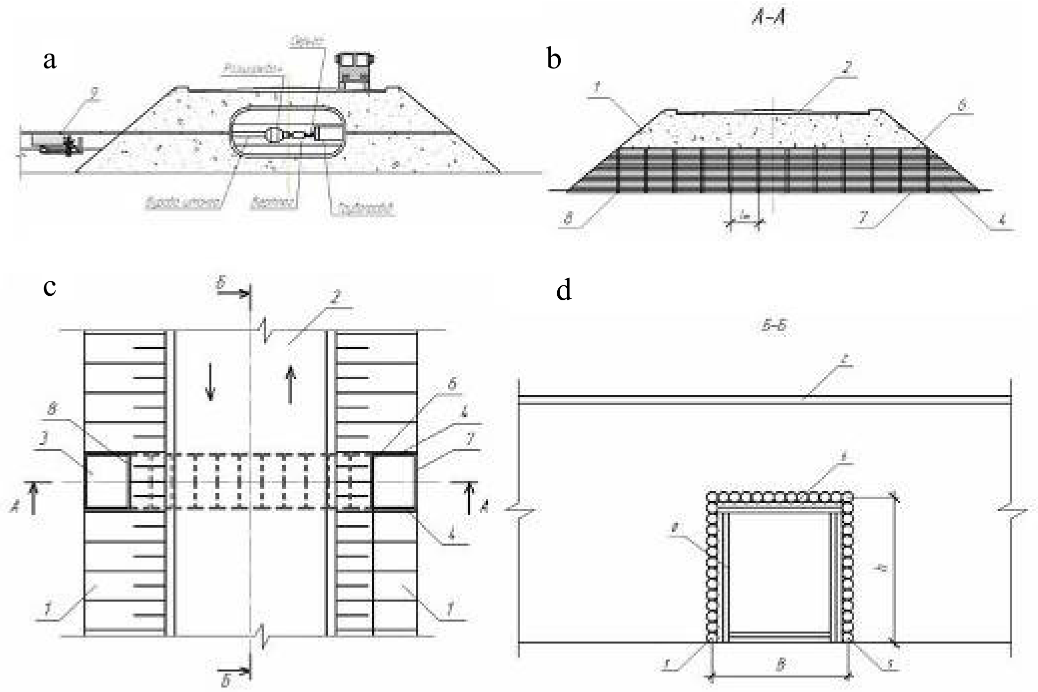

Figure 2 – the constructive solution of the transverse tunnel:
a – pulling of metal pipes; b – section A-A;
c – mound that is on the crest of the highway, with a transverse tunnel in terms ;
d – section B-B;
1 – array; 2 – transport highway; 3 – ross tunnel; 4 – walls; 5 – pipes;
6 – overlapping; 7 – the bottome; 8 – transverse frames; 9 – installing HDD

The construction by the technology of horizontal drilling is carried out in three stages: drilling of a pilot production, scaling up production and extending pipes.

Drilling of the pilot production is carried out using rock cutting tool – a boring head. The latter, in turn, is connected by way of gentle shell with flexible drive rod that allows you to manage the process of building pilot production and it is detected at the stage of preparation for the drilling of underground obstacles in any direction of horizontal directional drilling method within the natural bending prothomalo working thread. Construction of pilot production is completed with the output of the drill head at a given point.

Expansion of production is carried out after the completion of the pilot drilling. In this case the drill head is disconnected from the drilling rods and instead joins extender reverse action. App traction with simultaneous rotation of the expander is pulled through the target generation in the direction of the drill, expanding required pilot production for pulling the pipe diameter. To ensure smooth pulling of the pipe through the expanded production of its diameter is 20 – 30% larger than the diameter of the pipe.

Opposite the rig side of the production is ready to stretch the pipe. To the front end of the pipe to the head is attached , which senses the traction of the hinge and the extender. There is used steel pipe with an outer protective coating polyurethane (PPU). This coating is a structure consisting of steel pipe, insulation layer of polyurethane foam (gas-filled plastics or foams generated in the reaction process and the formation of air-filled capsules two components: socionatu and pololu), and also pipe-shell made of low pressure polyethylene (HDPE). This protective coating of pipes PPU today is the most effective and perfect way, because it combines the insulation of polyurethane foam layers, which adds waterproofing HDPE pipe shell.

After drilling of excavations and laying of pipes each pipe is filled with concrete.

Then digging parallel conducting installation of the metal cross frames starts. After installing the RAM and seizure the whole ground out concreting of the bottom is carried.

There is used steel pipe with a diameter of 300-500 mm with an outer protective coating of polyurethane foam. Today this protective coating of pipes is the most effective and the best form, as a combination of insulating and waterproofing properties of the pipe shell.

The calculation of loads should be performed in the following condition:

$$N_p \le N_{кр}, \tag{1}$$

where Np – the calculated load, kN.

$$N_p = \varphi \cdot q_p \cdot \frac{l_{cò} \cdot b}{2} \cdot \gamma_n, \tag{2}$$

where φ – the coefficient of buckling of a compressed strut;

qp – calculated line load, kN;

l_{cm} – the distance between the racks, m;

b – the width of tunnel, m

γ_n – the reliability coefficient for the appointment;

$N_{кр}$ – the critical load, kN,

$$N_{êp} = \frac{\pi \cdot Å \cdot \mathrm{I}}{h_0^2}, \tag{3}$$

where E – the modulus of elasticity, MPa;

I – the moment of intersection stand inertia, m^4;

h_0 – the estimated length of the rack, m.

The distance between racks is determined by the formula:

$$l_{cm} = \frac{2 \cdot \pi \cdot EI}{\varphi \cdot q_p \cdot b \cdot h_0^2 \cdot \gamma_n}. \tag{4}$$

After installing the RAM and excavating all soil, there is produced bottom concreting 7. Rigid characteristics of a structure are governed by the pipe diameter and class of concrete.

The proposed method of tunnel meets the requirements of strength, reliability, durability.

The problem is solved due to the fact that the design of the tunnel with large cross sections include the excavation contour of tunnel faster with laying pipes and subsequent concreting. The following technological operations are associated with the device support in the leading mines, the connection support between the supporting edges, the recess of the soil core and the implementation of tunnel lining.

Conclusions. The proposed design of the transverse tunnel may be performed in cramped urban environments with modern technological equipment use. The estimated parameters allow to project design in compliance with the requirements of strength and reliability.

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EXPERIMENTAL RESEARCH TECHNIQUE OF RETAINING WALLS OF A SPECIAL TYPE

The article reviewed the issue of wide use of the retaining walls in construction. It is established the existing retaining walls are not designed for additional forces from horizontal soil displacement that consequently leads to the destruction of the structure. In this regard, there is a need for the development of new structural solutions for retaining walls. The purpose of the research is to develop the technique for conducting experimental studies of the contact interaction of retaining walls and a deformable base. The experiments were carried out on small-scale models in a specially designed tray. At modeling was applied the method of the expanded similarity in which geometrical, mechanical and power analogues with a real object are maintained. As base soil, the models used loamy structure. Models of retaining walls were made on a digital 3D model. A technique for conducting experimental studies of the contact interaction of retaining walls and a deformable base has been developed. The technique is universal and will allow carrying out model experiments under equal conditions, which will ensure reliable results.

***Keywords:** technique, experiment, model, foundation, retaining wall.*

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МЕТОДИКА ЕКСПЕРИМЕНТАЛЬНОГО ДОСЛІДЖЕННЯ ПІДПІРНИХ СТІН СПЕЦІАЛЬНОГО ТИПУ

Висвітлено питання широкого застосування підпірних стін у будівництві. Встановлено, що існуючі конструкції підпірних стін не розраховано на додаткові зусилля від горизонтального зрушення ґрунту, що призводить до руйнування конструкції. Доведено необхідність розробки нових конструктивних рішень підпірних стін. Розроблено методику проведення експериментальних досліджень контактної взаємодії підпірних стін і деформованої основи. Проведено експерименти на маломасштабних моделях у спеціально сконструйованому лотку. При моделюванні застосовано метод розширеної подібності, в якому витримуються геометричні, механічні та силові аналоги з реальним об'єктом. Як ґрунт основи в моделях використано суглинок порушеної структури. Моделі підпірних стін виготовлено за цифровою 3D-моделлю. Доведено, що ця методика дозволяє проводити модельні експерименти в рівних умовах, що забезпечить отримання достовірних результатів.

***Ключові слова:** методика, експеримент, модель, основа, підпірна стінка*

Introduction. The retaining walls are now widely used not only in civil and industrial construction, but also in town planning for complex landscapes.

There are situations of objects construction in cramped conditions, in unfavorable areas with active deformation effects, which complicates the use of existing types of retaining wall structures.

According to expert estimates, 90% of the territory of Ukraine is characterized by complex engineering and geological conditions, deteriorating due to the impact of natural and man-made factors [1].

Analysis of recent studies and publications sources. Currently, this problem is relevant for work areas and subsidence grounds, because with complex deformations of the base, it is not always possible to realize the available technical solutions in view of their inability to work conditions. Existing constructions of retaining walls are not designed for additional forces from horizontal soil displacement, which causes stress concentration in the lower part of the faceplate, which consequently leads to the destruction of the structure [2, 3].

Numerous studies of the behavior 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 rigidity characteristics of the base, which can cause uneven subsidence [4, 5].

Allocation of previously unresolved parts of a common problem. Designing optimal design solutions, taking into account specific operating conditions, is one of the main engineering tasks. For retaining walls used in work areas with horizontal and vertical soil movements, this task is particularly important. Experimental studies have shown that the stress-strain state of the substrate is largely determined by the design characteristics of the operating and loading conditions [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 research is to develop a methodology for conducting experimental studies of the contact interaction of retaining walls and a deformable base.

Main part and results. Classification of the experiment:

– by structure – a model experiment (a retaining wall model, a base model);

– at the stage of scientific research – bench tests (obtaining information about the SSS of the retaining wall);

– organization of the experiment – conducting an experiment in the laboratory;

– by the method of conducting – an active experiment (with the possibility of active influence on the object).

The purpose of the tray studies is to determine the optimal design parameters of the proposed design of the retaining wall of a special type, as well as to identify the qualitative patterns of its joint work with the base.

The experiments were carried out on small-scale models in a specially designed tray. At modeling the method of the expanded similarity in which geometrical, mechanical and power analogues with a real object [6, 7].

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. Taking into account the necessary moisture content of the soil, its density and volume determined the necessary amount of powder and water for its moistening. Humidification was carried out by a nebulizer with a constant stirring of the mixture. The base of the paste 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.

We simulated loams with the following characteristics (E = 13,5 MPa, c = 19,5 кPa, γ = 1,82 t/m3, φ =22°).

The physical and mechanical properties of the base model were determined using a field laboratory FLL-9 (ПЛЛ-9) (fig. 1 a, b) in accordance with the methodology [8]. The strain modulus was determined with the help of a compression device of the Litvinov system. The cohesion coefficient of the soil and the angle of internal friction were determined by means of a shearing device P10-S (П10-С). Sampling was carried out from a tray with a pitch of 150 mm in height, the results of certain characteristics are presented in (tab. 1), the comparative characteristics of soils are presented in (tab. 2).

a b

Figure 1 – Preparation of the device:
a – FLL-9 (ПЛЛ-9); b – characterization of model soil

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	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

Design features of a retaining wall of a special type: a monolithic retaining wall of an angular type that has voids on the contact surface of the vertical and foundation elements, in the form of truncated pyramids of the same size and directed in a smaller base in depth [9, 10]. With the development of the deforming load in time, that is, with vertical and horizontal movements of the soil with respect to the monolithic wall of the angular type, after its installation, gradual penetration of the soil into voids occurs. 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: a thickness of 200 μm, a density of 916 kg / m^3, a 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 a digital 3D model (fig. 2 a, b). For this, a 3D printer was used Graber i3.

a b

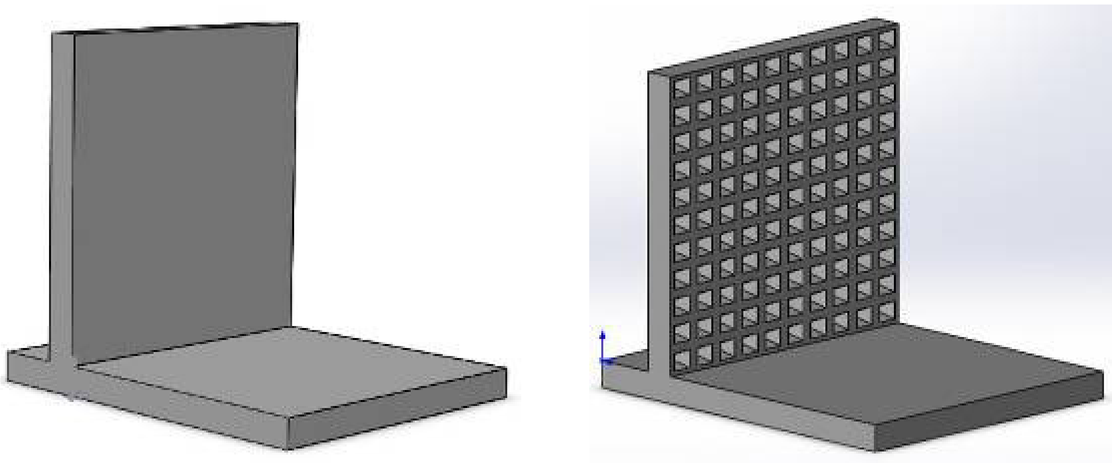

Figure 2 – Experimental models:
a – angled retaining wall; b – retaining wall of a special type

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 strip 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 friction of the soil against the wall of the tray, the inner part of the walls was covered with an easily deformable polyethylene film in two layers with a layer of technical petroleum jelly.

The purpose of the first series of tests was to identify the degree of structural factors influence on the bearing capacity of the model of the proposed retaining wall of a special type.

The second series of tests were conducted to compare the stability of the anti-shear position of the retaining wall of the corner type and a retaining wall of a special type.

Preparation of the tray for testing was carried out by installing it on the supports, after which 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.

Preparation of the base model 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 surface of the ground was planned, after which a retaining wall was installed. Further, the layered laying of the soil paste 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, we used hour-type indicators ICh-10 (ИЧ-10), 6-PAO (6-ПАО) programmers, which were verified in the center of metrology, standardization and certification. Before the free surface of the retaining wall, a bar with two clock-type indicators was rigidly mounted to measure horizontal deformations (displacements) of the wall in two levels. Over the retaining wall, on a specially prepared console, there were installed deflectors to measure 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 the conditioned stabilization of the soil. The sedimentation rate of the model that does not exceed 0.1 mm in 30 min was taken as a criterion for the conditional stabilization of deformation. Each subsequent stage of pressures was also maintained during the time of conditional stabilization (fig. 3 a, b).

Loading models were carried out until the full loss of stability of the retaining wall. The values of the devices were recorded and recorded in the log, after which the graphs were constructed.

At the end of each experiment, the soil from the tray was removed, dried in a drying cabinet and ground. In order to carry out the next experiment, again, according to the foregoing technology, the ground paste was prepared and re-stacked in a tray.

The conducted studies showed that with the same ground base (the geometry of the layers and the physical and mechanical characteristics), the load and the boundary conditions evident for the retaining wall of a special type is the inclusion in the work of the entire soil massif and the uniform redistribution of stresses at the contact along the face and base slabs; uniformity of general deformations of structures and soil base, which, in turn, provide greater stability of the retaining wall of a special type.

a b

Figure 3 – Testing:
a – pad loading; b – readings from instruments

Conclusions. A technique for carrying out experimental studies of the contact interaction of retaining walls and a deformable base has been developed. This technique is universal and will allow carrying out model experiments under equal conditions, which will ensure reliable results.

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MOISTURE CONDITIONS PATTERNS IN ROAD EMBANKMENT CLAY SOILS DEPTH

The article deals with optimal compaction criteria of road embankment soils improving, which provide their long-term strength. Physical experiment methodology for patterns establishment of water migration in subgrade embankment depth, in the capacity factors of what it is accepted: clay soil type (its number plasticity); moisture, at what the soil was compacted; soil skeleton density; embankment height; «rest» time after subgrade erection and before it's operation are developed and realized. By laboratory and field tests water migration patterns in compacted subgrade soils depth are established. As a result of statistical processing of laboratory and field research results, the empirical dependence of compacted clay soil stabilized moisture for their multilayer consolidation in relation to soil skeleton density and plasticity number values is obtained.

Keywords: *road embankment clay soils, long-term strength, water migration, loam multilayer consolidation, maximum soil skeleton density, maximum molecular moisture capacity (maximum quantity of unfree water).*

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ЗАКОНОМІРНОСТІ ВОЛОГІСНОГО РЕЖИМУ В ТОВЩІ ГЛИНИСТИХ ҐРУНТІВ ДОРОЖНІХ НАСИПІВ

Удосконалено оптимальні критерії ущільнення глинистих ґрунтів дорожнього насипу, за якими забезпечується їх тривала міцність. Розроблено та реалізовано методику фізичного експерименту зі встановлення закономірностей міграції води в товщі земляного полотна, фактори котрого: вид глинистого ґрунту (його число пластичності); вологість, при якій його ущільнювали; щільність скелету ґрунту в насипу; висота насипу; час «відпочинку» після зведення і до початку експлуатації земляного полотна. Шляхом лабораторних і польових дослідів установлено закономірності міграції води в товщі ущільнених ґрунтів земляного полотна.

Ключові слова: *глинистий ґрунт дорожнього насипу, тривала міцність, міграція води, пошарово ущільнені суглинки, максимальна щільність скелету ґрунту, максимальна молекулярна вологоємність (максимальна кількість зв'язаної води).*

Introduction. When soil compacts due to high moisture, with a degree of saturation close to S_r = 1,0, soil dries and holds its sedimentation and therefore additional deformations. When soil compacts due to low moisture, it will be difficult to do the compaction, there is a little probability of desired soil skeleton density achieving, even with the modern mechanisms possibilities. Also national standards recommend to take plastic limit W_p,for optimum clay soils moisture content, at the compaction by roller, but this parameter is not related on how much unfree water the soil is actually containing [1 – 5]. That is why for subgrade erection it is important to have long-term strength ensuring, i. e., when during normative operational time the values of soil mechanical characteristics, obtained after compaction, have been saved and excess soils deformation does not appear.

Recent sources analysis of research and publications. Now both in Ukraine and in the world at the highway embankments erection it is normalized soil skeleton density, determined for each type of soil in the laboratory by Proctor test or its modification [5, 6]. However, the problem is that domestic regulations prescribe optimal parameters of compacted clay soil (maximum soil skeleton density ρ_{dmax} and optimum moisture content W_{opt}), based on the obtained values of laboratory conditions for a particular soil type and dynamic load characteristics without actual mechanisms characteristics [6 – 9].

The disadvantages of the soil subgrade compaction methods are: the necessity to «bind» the soil compaction curve to specific compaction mechanism certain parameters; sufficiently wide boundaries of optimum soil moisture; some subjectivity in soil plastic limit determining etc. So, the most common today concept in road soil compaction construction solves is mainly the technological side of problem – the maximum soil skeleton density at fewest mechanism passage according to [6, 8].

Identification of general problem parts unsolved before. For the reliable subgrade operation it is necessary not only to achieve maximum multilayer consolidation values of its soil skeleton density and soil strength, but also to save them during normative time. On the embankment soils condition over time moisture affects significantly where the compaction was done and the particular water type split in compacted soil [10 – 13]. But the next factors as for quantitative impact on moisture conditions patterns in road embankment clay soils depth still have not been researched: soil skeleton density; compacted embankment height; the number of days compacted clay embankment «rests» after its erection, and before the operation [10, 12].

In soil mechanics, minimum stress where soil sample is destroyed due to aeonic span of time, accepted as long-term strength limit. Stress where the soil sample is destroyed due to certain period after load imposing as a result of unchangeable soil flow and advance flow of ground, corresponds to the soil long-term strength. So the theory considers the most favorable conditions for the subgrade clay soils long-term strength ensuring and minimum ground distortion during specified time operation is to compact the soil in layers at moisture, close to maximum molecular moisture capacity [8, 10 – 13].

Basic material and results. For new optimal compaction criteria of road embankment soils substantiation where subgrade clay soil long-term strength is ensured, the new physical experiment methodology for water migration patterns establishment in subgrade embankment depth is already developed and described in the works [10 – 13].

As a result of statistical processing by least squares method of research data it is found that the decreasing of stabilized moisture value w_k of compacted heavy silty loam (research soil № 1) depending on the soil skeleton density growth within the experimental range ρ_d = 1,50 – 1,65 g/cm^3 it is most correctly to describe by logarithmic function of the form [13]

$$w_k = a + b\ln\left(\frac{\rho_d}{\rho_{d0}}\right) , \quad (1)$$

where is ρ_{d0} = 1 g/cm^3;

empirical coefficient is: a = 0,358; b = - 0,384.

At this multiple correlation coefficient r and variation coefficient v values is: r = 0,997; v = 0,008, that indicates close relationship between the experimental data and about the correctness of their approximation by the logarithmic function [10, 13].

Analogous logarithmic dependence is obtained also for stabilized moisture value w_k of compacted light silty loam (research soil № 2). Empirical coefficient of equation (1) is a = 0,362; b = - 0,494.

At this multiple correlation coefficient r and variation coefficient v values is: r = 0,998; v = 0,0115, that indicates close relationship between the experimental data and about correctness of their approximation by the logarithmic function [13].

Comparing by data [11 – 13] of the final average soil moisture values w_k of compacted loams after two months «rest» with initial moisture values w of this soil it can be stated, that:

– final average soil moisture value w_k of compacted loams is compared with initial moisture w, where the clay soil was compacted and decreased for all soil skeleton density value ρ_d almost for all tube height except its upper link, where soil moisture approached to the value w_{sat}, that corresponds to degree of saturation $S_r \approx 1,0$ by raising capillary moisture. Soil moisture in lower tube link decreased to w = 0,10 – 0,12 and light silty loam moisture to w = 0,08 due to evaporation of free water;

– final moisture value w_k of compacted subgrade loams within experimental range ρ_d = 1,50 – 1,65 g/cm^3 decreases by dependence, close to logarithmic with soil skeleton value increasing, that is explained by the fact that with ρ_d increasing due to the fact of ρ_d increasing film thickness of unfree water decreases and besides, the coefficient of permeability also decreases, that reduces to moisture speed redistribution;

– stabilized soil moisture w_k in all cases is less than soil plasticity number W_p and approach to this maximum molecular moisture capacity *is* w_{mm});

– moisture decreasing of initial w where the soil was compacted, within highway embankment in practice causes its additional settlement.

The average values of compacted light silt loam stabilized moisture w_k determining dependence on pipe height at soil skeleton density value ρ_d = 1,55 g/cm^3 are:

at pipe height of 0,45 m – w_k = 0,138;

0,9 m – w_k = 0,141;

1,50 m – w_k = 0,129;

2,10 m – w_k = 0,129

and at 2,85 m – w_k = 0,133.

Moisture plots changes analysis of compacted loam height wise the pipe of 0,45 m, 0,9 m, 1,50 m, 2,10 m and 2,85 m shows, that the compacted clay soil height as a part of subgrade did not significantly affect its moisture condition.

From the moisture plots changes analysis of light loam height wise, the embankment (pipe) is compacted at moisture w = 0,231 to soil skeleton density ρ_d = 1,55 g/cm^3, after 74, 120 and 180 days of the «rest» in particular, it can be seen, that average stabilized moisture of compacted loam height wise the pipe, except its upper and lower links after 74 days, is w_k = 0,143, 120 – w_k = 0,134, and 180 – w_k = 0,131, so moisture increased just on 1,0% approximately.

Whereas road embankment height with compacted loam did not significantly affect on its moisture conditions and subgrade «rest» time increasing after 2 months did not significantly affect on the stabilized soil moisture value, it is advisable to perform two-factor statistical analysis of compacted clay soil stabilized moisture, depending on its soil skeleton density and plasticity index.

As a result of this statistical processing by least squares method the empirical dependence is obtained [12]

$$w_k = a_0 + a_1\left(\frac{\rho_d}{\rho_{d0}}\right) + a_2 \cdot I_p \ , \qquad (2)$$

empirical coefficient is: a_0 = 0,531; a_1 = -0,279; a_2 = 0,570.

At this multiple correlation coefficient is r = 0,995, and Fisher's ratio test F = 106,326, that is more than its table-valued F = 4,89 at test significance p = 5% and the degree of freedom v_1 = 7 and v_2 = 5.

Statistical values indicates about close relationship between the research data and therefore, about the logarithmic function (2) correctness.

The results of physical laboratory experiment related to quantitative patterns of water migration in compacted heavy and light silt loams (clay soils type – its plastic index I_p, soil skeleton density ρ_d, g/cm^3, stabilized (final) moisture of compacted clay soil w_k) are presented in Tab. 1.

The Tab. 1, in particular, clearly shows that an increase of its plasticity number I_p at the same soil skeleton density values ρ_d, stabilized moisture of compacted clay soil w_k increases.

Table 1 – Stabilized (final) moisture values of compacted heavy and light silty loams within pipe height for each preset subgrade soil skeleton density

Preset soil skeleton density, ρ_d, g/cm^3	Soil plasticity number, I_p	
	0,162	0,080
1,50	0,203 -0,95%	0,162 2,36%
1,55	0,190 -0,51%	0,143 -0,86%
1,60	0,176 -0,58%	0,130 -0,21%
1,65	0,167 2,35%	0,144 -2,04%

Note: numerator – the experimental values of stabilized clay soil moisture w_k; the denominator – the relative error of this parameter, calculated by the expression (2)

Also, two physical laboratory experiment series for possible water migration research in clay soils depth, compacted at stabilized moisture, were done.

In this regard the light loam was compacted at initial soil moisture, that corresponds to stabilized moisture value for this soil type (notably at plasticity number I_p = 0,08 the moisture was $w = w_k$ = 0,130) to soil skeleton density ρ_d = 1,60 g/cm^3 at pipe height of 150 cm.

The first test series methodology does not differ from earlier described [10 – 13] (the experiment lasted for 70 days), and in the second experiment series for checking the possible capillary ascension slot channel drain with stone screening dust was filled with water (Fig. 1).

Figure 1 – Chute, filled with water for possible capillary ascension research for road embankment

The lower pipe links were input in the chute, i. e., research soil had the opportunity to boggy action. Lower pipe link was located at distance of not more than 2 – 3 cm of chute water level and as far as water evaporation it was poured into the chute periodically.

The experiment lasted for 68 days.

Moisture plot changes of compacted light silt loam height wise the pipe at soil skeleton density 1,60 g/cm^3, moisture w = 0,130 after 70 days of «rest» is shown in Fig. 2 (a).

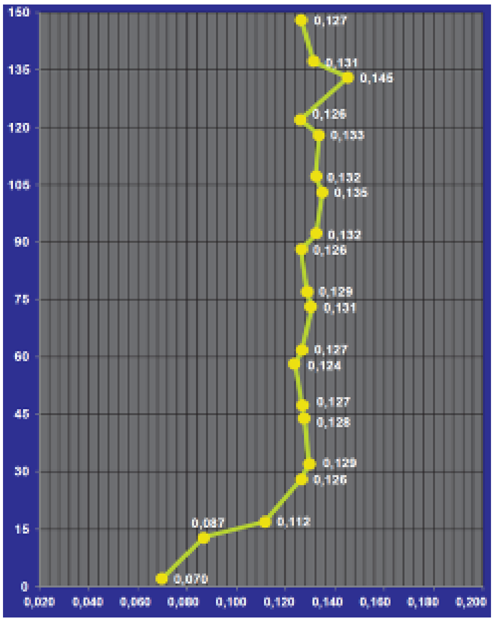

Figure 2 – Soil moisture plot changes depending on pipe height – moisture plot changes of compacted light silt loam, compacted at moisture corresponding to stabilized moisture value $w = w_k$ = 0,130, to soil skeleton density ρ_d = 1,60 g/cm^3 and pipe height of 150 cm,

Maximum moisture value w_k in accordance to the plot was 0,145, minimum moisture value in the lower pipe link was 0,070, the average moisture of pipe height wise was 0,124.

Moisture plot changes of compacted light silt loam height wise the pipe at soil skeleton density 1,60 g/cm^3, moisture w = 0,130 after 68 days of «rest» (at that research soil had the opportunity to boggy action) is presented in Fig. 2 (b).

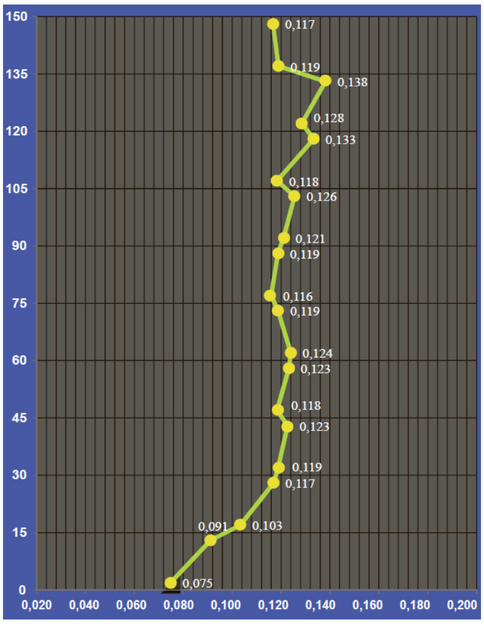

Figure 2 – Soil moisture plot changes depending on pipe height – the same, but in the experiment slot channel drain with stone screening dust was filled with water

Maximum moisture value w_k in accordance to the plot in pipe was 0,138, minimum moisture value in the lower pipe link was 0,075, the average moisture height wise the pipe was 0,121.

Fig. 2 a, b clearly shows that the average soil moisture in a plastic pipe did not change significantly (especially in the experiment, when the chute with stone screening dust was not filled with water) comparatively with initial soil moisture $w = w_k$ = 0,130 where the soil was compacted.

Thus, it can be **concluded** that subgrade clay soil moisture value, compacted at stabilized moisture (or maximum quantity of unfree water) does not significantly change through time.

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LONG-TERM SUBSIDENCE OF A MULTISTORY BUILDING ON THE BASE REINFORCED WITH SOIL CEMENT ELEMENTS

The methodology and results of long-term geodetic observation at subsidence of nine-ten storey building with a strip cast-in-place foundation on sandy and peaty base reinforced with soil cement elements at the process of its construction and exploitation are presented in the article. The correctness of elastic-plastic model use with Mohr Coulomb strength criterion and planar task finite element method for the evaluation of the deformed state «strip foundation – reinforced soil layer – the natural basis» system are substantiated.

Keywords: *soil cement element, strip cast-in-place foundation, settlement, geodetic observation, modulus of deformation, method of ultimate elements.*

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ТРИВАЛІ ОСІДАННЯ БАГАТОПОВЕРХОВОГО БУДИНКУ НА ОСНОВІ, АРМОВАНІЙ ҐРУНТОЦЕМЕНТНИМИ ЕЛЕМЕНТАМИ

Наведено методику та результати багаторічних геодезичних спостережень за осіданнями дев'яти-десятиповерхового будинку зі стрічковими монолітними фундаментами на піщаній і заторфованій основі, посиленій ґрунтоцементними елементами, в процесі його зведення та експлуатації. Обґрунтовано коректність використання пружно-пластичної моделі з критерієм міцності Мора-Кулона та плоскої задачі методу скінченних елементів для оцінювання деформованого стану системи «стрічковий фундамент – армований шар ґрунту – природна основа».

Ключові слова: *ґрунтоцементний елемент, стрічковий монолітний фундамент, осідання, геодезичні спостереження, модуль деформації, метод скінченних елементів.*

Introduction. The territory of Ukraine is characterized by the following difficult geotechnical conditions: collapsible and peaty soils, silts, poured soils, hydraulic fills, flooded area, dense housing. Soil cement element (SCE) and piles (SCP) are performed by stream and drilling-mixing technologies directly on building site. They are efficient in the soft soils strata with thickness up to 30 m [1].

Analysis of recent sources of research and publications. The material for their production is the ground, cement, water. A special bit of soil foundation from the bottom of the pit within the area of the bit is loosened, soaked with a water cement suspension, mixed to a condition of a mobile homogeneous cement mixture. If necessary, the spatial framework of steel fittings is immersed in a mixture. The stability of the well walls is unequivocally ensured by the presence of a mobile soil cement mixture in all soils, including floodplains. After of the soil cement mixture, the SCE or SCP of the design diameter and laying depth is formed [1 – 7].

The world experience of using the drilling-mixing method of cementations disperses soils showed that there is a complexity of soil cement mixture consolidation and its high water-cement ratio (in particular, below groundwater level). This leads to the fact that a significant portion of the water of the mixture is superfluous in the process of hydration of cement and forms its additional porosity. Such a phenomenon reduces the strength of soil cement, which leads to a decrease of bearing capacity of SCE and SCP on the material. In many cases, their bearing capacity on the soil is higher than the material. This reduces the efficiency of SCE and SCP and requires additional steps in the manufacture of such elements and piles [3 – 7].

It is also set that cement content increasing from 5% to 50% lead to the soil-cement mechanical properties increasing by linear dependence, so, structural strength of soil-cement is possible to regulate by cement content even for the complete replacement of soil by cement in mortar; in soil with lower content of clay grains is higher mechanical characteristics, for production of strong soil-cement sand with low content of clay grains is most effective; additives (sands, tails) using lead to soil-cement strength and deformation modulus increasing, so we recommend to use additives, tails using are more efficiency; soil-cement piles reinforced by steel frame allows to increase the carrying capacity by material to a value that exceeds the value of their carrying capacity by the soil [7].

Identification of general problem parts unsolved before. In massive of weak soils, due to their small deformation module, significant settlement of the base foundations of buildings and structures is possible, even under the condition of reinforcement of the SCE of a part of the compressive layer. Therefore, in order to expand the normative base of the design of SCE and increase its reliability, the method of determining the settlements of objects with strip foundations on the base of reinforced needs further improvement. The most reliable option for solving this problem is comparing the calculated and measured long-term geodetic observations of the values of settlements of natural objects.

That is why the **goal** of this article is to analyze the results of geodetic observations in time for the settlements of buildings with strip foundation on the basis of the reinforced SCE and the substantiation of the most reliable method of forecasting settlements of such objects.

Basic material and results. The platform for a residential building (sections I, II, IV, V – nine stores, III – ten) with a basement on the Panianka street, 65-b in Poltava, is made up of flood plains and stream-laid deposits of the Vorskla River, which are covered with 2.5 – 2.7 m of filled soil. For sections I – III, a layer (up to 2 m) of silty sand with impurities of organic substances is located under the filled soil. For IV and V there is a layer (also up to 2 m) of light silty clay, from slightly plastic to soft, with impurities of organic substances, as well as a layer (0.3 – 0.6 m) of clay heavy, fluid, strongly peaty. Under these layers is a three-meter thick of silty and fine sand of medium density. The ground water level is 2.3 – 2.5 m from the surface of the site.

Under the project of OJSC «Poltavtransbud» two-meter layer under the fill was reinforced with vertical SCE, which were made by drilling-mixing technologies (fig. 1, a). SCE was executed by the firm «Fundamentbud-3» using the drill machine BM-811 on the basis of the car «Ural». At the top of the SCE, a 0.5 m thick gravel pillow was poured into it, which erected monolithic reinforced concrete strip foundations 2200 mm wide under external longitudinal bearing walls and 3200 mm – under the median longitudinal bearing wall (Fig. 1, b). According to the project, to increase the overall rigidity of the building, monolithic reinforced concrete belts were installed and brick walls were reinforced.

a b

Figure 1 – The bottom of the foundation pit (sections IV and V) after reinforcing the SCE of the base (a), the gravel pillow, sprinkled on top of this base, and the strip monolithic foundations (b)

To determine the actual values of settling of the reinforced bases of the building, the observation of its settlements is organized by means of geometric leveling of the III class of accuracy by the method of Professor M. Zotsenko [8]. For this purpose, in the characteristic places of all sections at the level of the socle of the bearing walls, surface marks are installed: metal plies with a diameter of 20 mm, laid in the wall at a depth of about 130 mm, which protrude from the wall by 20 mm (Fig. 2). Immediately with the construction of the socle of the building, they arranged the wall marks, laid a leveling line and performed a zero-cycle observation. The following cycles were carried out after the construction of each storey, the acceptance of the building for exploitation, as well as its settlement. In the first two – three years of operation of the building measuring performed 2 – 3 times a year, and now approximately once a year.

Fig. 3 contains a scheme of placing leveling deformation marks on the object of research and connecting points. The modern look of the object is shown in Fig. 4

a b

Figure 2 – Wall siege marks at the research object:
a – at the time of their arrangement; b – at the time of leveling 01.10.2017

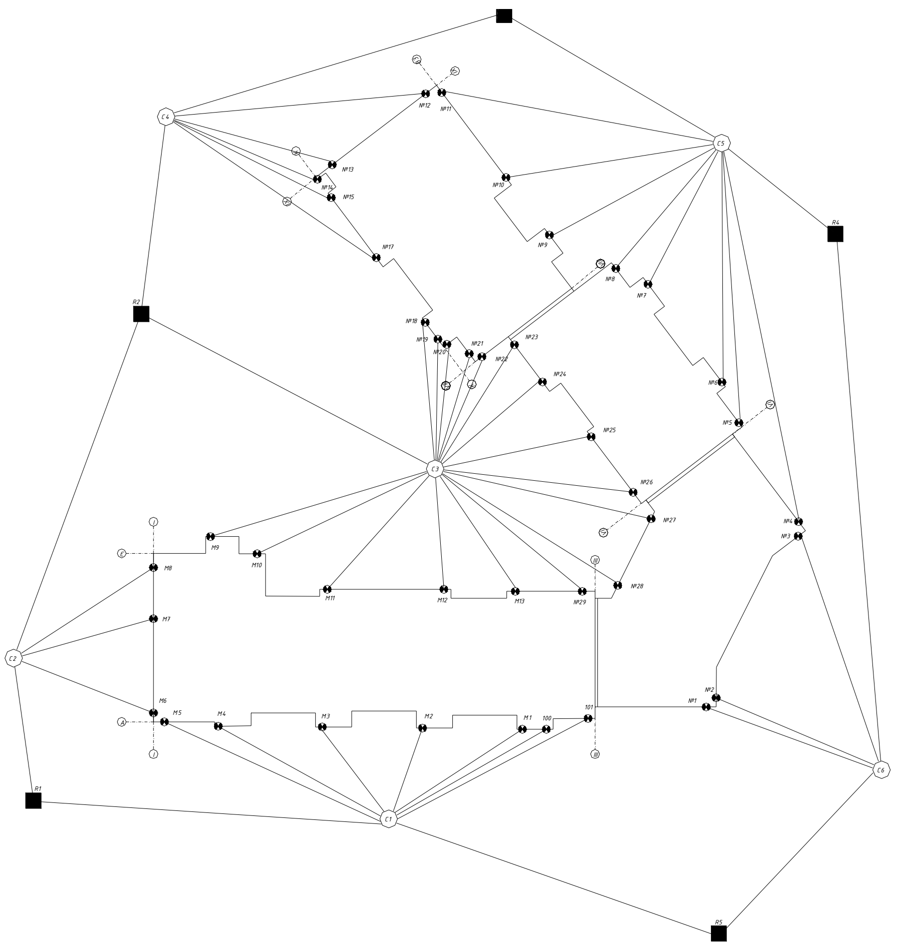


Figure 3 – Scheme of geodetic leveling deformation marks established at the research object and the connecting points between them

Figure 4 – Modern observations of the object – a house on the Panianka street, 65-b in Poltava (01.10.2017)

The results of field observations of the building were the following: the scheme of its leveling with the placement of marks and connecting points; diagrams settlements marks on the subject during its construction and exploitation (Fig. 5); graphs of development of minimum $S_{min}(t)$, average $\overline{S}(t)$ and maximum $S_{max}(t)$ settled marks in time, combined with the schedules of the construction of stories and the exploitation of buildings (Fig. 6); absolute and relative magnitude of settlements of buildings at the time of the last cycle of observations.

First, we note that the settling sections I and II are less than the other three. In addition, the conditional stabilization of deformations in this section can be traced more clearly than in the other three. This can be attributed to the following factors: the construction of section I-II began and ended earlier than others, and, consequently, the time span from the application of the last significant load on the building is less; the soil conditions under section I-II are better, while the constructive solutions and dimensions of the foundations are the same.

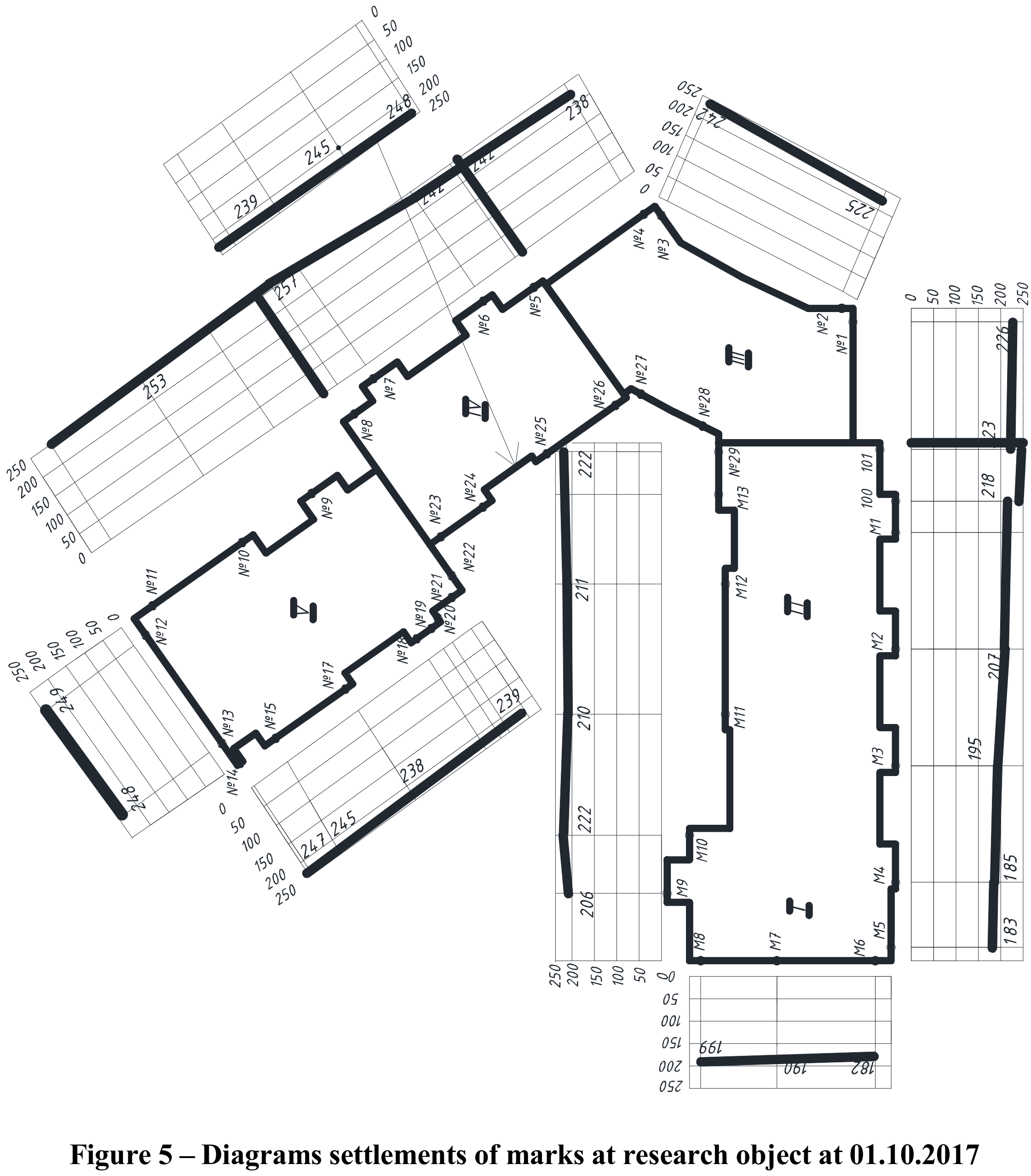
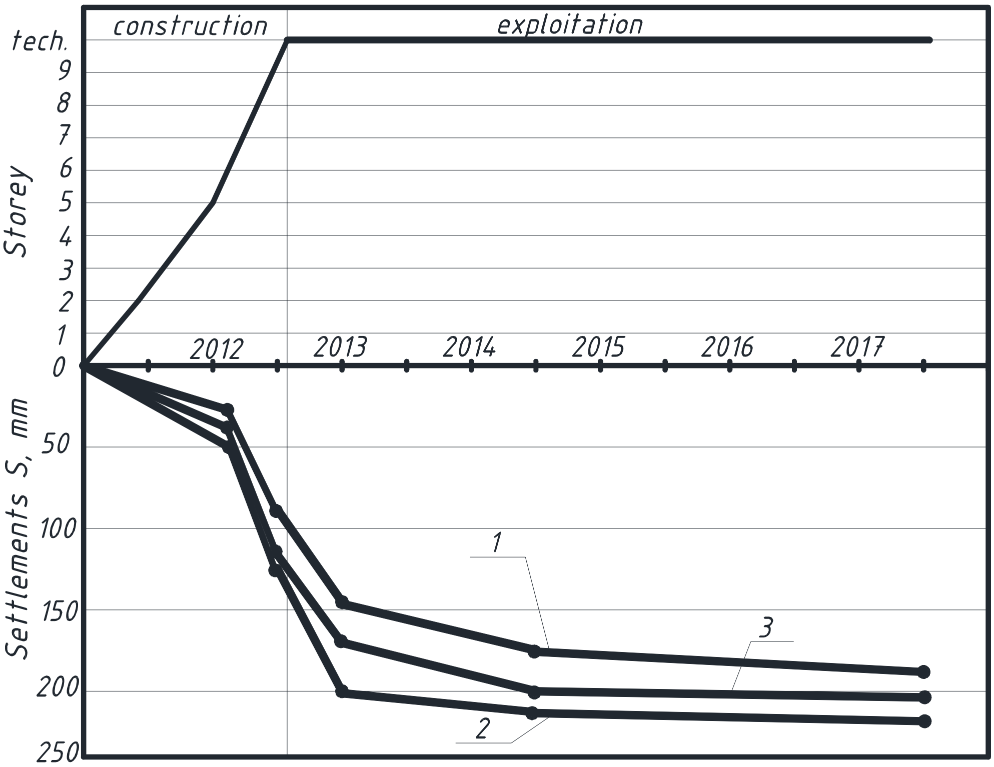


Figure 5 – Diagrams settlements of marks at research object at 01.10.2017

a)



b)

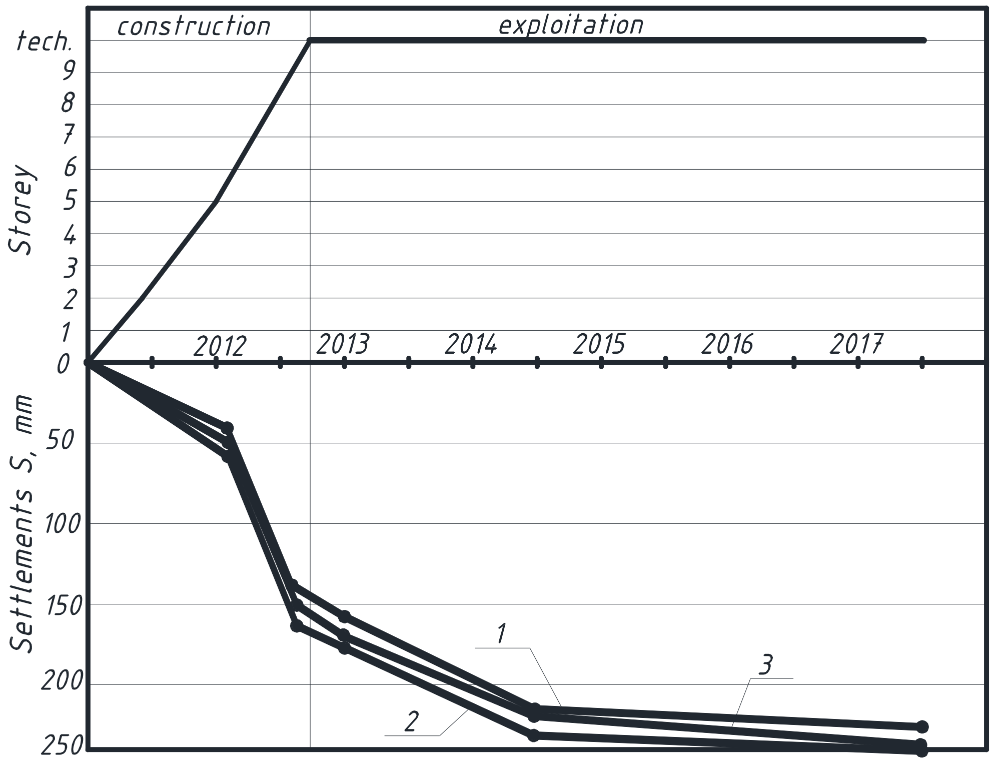


Figure 6 – Graphs of the development of maximum, minimum and average settlements marks in time at the research object:
a – section I i II; b – section IV

In particular, it was found that at 1.10.2017:

– the minimum settlements of the wall marks of section I-II building was 182 mm, average – 204,5 mm, and maximum – 223 mm; similar data are for section III – 225, 235,8 and 248 mm respectively; for section IV – 239, 245,7 and 257 mm respectively; for section V – 238, 245,6 and 253 mm respectively;

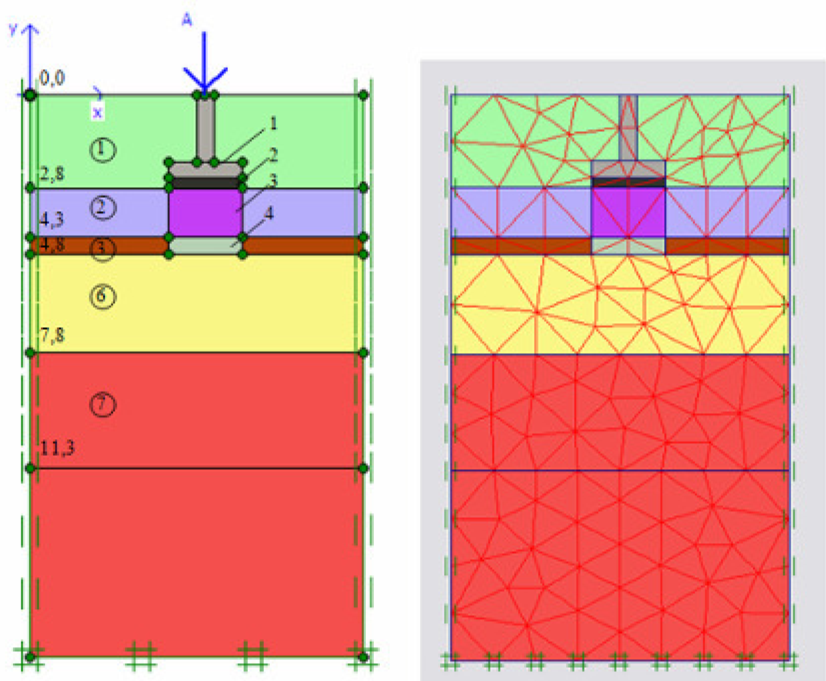
– the average values of settlements exceeded the maximum allowable values for all sections of the building Su = 180 mm, but the relative differences in settlement; ΔS/L did not exceed the maximum permissible values of the magnitude ΔS/L = 0,004 [2];

– cracks and other visible defects or deformations in the building were not revealed, and its technical condition is defined as the state I – normal;

– there is a tendency towards stabilization of settlements (conditional stabilization – 1 mm / year) basis of the foundations of the building, especially for section I-II.

After this, the settlements bases of the foundations of section IV are calculated by normative methods (layered summation and I.O. Rosenfeld) [2]. They turned out to be more than twice as small as the actual values.

Therefore, in the future, the analysis of settlement of the reinforced bases of strip cast-in-situ foundations was performed by modeling the flat (2D) version of the Finite Element Method (FEM) using an elastic-plastic soil model. At the same time, the value of the deformation modules of soils within the reinforcement mass was determined as a weighted average depending on the percentage of reinforcement. The values of unit cohesion soil in the reinforced mass were also taken as weighted average, and the angle of internal friction – as for soils in the natural state. In fig. 7 shows the calculation scheme of the base of the building.

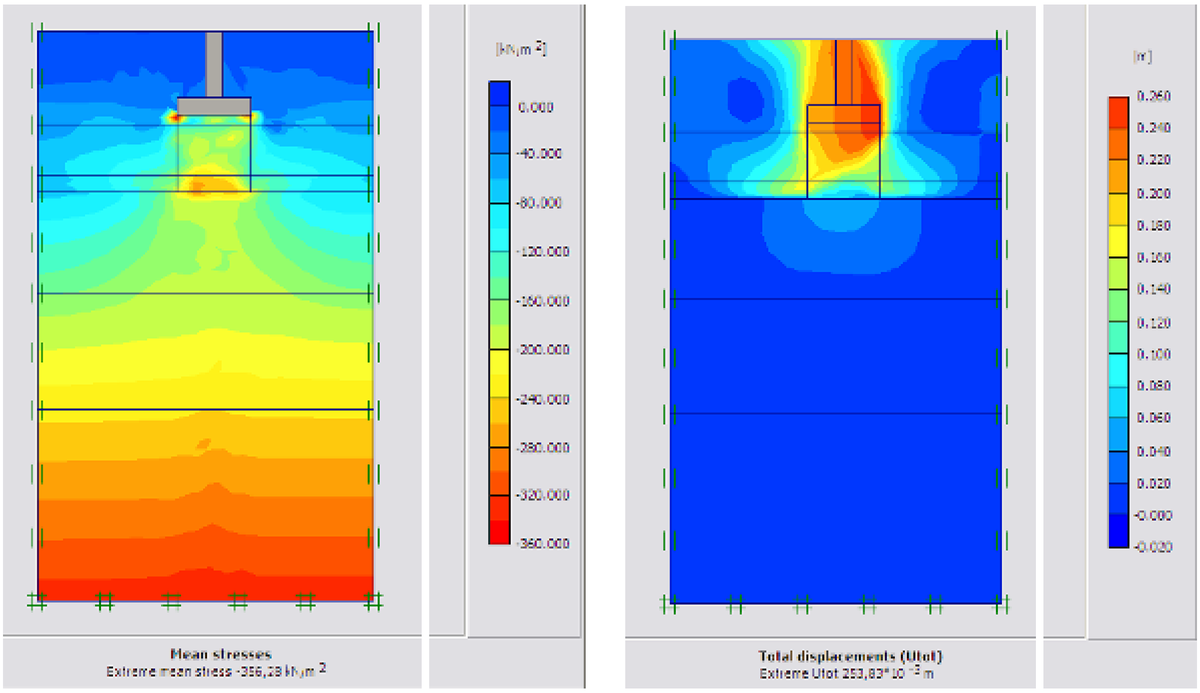


a b

Figure 7 – Estimated 2D (a) deformed (b) scheme o f the base of the foundation of section IV:

1 – foundation; 2 – gravel pillow; 3 i 4 – zones of soil layers, reinforced by SCE

Fig. 8 shows the general vertical stresses and deformations of the base of the section IV after applying the load to it F_V = 650 kN.



a b

Figure 8 – General stresses (a) and deformations (b) of the base of the section IV

In particular, for the system «reinforced GCE base – strip foundation» the corresponding settlements were about 250 mm, which has a high convergence with the results of field observations.

So, **conclusions**, as a result of geodetic observations of the settlements of nine to ten storey buildings with strip cast-in-place foundations on the basis of compiled soils containing organic matter and hardened SCE. New experimental data on the development of actual deformations of such bases in time have been obtained. The high reliability of the FEM modeling of the «reinforced base – strip foundations» system with the use of the flat version of the PLAXIS complex has been confirmed. Application of an elastic-plastic soil model with a strength criterion for Mohr-Coulomb is useful in determining the settlements of reinforced SCE weak bases of strip cast-in-place foundations.

The strengthening of the base only within the layer of weak soils (its capacity is even less than the width of the foundations) for the strip cast-in-place foundations was not sufficient. In order to comply with the norms for limiting sediments, the reinforcement of such bases within a short layer must be carried out at a much higher depth, which should be established by simulation. The same method of reinforcing the bases of SCE has confirmed its effectiveness for soils with high content of organic substances.

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TRANSVERSAL LOADED PILES DEFORMATION TASK DECISION METHOD

The deformation method allows to describe the stress-strain state of foundation structures by means of dependency that binds settling of foundations to the parameters of stiffness in the system «base – piles foundation» at different stiffness coefficients of the basis, along length or depth of foundation. The proposed methodology allows improving the calculations of the stress-strain state of laterally loaded piles, which can significantly improve the performance of buildings and structures. Deformations of a foundation structure are described by approximate dependency that includes the sinking of ends of foundation and the stiffness parameter of the system "basis-foundation". The calculation embraces various (linear and nonlinear) distributions patterns of the of the stiffness coefficient of the basis along the length of the structure as well as distribution properties of the ground basis.

***Keywords:** pile, Winkler basis, lateral load, horizontal deformations, soil stiffness coefficient.*

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ДЕФОРМАЦІЙНИЙ СПОСІБ РОЗВ’ЯЗАННЯ ЗАДАЧ ПОПЕРЕЧНО НАВАНТАЖЕНИХ ПАЛЬ

Розв’язання задач деформаційним способом дозволяє описувати напружено-деформований стан фундаментних конструкцій за допомогою залежності, яка пов’язує осідання фундаментів з параметрами жорсткості в системі «основа – пальовий фундамент» при різних коефіцієнтах жорсткості основи по довжині або глибині фундаментів. Запропонований спосіб дозволяє вдосконалити розрахунки напружено-деформованого стану поперечно навантажених паль, що дає змогу значно підвищити експлуатаційні якості будівель і споруд. Описано деформації фундаментної конструкції наближеною залежністю, що включає в себе осідання кінців фундаменту та параметр жорсткості системи «основа – фундамент». Розрахунок охоплює різноманітні (лінійні й нелінійні) закономірності розподілу коефіцієнта жорсткості основи по довжині конструкції та розподільні властивості ґрунтової основи.

***Ключові слова:** паля, основа Вінклера, поперечне навантаження, горизонтальні деформації, коефіцієнт жорсткості ґрунту.*

Introduction. In view of construction and reconstruction expansion [5] of buildings and facilities in town cramped conditions, as well as in complicated engineering and investigation conditions, there is a significant growth of piles usage for foundations and enclosure structures. New experimental and technical research [1 – 4, 6] has been accumulated within last years, which allows to make significant improvements in the regulations of existing standards, and besides, in many cases it allows to produce more cost-effective or more reliable buildings. The deformation method allows to improve the calculation of stress-strained state of piles and other foundation types through the expression of foundation structures' deformation as a dependency of foundation settling on the "basis-foundation" system stiffness and the basis stiffness coefficient that varies along the length or the depth of foundations, which allows to improve calculations quality for various types of foundations.

Analysis of the latest sources of research and publications. V.A. Florin [8] in his research came very close to the «deformation» method in solving of contact problems for foundation calculations, on the basis of the bed coefficient linear distribution. He examined only absolutely stiff structures. However, V.A. Florin demonstrated how, having used the laws of bed coefficient variation and foundation deformation, it would be possible to obtain the balance equations from where angle of structure twist can be obtained, and subsequently, the response of the basis can be obtained as well.

The geeneral problem parts selection unsolved earlier. This approximation method relates to the piles calculation implementing various laws of stiffness variation in the Winkler's basis [4 – 5], where solving of differential equations for bending of structures, based on such a basis, causes mathematical challenges, and sometimes would be impossible altogether, for example, if it is tried to implement the basis stiffness coefficient with nonlinear distribution along the length, into a bending pile axis equation, it is obtained a non-converging function that neither allows to give a specific solution nor allows to generate the calculation tables.

Problem definition. The task is to define the calculation method for laterally loaded piles foundation that would more accurately describe its interaction with the Winkler's basis, as well as developing the calculation methodology of «pile-ground» system.

To achieve this task, the following problems were being solved:

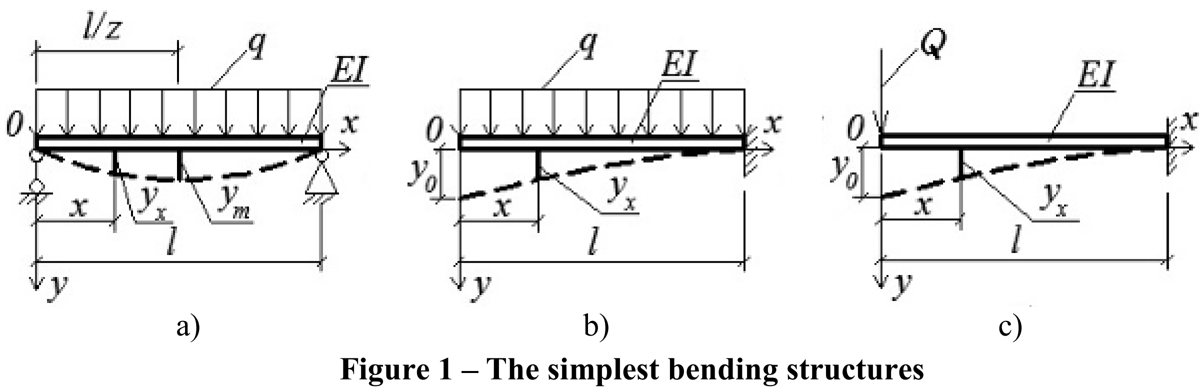
– the analysis of the current calculation models of the «laterally loaded pile-soil» system as well as experimental laboratory and field studies;

– the formulation the of analytical and numerical calculation method;

– the comparison of the theoretical results obtained with the current solutions and results that were produced in the software package.

The primary material and results. The essence of the deformation method is the replacement of the equation of a bending pile axis or foundation by an approximation equation that includes a correction coefficient for stiff structures – ξ.

If analyse from the simple to the complex, it can be considered this device to be used the simplest single-span beams (Fig. 1, *a*) and consoles (Fig. 1, *b, c*).



a) b) c)

Figure 1 – The simplest bending structures

The solution of differential equations for bending of these structures is rendered in the course of Materials Resistance like this:

$$\text{a) } y_x = \frac{q \cdot l^4}{24 \cdot EI} \cdot \left(\bar{x}^4 - 2 \cdot \bar{x}^3 + \bar{x} \right) , \tag{1-a}$$

$$\text{b) } y_x = \frac{q \cdot l^4}{24 \cdot EI} \cdot \left(\bar{x}^4 - 4 \cdot \bar{x} + 3 \right) , \tag{1-c}$$

$$\text{c) } y_x = \frac{Q \cdot l^3}{6 \cdot EI} \cdot \left(\bar{x}^3 - 3 \cdot \bar{x} + 2 \right) . \tag{1-c}$$

Various dependencies were applied to describe the deformation of these beam structures. For example, it is suggested for a single-span beam that is symmetrically loaded

$$y_1 \approx 4 \cdot y_{max} \cdot \bar{x} \cdot \left(1 - \bar{x} \right) , \tag{2-a}$$

or

$$y_1 \approx y_{max} \cdot sin\left(\pi \bar{x} \right) . \tag{2-b}$$

Where in this case of the load

$$y_{max} = \frac{5}{384} \cdot \frac{q \cdot l^4}{EI} .$$

For console beams it was recommended:

$$y_1 = y_0 \cdot \left\{ 1 - \bar{x} \cdot \left[1 + \xi \cdot \left(1 - \bar{x} \right) \right] \right\} . \tag{3}$$

Wherein in the cases *б)* and *в)* it is got

$$\left. \begin{aligned} y_0 &= \frac{q \cdot l^4}{8 \cdot EI} ; \\ y_0 &= \frac{Q \cdot l^3}{3 \cdot EI} . \end{aligned} \right\} \tag{4}$$

As demonstrated by the results of calculations found in the Table 1, the deformations in strict solution can be rendered accurately enough when applying recommended dependencies. In this case, however, the structures of uniform bending stiffness were considered.

However, this dependency cannot be applied for beam and banded structures having contact with the basis, as both ends of these structures get two types of movement such as y_0 and y_l (Fig. 2) under the load, and besides, these types of movement may have different signs.

The deformation of piles (Fig. 2, *a*) is represented by the formula

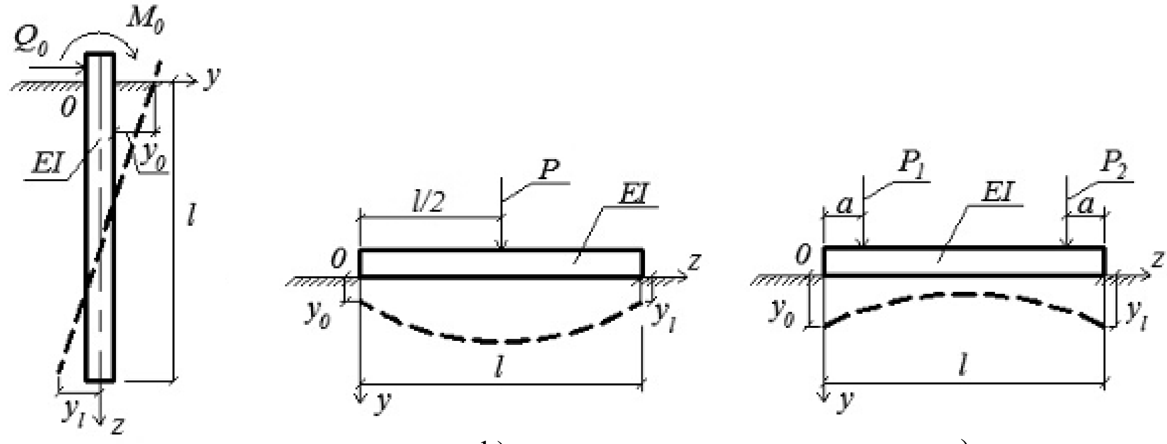
$$y_z = y_0 - \bar{z} \cdot \left(y_0 - y_l \right) \cdot \left[1 + \xi \cdot \left(1 - \bar{z} \right) \right] , \tag{5}$$

which can be rendered in relevant units as

$$\overline{y_z} = 1 - \left(1 - \overline{y_l} \right) \cdot \left[1 + \xi \cdot \left(1 - \bar{z} \right) \right] \cdot \bar{z} , \tag{6}$$

Table 1 – The results of the simplest bending structures calculation

Structure	$\bar{x}$ / $\bar{y}$	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Beam *a)*	$\bar{y}$	0.0	0.300	0.594	0.813	0.952	1.0	0.952	0.813	0.594	0.300	0.0
	$\bar{y}_1$-*a*	0.0	0.360	0.640	0.840	0.960	1.0	0.960	0.840	0.640	0.360	0.0
	$\bar{y}_1$-*b*	0.0	0.309	0.588	0.809	0.951	1.0	0.951	0.809	0.588	0.309	0.0
Console *b)*	$\bar{y}$	1.0	0.867	0.734	0.603	0.475	0.354	0.243	0.147	0.070	0.019	0.0
	$\bar{y}_1$ if ξ=0.6	1.0	0.846	0.704	0.574	0.456	0.350	0.256	0.174	0.104	0.046	0.0
Console *c)*	$\bar{y}$	1.0	0.850	0.704	0.564	0.432	0.313	0.208	0.122	0.056	0.015	0.0
	$\bar{y}_1$ if ξ=0.6	1.0	0.846	0.704	0.574	0.456	0.350	0.256	0.174	0.104	0.0	0.0



a) b) c)

Figure 2 – Piles, beam and banded foundations

If it is calculated a band foundation that is symmetrically loaded at the center (Fig. 2, *b*), it would be convenient to describe the deformation as the following dependencies

$$y_z = y_0 \cdot \left[1 + 4 \cdot \xi \cdot \bar{z} \cdot \left(1 - \bar{z}\right)\right] , \tag{7-a}$$

or

$$y_z = y_0 \cdot \left[1 + \xi \cdot sin\left(\pi \cdot \bar{z}\right)\right] . \tag{7-b}$$

In case of symmetrically loaded band ends (Fig. 2, c) it is used the following formulas

$$y_z = y_0 \cdot \left[1 - 4 \cdot \xi \cdot \bar{z} \cdot \left(1 - \bar{z}\right)\right] , \tag{8-a}$$

or

$$y_z = y_0 \cdot \left[1 - \xi \cdot sin\left(\pi \bar{z}\right)\right] \tag{8-b}$$

The calculation of laterally loaded piles with linearly increasing law of bed coefficient distribution.

If it is considered three types of piles loadings from an external load to be: Q_0, M_0 – at free end of a pile and q – along the length of a pile trunk (Fig. 3).

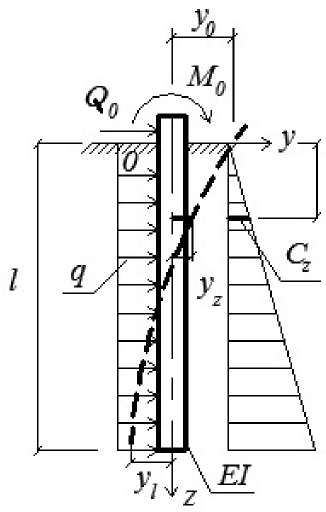


Figure 3 – The calculation diagram of a laterally loaded pile

It is represented the equation of a bending pile axis as

$$y_z = y_0 - (y_0 - y_l) \cdot \bar{z} \cdot \left[1 + \xi \cdot (1 - \bar{z})\right] \ . \tag{9}$$

The law of the bed coefficient variation

$$C_z = K \cdot z \ . \tag{10}$$

Therefore, the law of contact strain variation σ_z along the depth of a pile will be expressed as

$$\sigma_z = K \cdot l \cdot \bar{z} \cdot \left\{y_0 - (y_0 - y_l) \cdot \bar{z} \cdot \left[1 + \xi \cdot (1 - \bar{z})\right]\right\} . \tag{11}$$

Next, it is got the total resistance of a basis

$$\int_0^l \sigma_z dz = K \cdot l^2 \cdot \left\{\frac{y_0}{2} - (y_0 - y_l) \cdot \left[\frac{1}{3} + \xi \cdot \left(\frac{1}{3} - \frac{1}{4}\right)\right]\right\} = \\ = \frac{K \cdot l^2}{12} \cdot \left[6 \cdot y_0 - (y_0 - y_l) \cdot (4 + \xi)\right] \tag{12}$$

The equation (11) will be represented as

$$\frac{\sigma_z}{K \cdot l} = y_0 \cdot F_1^z + y_l \cdot F_2^z \quad , \tag{13}$$

where

$$\left.\begin{aligned} F_1^z &= \bar{z} - \bar{z}^2 \cdot \left[1 + \xi \cdot (1 - \bar{z})\right]; \\ F_2^z &= \bar{z}^2 \cdot \left[1 + \xi \cdot (1 - \bar{z})\right]. \end{aligned}\right\} . \tag{14}$$

Next, it is discussed various cases of the load on a pile.

1. Evenly distributed load q

It is put together the conditions of pile balance.

From the condition $\Sigma N = 0$, applying the equation (12), it is got

$$y_0 \cdot (2-\xi) + y_l \cdot (4+\xi) = \frac{12 \cdot q}{K \cdot l} \; . \tag{15}$$

From the condition $\Sigma M = 0$ and the equation (13) it can got

$$\left(y_0 \cdot f_1 \cdot \overline{z_1^0} + y_l \cdot f_2 \cdot \overline{z_2^0}\right) \cdot K \cdot l^3 = \frac{q \cdot l^2}{2} \; , \tag{16}$$

Wherein f_i and $\overline{z_i^0}$ are the areas of functions F_1^z, F_2^z and their distance to the axle F_i, are calculated with the help of integration of these functions.

Therefore, the system of two equations (15) and (16) was derived to determine the two unknown variables y_0 and y_l,

For example, if a pile is «long», with the reduced length $\overline{l} = 4$, according to the proposed methodology, it is assumed ξ = 1. Next, it is calculated the values $f_1 = 0{,}0833$; $f_2 = 0{,}41667$; $\overline{z_1^0} = 0{,}404$; $\overline{z_2^0} = 0{,}720$ (Fig. 4), which allows to put together the system of equations (15) and (16):

$$\left.\begin{aligned} y_0 + 5 \cdot y_l &= \frac{12 \cdot q}{K \cdot l} \\ y_0 \cdot 0.0833 \cdot 0.404 + y_l \cdot 0.41667 \cdot 0.720 &= \frac{1}{2} \cdot \frac{q}{K \cdot l} \end{aligned}\right\} .$$

The solving of this problem gives $y_l = 0.729 \cdot \dfrac{q}{K \cdot l}$; $y_0 = 8.355 \cdot \dfrac{q}{K \cdot l}$.

When it is put it into (9), it is obtained the equation for pile deflections, and when it is put it into (11), it is obtained the distribution of contact strains, which allows to draw diagram of bending moments.

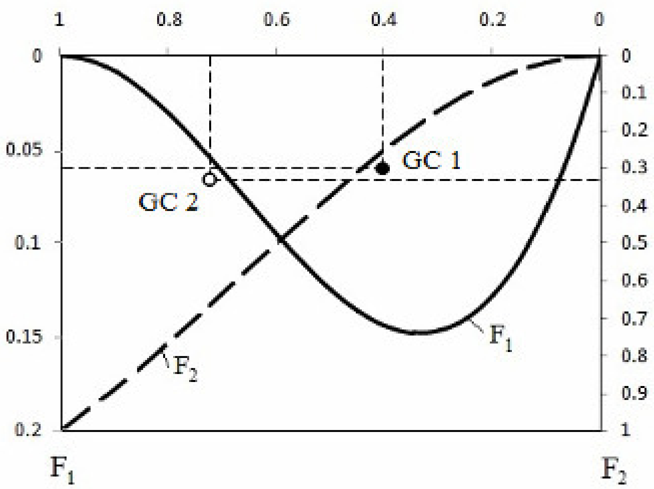


Figure 4 – Functions F_1, F_2 and positions of their gravity centers if $\xi = 1$

The equation for deflections and resistances of the soil can be represented as

$$\left.\begin{aligned} y_z &= L_{1q}^z \cdot \frac{q}{K \cdot l} \\ \sigma_z &= L_{2q}^z \cdot q \end{aligned}\right\} , \tag{17}$$

where L_1^z and L_2^z are functions of deflections and resistances,

$$\left.\begin{aligned} L_{1q}^z &= 0.729 + 7.626\cdot\left(1-\bar{z}\right)^2 \\ L_{2q}^z &= \bar{z}\cdot\left[0.729 + 7.626\cdot\left(1-\bar{z}\right)^2\right] \end{aligned}\right\} . \tag{18}$$

If it is replaced, for example, the resistance by ten concentrated forces

$$-P_z = 0.1\cdot L_2^z\cdot q\cdot l ,$$

And also, if it is replaced the distributed load *q by ten forces*

$$Q_z = 0.1\cdot q\cdot l ,$$

Then it becomes possible to figure out the bending moments along the axis of a pile (a beam)

$$M_z = L_3^z\cdot q\cdot l^2 , \tag{19}$$

where L_3^z is a function of bending moments.

The values of new functions L_1^z, L_2^z and L_3^z are represented in the Table 2.

Table 2 – Values of functions L_1^z, L_2^z and L_3^z for various relative depths $\bar{z}$

$\bar{z}$	0.00	0.05	0.15	0.25	0.35	0.45	0.55	0.65	0.75	0.85	0.95	1.00
L_1^z	8.355	7.6115	6.239	5.019	3.951	3.036	2.273	1.663	1.2056	0.9006	0.748	0.729
L_2^z	0.000	0.3806	0.936	1.2547	1.383	1.366	1.2503	1.081	0.9042	0.7655	0.7107	0.729
L_3^z	0.000	0.0048	0.0113	0.0185	0.0231	0.0238	0.0210	0.0156	0.0094	0.0042	0.0004	0.000

Calculation example 1

To draw diagrams for y_z, σ_z and M_z for a flexible pile $(\alpha_д\cdot l = 4)$, evenly loaded along its buried part, the load being q = 10 t/m^2.

Initial data: pile length l = 5 m, design width $b_p = 1$ m, deformation coefficient $\alpha_д = \sqrt[5]{\dfrac{K\cdot b_p}{EI}} = 0.8$ m^{-1}.

The calculation was completed according to the Table 2 and represented at the illustration 5. Concurrently, pile calculations were carried out with the software package «SCAD» and according to the methodology by A.S. Maliev [7].

The suggested deformation methodology gave some intermediate values for deflections, resistances and bending moments.

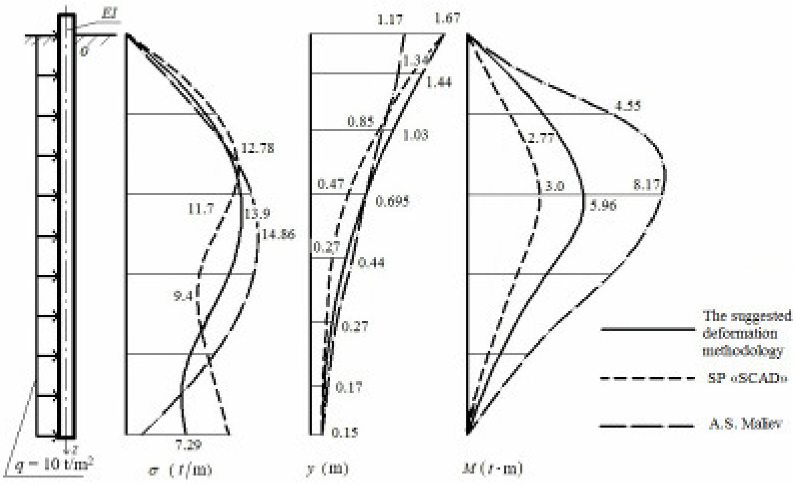


Figure 5 – An example of calculation for a flexible pile with distributed load

2. Concentrated load Q_0

To solve this problem it is meant to make up balance equations $\Sigma N = 0$ and $\Sigma M = 0$, which in this case is:

$$\left.\begin{array}{l} y_0 \cdot (2-\xi) + y_l \cdot (4+\xi) = \dfrac{12 \cdot Q_0}{K \cdot l^2} \\ y_0 \cdot f_1 \cdot \overline{z_1^0} + y_l \cdot f_2 \cdot \overline{z_2^0} = 0 \end{array}\right\} . \tag{20}$$

If note that the functions F_1^z and F_2^z are dependent only on what model is accepted for basis, and in this case they are described by formulas (14). Therefore, when it is determined f_i and $\overline{z_i^0}$ their values depend on what stiffness coefficient ξ has been assumed. With this type of the load on a long pile $(\overline{l} = 4)$ it is recommended to assume ξ = 1.4.

In this case the numerical methodology would give

$$f_1 = 0.05 \ ; \ f_2 = 0.450 \ ; \ \overline{z_1^0} = 0.3603 \ ; \ \overline{z_2^0} = 0.7111 \ .$$

When applying these values to solve the system (20), it is got (if ξ = 1.4)

$$y_l = -2.2824 \cdot \frac{Q_0}{K \cdot l^2} \ ; \ y_0 = 40.54 \cdot \frac{Q_0}{K \cdot l^2} .$$

Wherein the functions of deflections and resistances are:

$$\left.\begin{array}{l} L_{1Q}^z = 40.54 + 42.822 \cdot \left(1-\overline{z}\right)^2 \\ L_{2Q}^z = \overline{z} \cdot \left[40.54 + 42.822 \cdot \left(1-\overline{z}\right)^2\right] \end{array}\right\} . \tag{21}$$

3. Pile loaded by the moment M_0

In this case of the load, the calculation gets somewhat simplified, because from the condition $\sum N = 0$ it is obtained

$$y_0 \cdot (2-\xi) + y_l \cdot (4+\xi) = 0,$$

where the ratio of movements of the ends:

$$\frac{y_0}{y_l} = -\frac{(4+\xi)}{(2-\xi)} . \tag{22}$$

Next, from the condition $\sum M = 0$ it is obtained

$$\left(y_0 \cdot f_1 \cdot \overline{z_1^0} + y_l \cdot f_2 \cdot \overline{z_2^0}\right) = -\frac{M_0}{K \cdot l^3} . \tag{23}$$

When it is considered a long pile ($\overline{l} = 4$), it should be assumed $\xi = 1.6$.

In this case, it is obtained (provided ξ = 1.6 is assumed)

$$f_1 = 0.0333 \ ; \quad f_2 = 0.4667 \ ; \quad \overline{z_1^0} = 0.3933 \ ; \quad \overline{z_2^0} = 0.7071 \ .$$

Using the conditions (22), (23), it is obtained

$$y_l = -7.069 \cdot \frac{M_0}{K \cdot l^3} \ ; \ y_0 = 98.968 \cdot \frac{M_0}{K \cdot l^3} .$$

Wherein the functions of deflections and resistances are:

$$\left.\begin{aligned} L^z_{1M} &= 98.968 + 106.037 \cdot \left(1-\bar{z}\right)^2 \\ L^z_{2M} &= \bar{z} \cdot \left[98.968 + 106.037 \cdot \left(1-\bar{z}\right)^2\right] \end{aligned}\right\}. \tag{24}$$

Conclusions. The above mentioned calculation approaches can be united into the unified «deformation» methodology, which allows:

– to express deformation of foundation structure as approximate (or exact) dependency that includes the settling of foundation ends and unknown (or preset) parameter ξ; of the «basis-foundation» system stiffness

– to utilize random law of basis stiffness coefficient variation (proportionality), including the one imitating distribution properties of the current models that have distribution properties (half-plane, half-space, finite thickness basis, etc.);

– to establish the law for basis resistance variation along the structure;

– to find out the unknown movements, by integration, using balance conditions.

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FOUNDATIONS OF THE HIGH RISE BUILDING IN THE AREA OF UNDERGROUND MINING

Problematic issues of construction of pile-foundation slab of high-rise residential building in the area of underground mining (underground mining with general under working area 25%; the fissured limestone may collapse under the weight of the building) are systematized. The experience of modeling by method of ultimate elements of pile-foundation slab of three-section residential building in the area of underground mining and results of the geodesic monitoring of complex building are presented.
Keywords: *underground mining, pile-foundation slab, fissured limestone, method of ultimate elements, stressed-deformed state, geomonitoring.*

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ФУНДАМЕНТИ ВИСОТНИХ БУДІВЕЛЬ У ЗОНІ ПІДЗЕМНИХ ВИРОБОК

Систематизовано проблемні питання влаштування пальово-плитних фундаментів висотного житлового комплексу в зоні підземних виробок: підземні виробки із загальною площею підробітки 25%; вістря паль спираються у тріщинуваті вапняки, які під вагою будівлі можуть продавитися. Наведено досвід моделювання методом скінченних елементів роботи пальово-плитного фундаменту трихсекційного житлового будинку в зоні підземних виробок і результати геодезичного моніторингу будівництва комплексу.
Ключові слова: *підземні виробки, пальово-плитний фундамент, тріщинуваті вапняки, метод скінченних елементів, напружено-деформований стан.*

Introduction. Terms of construction of modern residential complexes are constantly complicated. On the one hand, the surface area and, accordingly, the load on the base increase, and on the other – as areas for development, the territories are used in densely urban conditions with complex geotechnical properties [1, 2].

One of the options for these problems is the presence of foundations of underground voids. Approaches to determining the bearing capacity of rocks on a cut during drilling also remain relevant [2 – 6].

Analysis of recent sources of research and publications. The experience of erecting the foundations of high-rise buildings, including at the area of underground voids, is given in papers [2 – 6].

From their analysis, it is possible to generalize the high efficiency and reliability of pile-slab foundations for buildings and structures at the underground void area [2 – 8].

The current level of software, in particular the use of spatial solutions of the finite element method (FEM), makes it possible to direct methods for assessing the stress-strain state (SSS) of the foundations and foundations precisely to solve similar, purely practical tasks of geotechnical planners [8 – 11].

However, the problem of the possibility of crushing the thicker rocks from the weight of the building and the methods of their solution in the practice of geotechnical design is still not sufficiently investigated.

Identification of general problem parts unsolved before. Solutions on the erection of foundations in soils, composed of layers of inhomogeneous limestone, with underground voids, and the providing their reliable work require both experimental justification and numerical simulation, which are a debatable issue of geotechnics for the moment.

Basic material and results. The object of research – a residential complex of three separate sections on 24 floors and one – 20 floors (Fig. 1), located in the city of Odessa for the street Genoese, 24, d (Fig. 2). The building has a two-level underground parking. The first three sections are adjacent, the fourth is away from them. The sections have the following dimension: №1 – 32×25 m; №2 – 25×24 m; №3 – 29×24 m; №4 – 26,54×23,3 m. Constructive scheme of the building – monolithic-frame.

Figure 1 – General view of the residential complex

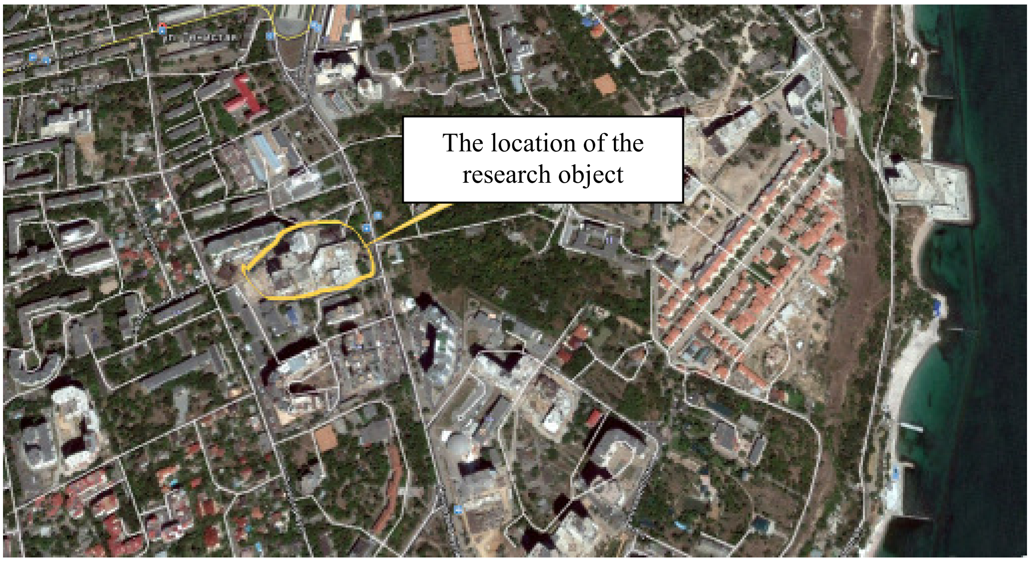


Figure 2 – Situation scheme of the location of research object

The authors have developed a robust, constructive solution to the foundations under these conditions. At the same time, the integrity of underground workings in the height of 2.4 m and 4.0 m in height (Fig. 3), which are in cracked limestones at a depth of 7 ... 8 m under the first three sections (Figure 4) (the value of forgery is 25 %) The design of the foundations provides a bearing ability to cut when crushing the thickness of cracked limestone (h_{min}=9,5 m) weight of the complex.

Figure 3 – General view of underground workings

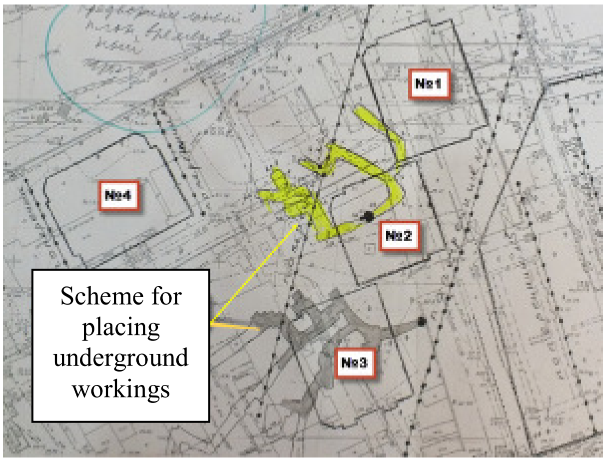


Figure 4 – General plan of the plot with the scheme of accommodation sections of the residential complex and underground workings

To clarify the geotechnical parameters of cracked limestones (underground workings are in them), which are the basis of piles and can be cut off when crushed against the weight of the building, field tests of soils have been conducted. The vertical loading load on a bale with a diameter of 500 mm and a length of 11.45 m with a base in limestone is brought to 2000 kN with its stabilized displacement of 0.95 mm. The vertical dismounting load on a catcher with a diameter of 500 mm and a working length of 1.24 m in lime limestone is brought to 325 kN with its stabilized displacement of 0.67 mm. Terms of limestone work on cuttings are determined by stamp tests.

The calculated resistance of the cut of limestone was R_{cp1} = 280 kPa at fracture on a plane with a slope 33º to the vertical, i R_{cp2} = 220 kPa – with a slope 45º to the vertical. In this case, the permissible stress of the cut is R_{cp} = 157 kPa. Actual compressibility of limestones is determined by stamp tests. The results of field tests of the base used for the geotechnical design of the complex. In zones with underground workings, additional reinforcing elements are the auger injected piles (Fig. 5 and 6).

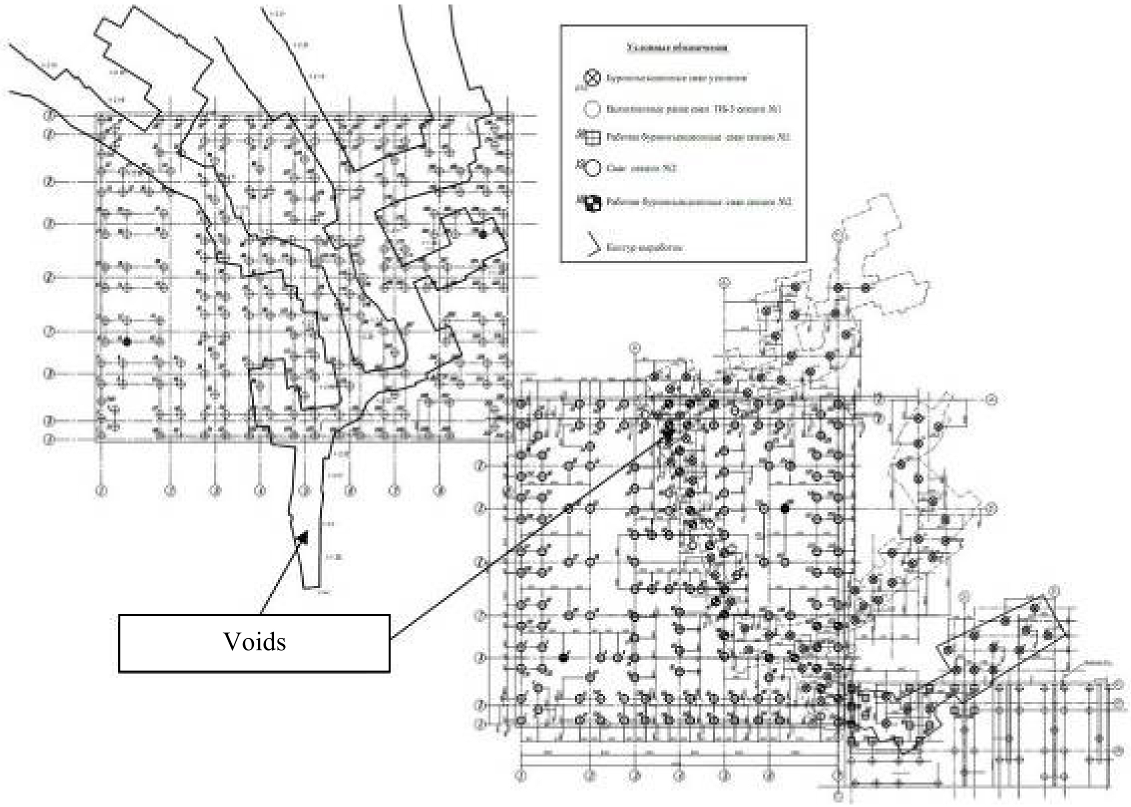
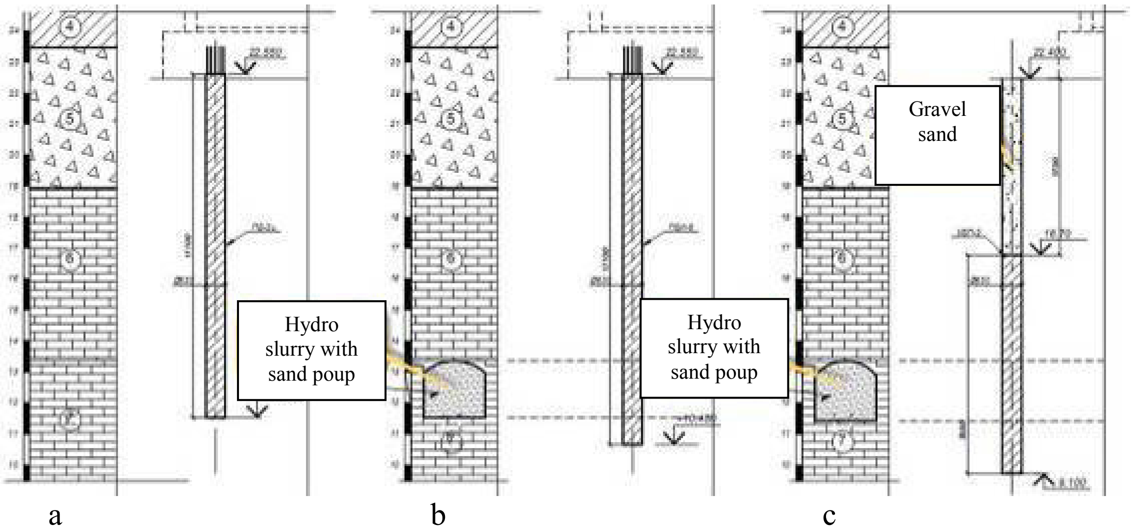


Figure 5 – Scheme of location of excavations and pile field
(fixing of workings in the area of the 3rd section is not shown conditionally)



a b c

Figure 6 – Scheme of piling:
a – working pile outside the working area;
b – a working pile that falls into the area of development; c – a gain booster;
EGE-4 – hard clay; EGE-5 – limestone hewn to the bottom, gravel, hardwood, with clay filler;
EGE-6 – limestone, slab, cracked, low strength; EGE-7 – limestone cracked, low strength

Their function is the perception of tensile forces arising from tangential stresses outside the cut, and the transfer of compression effort to an array of soil higher and lower than workings due to their work on the lateral surface. Underground hydrocarbons with sand puddings and subsequent «tightening» of their roofs with cement-sand mortar.

It is accepted in designing a solution tested by simulation of ITU in spatial formulation. In this model of soils and their parameters were selected on the basis of their field tests. In order to obtain the maximum possible deposition, including uneven, in the simulation of the stress-strain state of the pallets, the characteristics of non-limestone, and clay filler were set. The simulation results of the maximum total summation and the roll of sections of the building did not exceed the allowable values of values.

An imprint of soil behavior is the elastic-plastic model with the Mohr-Coulon strength criterion. For concrete, a linear elastic model is used. Interface (interface strength) was used to distinguish between the elastic behavior of the pile body, where small displacements, and the surrounding soil massif, where possible plastic behavior with the Coulomb-Mora strength criterion, was used. This is done to avoid the appearance of peak stresses and deformations that do not have a real physical meaning. Spatial rigidity of all structural elements was calculated in accordance with the adopted design decisions of the building (Fig. 7).

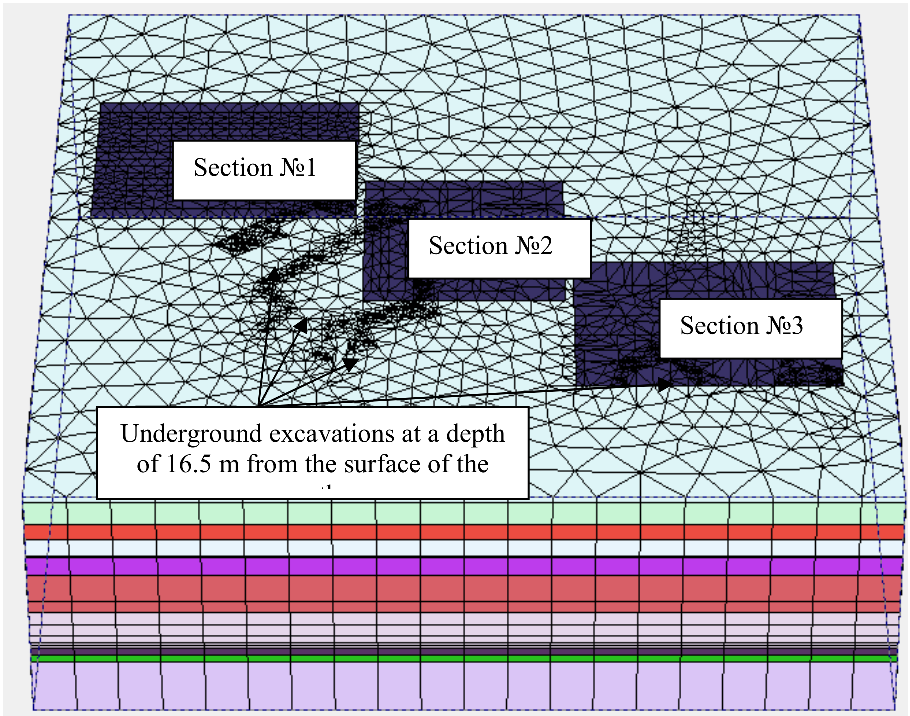


Figure 7 – Spatial design scheme for mutual influence modeling of foundations and foundations with underground workings of sections №1, №2 and №3

The problem was solved step by step:

1) gravitational loading of the calculated area with underground workings by the own weight of the soil and modeling of the initial VAT of the array;

2) excavation of the pit, water-curing of the cavity of the production spot, the device of piles and foundation plate, as well as the simulation of loading from the erection of section number 1, Fig. 8;

3) excavation of the pit, watering the cavity of the manufacturing spot, the device for piles and plates, modeling the load from the erection section number 2, Fig. 9;

4) excavation of the pit, watering the cavity of the production spot, the device of the pile and foundation plate, as well as modeling the load from the erection of section number 3, Fig. 10

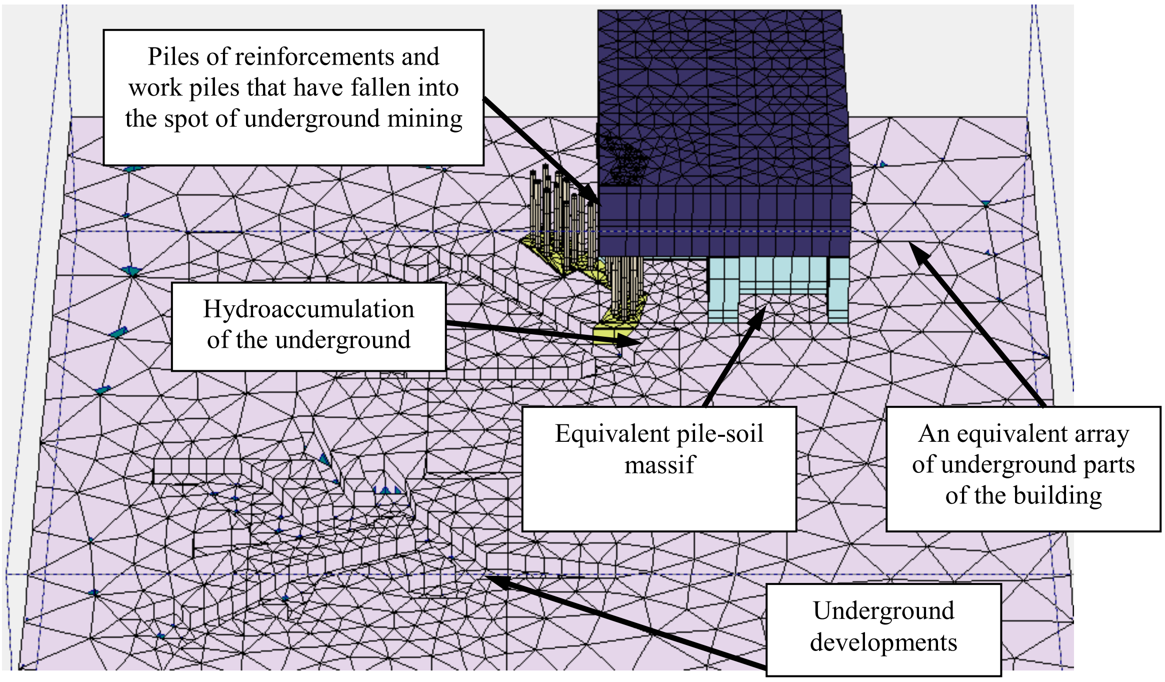


Figure 8 – Estimated spatial CE scheme of the second stage of modeling with the switched off clusters of EGE-1 ... EGE-5

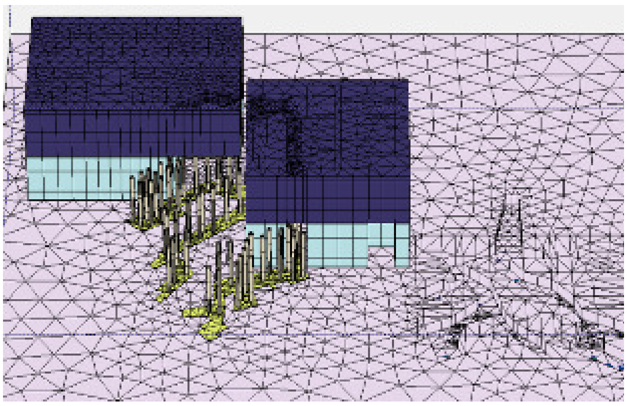


Figure 9 – Calculation scheme
(The second stage of modeling)

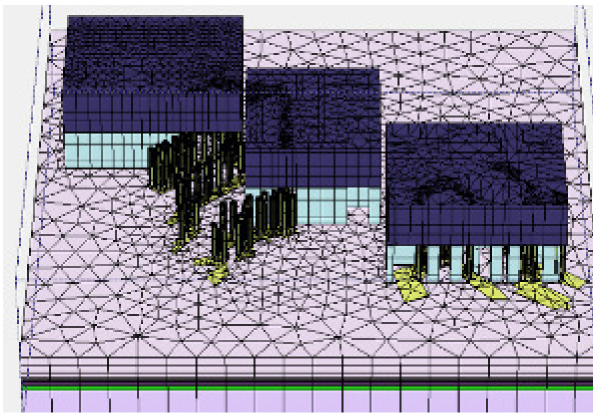


Figure 10 – Calculation scheme
(The third stage of modeling)

The maximum vertical displacement of the foundation of the pile-and-slab foundation was obtained after the construction of Section No. 1 – S = 9.9 cm. The base plate slope towards the underground mine was less than i = 0.0008.

The maximum vertical displacement of the base after the construction of Section No. 2 is S = 8.6 cm (Fig. 11) with the foundation plate sloping – less than i = 0.0006. Additional drafts of Section No 1 from the construction of Section No. 2 will be about SD = 1.5 cm with the plate heel – i = 0.0016. The maximum vertical displacement of the base from the construction of Section No. 3 is S = 9.9 cm (Fig. 11) when the slab is sloping towards the underground mine – to i = 0.0008. The additional drafts of Section No. 2 from the construction of section No. 3 are about SD = 1.8 cm with the plate heel – i = 0.0016. Effects of the construction of section number 3 on section number 1 will not be. The maximum total precipitation and roll does not exceed the maximum permissible values Su = 18 cm, iu = 0.005. As a result of the numerical analysis, the sediment values obtained are overestimated, which indicates the need to verify these data by full-scale geodetic observations.

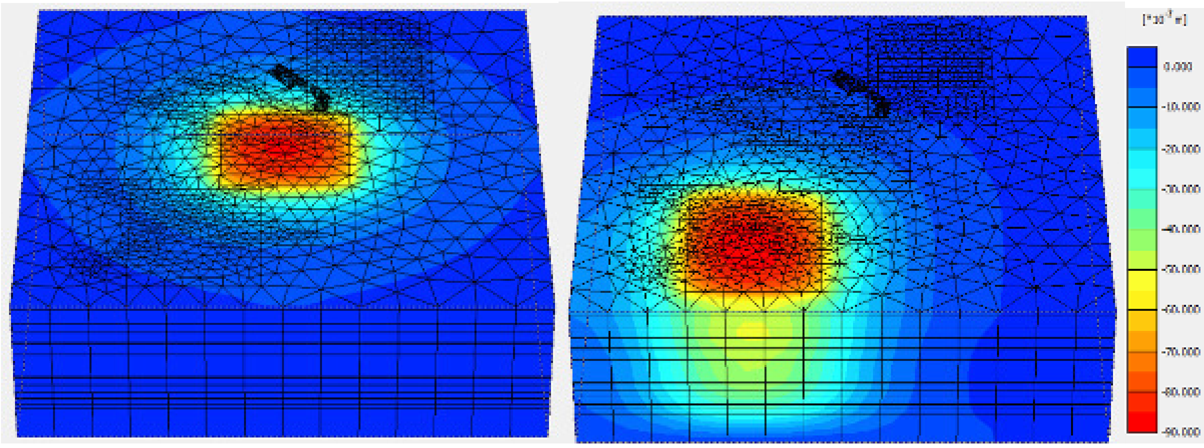


Figure 11 – Vertical displacement isopoles for 3D mesh after the construction of the second (left) and third (right) sections

Geotechnical monitoring in the process of construction of the complex was organized to verify the results of the calculation at the facility (Fig. 12).

The following results were obtained: the average subsidence of the first section is 9 mm (construction work performed on 98%); average settling of the second section – 10 mm (works executed on 85%); the average subsidence of the third section is 12 mm (works performed on 85%); The average subsidence of the fourth section is 10 mm (work performed on 80%, Fig. 13).

In fig. 14 the schedule of settling the foundations of the foundations of section №4 is given. The settlements by geodetic monitoring are similar to those simulated using high values of the deformation module IGE-6 ... IGE-8 ($E6$ = 100 MPa, $E7$ = 50 MPa, $E8$ = 500 MPa).

Figure 12 – General view of the frame of sections of the residential complex

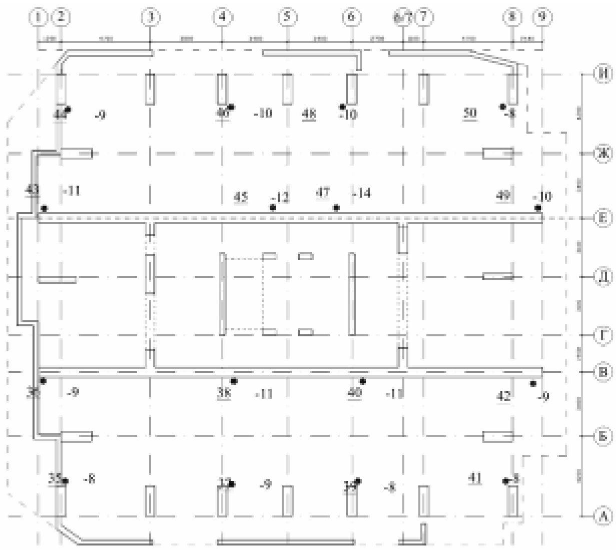


Figure 13 – Results of observations in the 4th section
(44 – the number of the mark,
9 – the value of settling in mm)

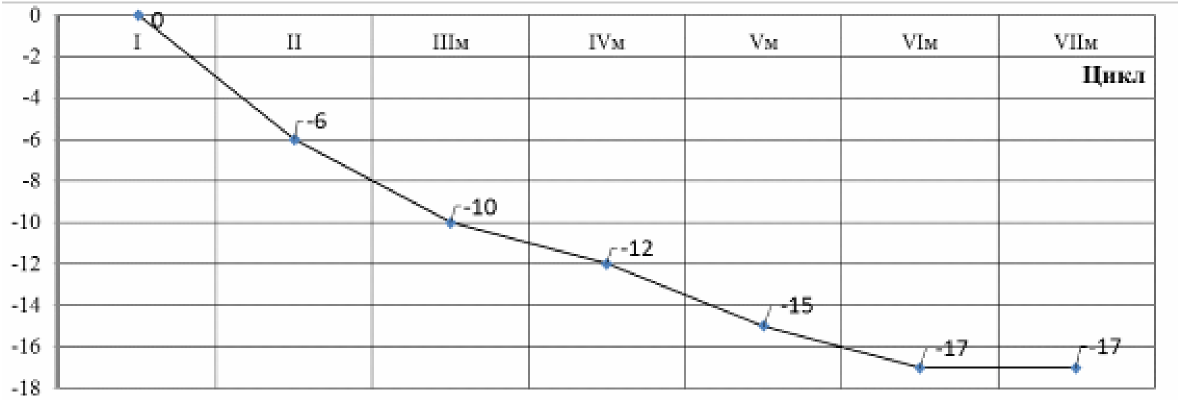


Figure 14 – Schedule of settlements of the foundation of the foundations of section №4

So, **conclusions**, from the analysis of the results of residential complex construction monitoring, it was found that the solutions adopted in geotechnical design have a sufficient level of reliability. Therefore, after additional analysis of the calculation scheme of Section 4, it was allowed to build it on 2 floors above (22 floors instead of the project 20) without changing the constructive decisions of the foundations.

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ENGINEERING PROTECTION ECO-SYSTEMS TERRITORIES ON THE BIOSPHERE COMPATIBILITY PRINCIPLES APPLICATION

The article is devoted to the solution of the actual scientific and applied problem issue - the search for organizational and technological solutions for biosferous construction on the example of marine and river coastal areas engineering protection in Ukraine. The main causes of imperfect activities in the field of coastal areas protection are: insufficient considering natural processes laws in the coastal zone of sea, reservoirs and rivers during the formation of design decisions; work incompleteness andcoastal protection and coastal regulating structures formation incompleteness in local complexes that fully cover coastal natural systems where there is a high level of natural processes interconnections that do not ensure their project effectiveness. It was substantiated that objective of shores engineering protection ecosystems increasing objective reliability is the most rational damaging construction of the gabions.

***Key words:** technological processes; biosphere compatibility; organizational and technological solutions; construction production*

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ЗАСТОСУВАННЯ ЕКО-СИСТЕМ ІНЖЕНЕРНОГО ЗАХИСТУ ТЕРИТОРІЙ НА ПРИНЦИПАХ БІОСФЕРОСУМІСНОСТІ

Стаття присвячена розв'язанню актуального науково-прикладного проблемного питання – пошуку організаційно-технологічних рішень біосферосумісного будівництва на прикладі інженерного захисту територій морського та річкового узбережжя в Україні. Основними причинами недосконалості діяльності в галузі захисту прибережних територій є: недостатнє врахування закономірностей природних процесів у прибережній смузі моря, водосховищ та річок при формуванні складу проектних рішень; некомплексність ведення робіт та незавершеність формування берегозахисних та берегорегулюючих споруд у локальні комплекси, що повністю охоплюють берегові природні системи, в яких існує високий рівень взаємозв'язків природних процесів, що не забезпечувало їх проектну ефективність. Обгрунтовано, що метою з підвищення надійності роботи екосистем інженерного захисту берегів найбільш раціональною утрумуючою конструкцією являються габіони.

***Ключові слова:** технологічні процеси; біосферосумісність; організаційно-технологічні рішення; будівельне виробництво*

Introduction. It is well known that the construction of any object is considered as a building system – a set of construction process all stages and its participants, which has an object-oriented direction which is implemented under conditions of environment established factors influence. The analysis of international and state programs of construction biosphere compatibility allows to distinguish the main global trends in the protection of the Earth biosphere, in particular, in construction [1 – 5]:

A. Reduction of greenhouse gas emissions (emission limitation, use of renewable energy sources – solar, wind, water, energy of tides, etc.);

B. Implementation of energy-saving technologies and equipment during construction, operation and liquidation of buildings and structures (energy saving throughout building life cycle);

C. Implementation of resource-saving technologies for the implementation and mechanization of construction processes based on the principles of purposeful management of material elements properties during their processing (the so-called «nanotechnologies») and on the construction production informatization principles (logistic support of construction with the complex of building processes robotization and creating artificial structures processes);

D. Development and implementation of environmentally friendly technologies, including the construction industry, based on the latest phenomena and processes of non-waste processing of labour objects to construction products (such areas of construction and architecture as «biotechnology», «bionics»);

E. Preservation of biodiversity and natural ecosystems (atmospheric air, land and oceans, geological environment and fertile soil, flora and fauna protection).

In EU countries, the applicant (developer) who submitted a tender, which includes construction and technology solution «biosphere compatibility» requirements, receives a significant advantage, along with other competitors. In these countries, biosphere compatibility prioritizes are even the criterion of «profitability / rationality of estimated expenditures».

In our country so far there has not been practice of such preferences, as well as effective mechanisms for enhancing the motivation of construction participants to involve the principles of biosphere compatibility in the development of architectural and construction solutions. This tendency forms conflicting requirements and criteria for evaluating projects to create new products and services. In such conditions, management innovative mechanisms of construction projects and programs based on the investment-building cycle modernization and construction organization system based on the principles of biosphere compatibility acquire a special significance.

The latest sources of research and publications analysis. Recently, there were attempts where was a new concept – the biospheric compatibility of construction. Authors of scientific and technical developments and real projects, namely O. A. Tugay [5], D. A. Kramer [6], D. B. Odly [7], T. Yu. Bystrov [8], O. V. Demidova [9], V. V. Savyovsky [10], I. P. Boyko [11] and others under biosphere compatibility mean local elimination of the consequences of previous contaminations with the simultaneous objects purpose change – reconstruction or industrial and civil purpose, urban development existing objects deep modernization.

Selection of previously unsettled parts of the general problem. In contrast to the approach to eliminate the consequences of previous contaminants with the simultaneous change in the purpose of the objects, in this study, biosphere compatibility of construction principles application is considered as a purposeful improvement of construction production, eliminating the causes of its negative impact on the environment in the projects of coastal areas engineering protection and is based on the use of environmental engineering protection systems with the use of natural materials and the consideration of natural processes in the coastal zone during design decisions formation.

The purpose of the article is to develop an innovative platform for the application of eco-systems for engineering protection of marine and river coastal areas (as the interaction of protection structures against the effect of geodynamic processes with a soil massif) on the principles of biosphere compatibility.

The achievement of this goal requires the search for an organizational and technological solution based on the use of environmental systems of engineering protection with the use of natural materials, deepened underwater structures that extinguish wave energy, protect the coastal zone and the environment.

Main material presentation. Geodynamics is a field of Earth sciences that examines geological processes in terms of operating forces. In order to describe the occurring processes, the continuous medium approach is very important for understanding a wide range of geological problems, since it allows us to apply well-developed physical and mathematical methods of the elastic deformations theory, the flow of ideal and viscous fluid, heat and mass transfer, etc.

From the standpoint of building eco-systems biosphere compatibility of territories engineering protection, the focus of such projects should be given globally.

As the main man-made unit, a finished construction object is adopted where the ultimate set of factors is determined as having significant impact on the ecosystem.

Solving these issues, modern theoretical developments on coastal processes regulation, simulation of coast strained-deformed state, energy efficient engineering protection use, etc. are of great importance.

Outbound processes can be predicted. It requires careful engineering, geological and hydrological research. The following conditions must be considered for the forecast of landslides: slope presence and a sufficient mass of rocks, which has a tangential direction to the surface.

Today, several methods for predicting landslides exist:

– long-term (for years);

– short-term (for months, weeks);

– urgent (for hours).

For long-term forecasting, the rhyth method is used, which is based on rainfall and other meteorological elements.

Short-term and urgent forecasts are based on geodynamic measurements use and predictive model of the shear process construction using regression analysis based on their basis, while considering slope stability which is determined by retaining forces and displacement forces ratio.

The methods of forecasting landslide phenomena include:

a) settlement (engineering calculations under simplified schemes);

b) simulation (numerical modeling with variable parameters);

c) the method of analogies (or comparative-geological) - comparison of slope main characteristics (geological structure, rock strength, height, steepness, etc.) with similar characteristics of other slopes which known stability.

d) the historical-geological method - comparison of slope actual conditions with its earlier conditions.

e) the earth masses balance consideration method- for prediction of rotation and extrusion repeated displacement.

e) the influence of factors consideration method- processes that change the size of the coefficient of slope stability.

g) observing the precursors of landslide processes method (landslides) – the growth of deformations, the origin or sources disappearance, sound phenomena, etc.

Most of the potential landslides can be prevented if timely measures are taken at the initial stage of their development.

Thus, an increase in the water cut in the Dnipro River in the upper peaks of each reservoirs has led to sharp and significant rise of the erosion corresponding local bases. A new coastline with a total length of about 3.5 thousand km was formed. A third of the new waterfall perimeter in the reservoirs is actively destroyed by denudation, especially abrasive and erosion processes, and needs protection [12].

Settlements and economic objects located along the reservoirs coastline, after filling each of them fall into negative processes and phenomena activation zones. These zones belong to territories with a special use mode. In the legal and technical literature they were called «zones of prohibition or restriction of new capital construction», «zones of buildings removal and population mandatory relocation». Use of such territories is possible only on the condition of liquidation or limitation of adverse processes in the reservoirs coastal zone or their planned management. Particularly relevant these issues are within the settlements [13, p. 26].

Coastal zones of reservoirs within the cities protect from water harmful effects (flooding, destruction of coastal land). The most important (and, consequently, the most expensive) shore-based structures are vertical sloping and stepped quays type strengthening , walls berthing and retaining, dams with collapsed drainage, etc.

Protected buildings are included in the complex of measures for the rational use and shores protection, which are united by the term «measures for the engineering protection of shores and coastal areas from harmful effects of water reservoirs». The realization of this complex of measures in the territories of settlements and economic objects refers to «engineering training of the territory». It minimizes manifestations of the coastal process (transit flows of water and streams of sediment, standing waves, erosion of the bottom at the shallows and accumulation of sediment), or contributes to the transformation of the abrasive or erosional shore into an analogue of the denudation shore in rocky rocks.

Outside the boundaries of settlements and economic objects, coastal measures in reservoirs are limited, as a rule, administrative and organizational (regulation of coastal areas use) and agro-forestry (foraging and afforestation of coastal areas, biological attachment of slopes and landslides). Engineering protection of shores and coastal areas in this case is carried out only in special cases (protection of valuable forest and land, natural monuments, recreational facilities, etc.).

Such problems current level consideration involves computer simulation of interaction processes in the system «foundation – design of engineering protection» sea coast and river banks of [14 – 18]. Significant progress recently achieved in hydrodynamics, is associated primarily with the development of mathematical modeling methods. Modern mathematical modeling of each physical process involves solving several tasks:

1) mathematical model of a particular physical process formulation (or group of processes);

2) formulation of the algorithm for solving this problem;

3) display of numerical algorithm on the architecture of the computer system used for calculations.

All these tasks are closely linked. Before exploring mathematical methods of any natural processes, it is necessary to highlight those basic principles and decisive moments that allow to describe course quantitative and qualitative terms satisfactory and simply and to create a model. The actual structure of the soil base is much more complicated than the simple objects that are available for research by the methods of modern theory. Hydrodynamic phenomena are described by equations based on the laws of mass conservation and number of motion, state equations and the thermodynamics laws. All these equations are approximate.

Solving a series of tasks for random processes of any kind is difficult. When considering geodynamic processes with time-varying probable state changes, one can specify a particular method of research – a direct dynamic method. This method is oriented to the study of orthogonal functional bases in the space of functions with limited energy, which corresponds to the physics of the results obtained on the one hand and contributes to the emergence of a special expression that describes the geological phenomena in the finite period of time. The nature of the resulting relations is the following: matrix representations of linear operators are used as media of process information. In these cases, it is possible to involve numerical simulation procedures that allow implementation at the level of modern computer programs. Of particular interest is the number of circumstances that are associated with the weakening of the models time dependencies which in the field of operator representations are reduced to parametric relations. In this way, it is not only possible to solve tasks from a larger class, but also it is possible to accumulate information, which is especially important for geological applications.

The direct dynamic method allows directly, by linear equation, to write the explicit expression of their solution in a closed form, using the symbolic matrix, which has obvious theoretical and practical significance for solving the problems of analysis, identification and synthesis. All tasks solved by simulation methods in the case of linear non-stationary systems are solved by this method without simplifying the system mathematical description. The form of algorithms does not depend on the type of the functions basic system, which provides the universality method. As for type of the basic system it depends only on the numerical expressions of the systems characteristics of the systems. The advantage of direct dynamic method is its correctness.

Mathematical methods can be applied to experimental and empirical material in geology in different ways. For hydrogeology, their main application is to identify and forecast the processes of water exchange. Proper water exchange estimates considering the maximum number of factors affecting these processes, based on a well-developed theory, will clarify the role of groundwater, for example, in erosion processes on the coast of the seas and rivers.

A sufficiently promising way of fixing slopes in our time is the use of gabion structures (Fig. 1), which can be in the form of: box-shaped (1, 2, 3) or cylindrical structures (G), so-called «Reno» mattresses.

Gabions are a double torsion metal grid construction, which is filled with any stone material, and its weight and characteristics are in accordance with the static and functional requirements of the structure. Usually, as filler, large rubble, pebble, or quarry stone are used. The size of the filler should be larger than the mesh of the cell so that it does not fall out of the gabion. At the same time, large stones are placed along the edges, and the middle is filled with smaller ones. The space between the rocks is covered with soil, which acts as connecting material.

Suspended walls of gabions can be massive outline (gravitational walls) and fine lines (semi-gravitational walls). They may be low: <1,5 high:> 1,5, where H – visible wall height, m; – effective width. The face of such walls can be arranged: step (vertical or angled to vertical) or smooth (vertical or inclined).

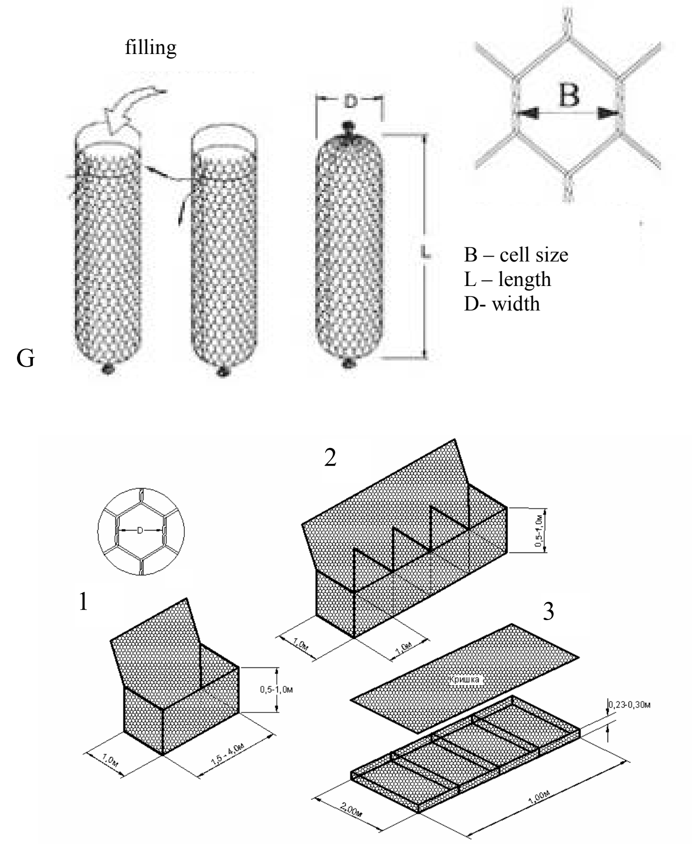

Figure 1 – Structures of slopes engineering protection artificial eco-systems

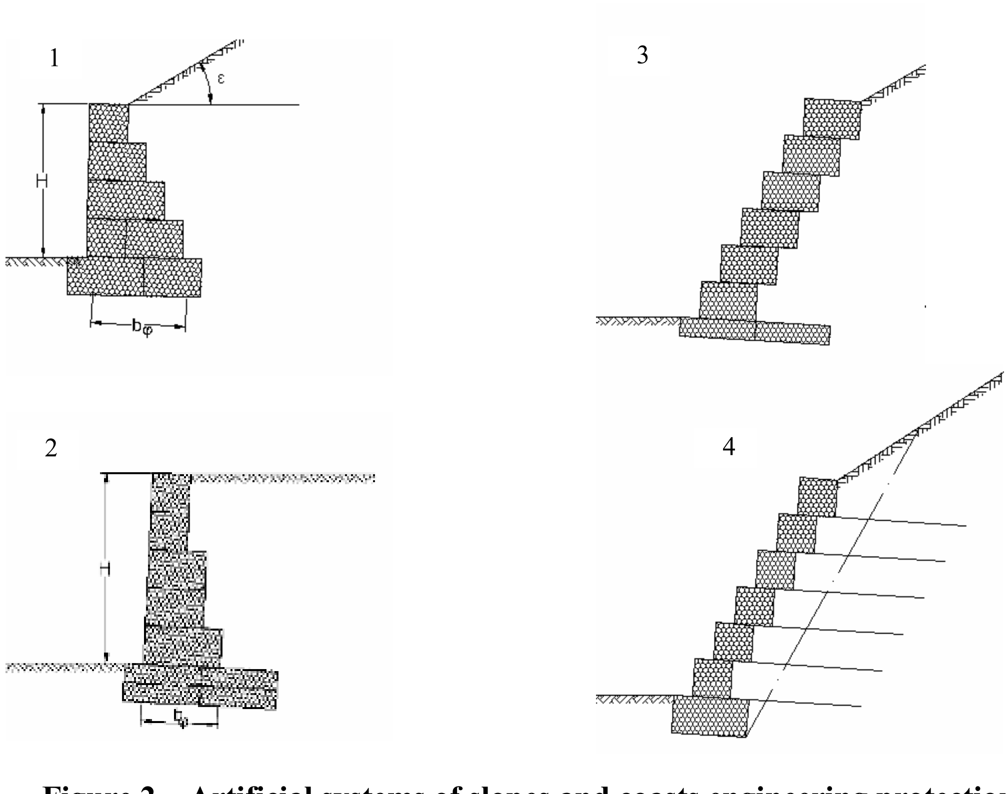

Figure 2 – Artificial systems of slopes and coasts engineering protection schemes:
1) gravitational wall; 2) half-gravity wall; 3) step; 4) thin wall with anchoring

Gabions are mainly used for erection of retaining walls, motor and railroads embankments strengthening, river and sea coast protection, landscape works, stabilization of soil erosion and conservation of soil. Due to very good hydraulic characteristics, they are used for coastal reinforcement of rivers, in the construction of spillway dams. PVC-coated gabions are used to protect the seaside coast. By the time, gabion structures are merged with the environment and become part of the natural landscape. They acquire the maximum strength and stability due to natural processes, since over time there is accumulation of soil particles between the stones, which contributes to the formation of vegetation on the surface of the gabions. The most rapid growth of plants becomes in the presence of horizontal terraces between each tier of gabions. Due to the porous structure of gabions, a high permeability of gabion structures for water and air is achieved.

Restoration of the territory eco-systems natural state, in particular the sea coast, on the principles of biosferous-compatible construction will ensure the ecological security of the regions, preservation of water resources and will have the corresponding social significance as they will become recreational zones. In order to achieve this goal, it is necessary, first of all, to conduct a series of studies to determine the impact of sea abrasion on the stability of the composite area, especially in Odesa and Odesa region, where shifts and landslides are widespread, mainly forest soils, which, due to moisture, on the one hand and the actions of the sea, on the other hand, predetermine the deformation processes of the soil massif, which indicates this issue study lack, and also points to the need to use preventive measures to ensure the territory stability.

Conclusions. To preserve the coastline it is necessary to develop engineering protection ecological systems program of natural and artificial seas, reservoirs and rivers coasts. Gabions should be classified as rational retaining constructions of slopes (coastline). Using gabions as shore protectors, in combination with biological fixation (the formation of vegetation on the surface of gabions) meets the requirements of reproduction and conservation of natural ecosystems and does not violate the aesthetic value of coastal landscapes. Under such conditions, engineering systems of coast protection, created on the principles of biosferous construction, act not only as the abiotic factors of water and adjacent coastal ecosystems, they themselves are also formed in the form of a biotic factor - coastal biocenosis.

In order to improve the reliability of shores and rivers ecosystems protection, further research should be aimed at obtaining information on the stress-strain state of the earth mass under the influence of geodynamic processes and technological influences, which can be accomplished by such systems numerically simulating with the use of powerful computers and modern calculation- software complexes (for example, ASND VESNA).

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EXPERIMENTAL RESEARCH METHODOLOGY OF FULL-SCALE STEEL AND CONCRETE COMPOSITE CABLE SPACE FRAME PROTOTYPE

The article describes the main features of experimental studies methodology of new space systems constructive solutions effectiveness. Research methodology is developed by the example of full-scale prototype of the steel and concrete composite cable space frame, which consists of space steel and concrete composite modules and flexible bottom chord. Methodology for study stress-strain state of the prototype by testing in the uniform load applied to nodes is developed. It also provides the nodes displacement definition of the prototype. According to the developed technique, stressed state of full-scale prototype of the steel and concrete composite cable space frame was studied with tensionmetric and photogrammetric methods. Equipment and devices arrangement schemes for measuring deformations and displacements of the full-scale steel and concrete composite cable space frames prototype are presented.

***Keywords:** steel-concrete composite material, plate, modulus of elasticity, stress-strained state.*

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МЕТОДИКА ЕКСПЕРИМЕНТАЛЬНОГО ДОСЛІДЖЕННЯ ВЕЛИКОГАБАРИТНОГО ЗРАЗКА ПРОСТОРОВОЇ СТРУКТУРНО-ВАНТОВОЇ СТАЛЕЗАЛІЗОБЕТОННОЇ КОНСТРУКЦІЇ

Наведено головні особливості методики експериментального дослідження ефективності конструктивного рішення нових просторових систем. Сформульовано методику дослідження деформативності великогабаритного зразка просторової структурно-вантової сталезалізобетонної конструкції, яка складається з просторових сталезалізобетонних модулів та гнучкого нижнього пояса. Розроблено методику дослідження напружено-деформованого стану великогабаритного експериментального зразка просторової структурно-вантової сталезалізобетонної конструкції шляхом випробування його на дію рівномірного навантаження, прикладеного у вузлах з'єднання, а також методику визначення прогинів у вузлах дослідної просторової структурно-вантової сталезалізобетонної конструкції. Для дослідження напруженого стану просторової структурно-вантової сталезалізобетонної конструкції застосовано тензометричний і фотограмметричний методи. Наведено схеми розміщення обладнання та пристосувань для вимірювання деформацій і переміщень.

***Ключові слова:** сталезалізобетон, плита, модуль пружності, напружено-деформований стан.*

Introduction. The development of construction industry needs to change and implement the latest designs. The obligatory condition for successful implementation of design concepts into real construction sector is their researching and compliance with today's requirements. The structure that completely satisfies these requirements is the steel and concrete composite cable space frame. The originality of this concept lies in combining various elements, which effectiveness is determined by the terms of their location in structure.

Analysis of recent sources of research and publications analysis showed that among composite structures often steel and concrete composite shells are highlighted, which specific is a combination of concrete slabs with steel rod elements for partial discharge slab. In such constructions, plates could be unloaded by means of steel rods. In [1] it is investigated the strength and deformability steel and concrete composite structure. The top chord of the structure is made of steel T-section beams, which are combined to monolithic concrete slab with anchors.

In [2] it is proposed improved design, consisting of modular steel elements, top and bottom chords and reinforced concrete slab. This design differs from the previous ones by way of ensuring collaboration steel component and reinforced concrete slab.

Considering the idea to combine the plates and rods with collaboration in the structure by the new way is the original and distinguishes the proposed structure among existing as a new type of roof and allows deny drawbacks inherent counterparts. The concept of the proposed solution is a synthesis of experience and new developments in which the modular elements, complexity and difficulty of manufacturing technology are used, which assembly and installation is less than that of counterparts [3].

Efficiency constructive solution space structures and their elements were investigated and confirmed earlier [4–7].

Highlighting of the general problems parts unsolved earlier. The analysis of previous works has showed that there are no studies of the proposed structure peculiarities on large-scale prototype.

The problem formulation. The task is to develop steel and concrete composite cable space frames experimental research methodology.

The main material and results. The experimental research methodology includes testing of steel and concrete composite cable space frame prototype that has span of 5.3 m under temporary load (Fig. 1).

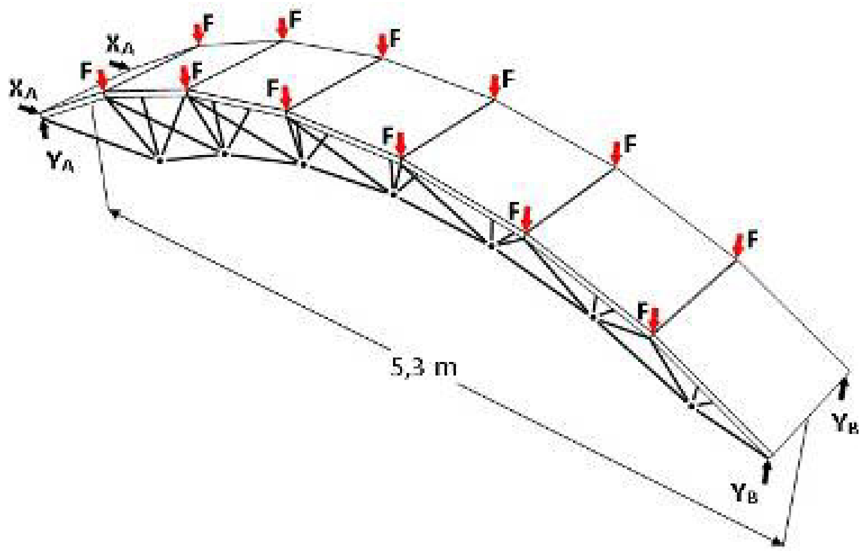


Figure 1 – Scheme of experimental research of steel and concrete composite cable space frame prototype

Steel and concrete composite cable space frame prototype testing was conducted in academic laboratories of Poltava National Technical Yuri Kondratyuk University. It was done only after it was finally installed and 28 days after concreting. Because of the large prototype size, the use of standard laboratory power equipment, including presses for the creation and application of temporary nodal load on the prototype was impossible. Therefore, prototype loading was performed using steel cargoes. Each of these cargoes had the form of a solid short cylinder and weight 42 kg. For applying force on the prototype, traverses were used, consisted of a crossbar and two rods, having cargoes on them (Fig. 2).

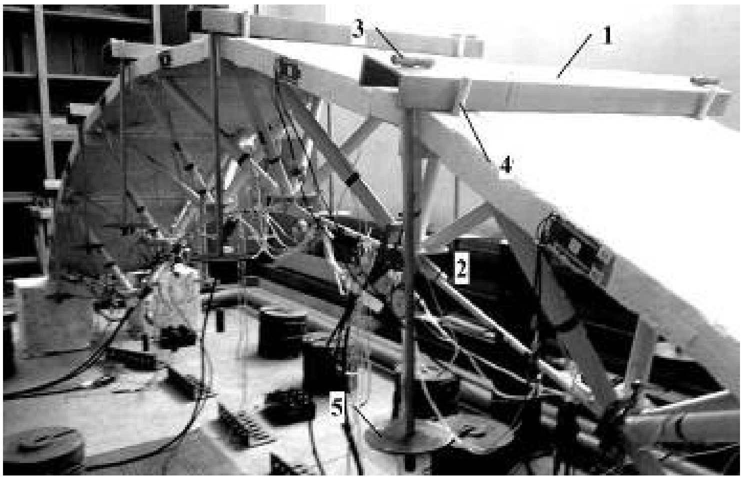


Figure 2 – The traverse to loading the prototype:
1 – crossbar; 2 – rod; 3 – hinge; 4 – fuse; 5 – plate

The traverses were set on top chord of the prototype at nodes. To prevent traverses displacement or movement, fuses in nodes were arranged (Fig. 3).

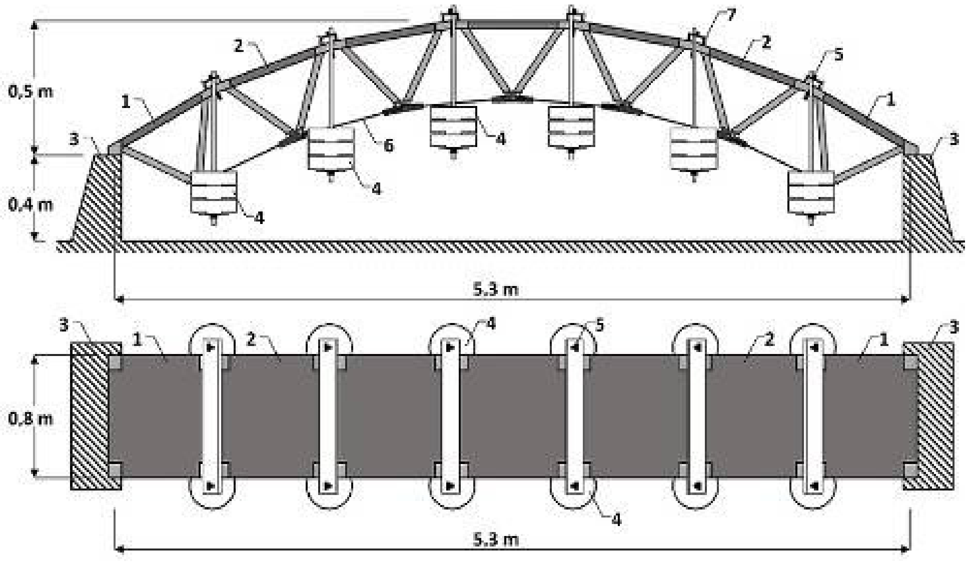
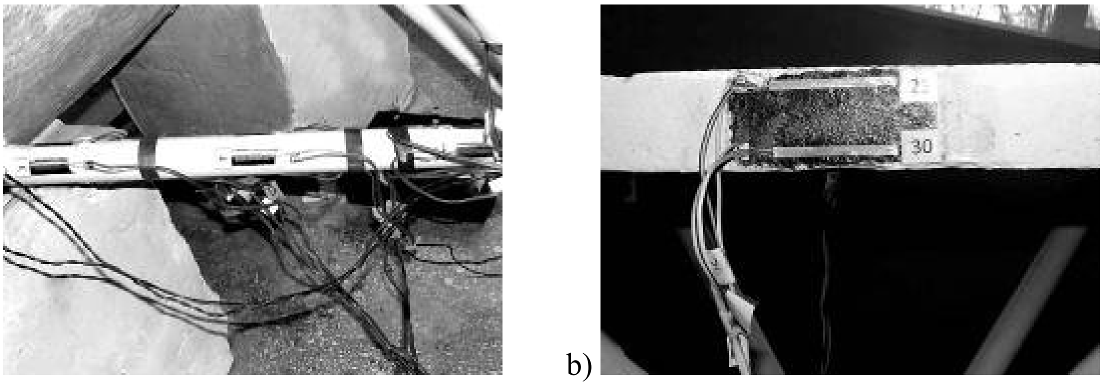


Figure 3 – The prototype test scheme:
1 – support module; 2 – span module; 3 – support; 4 – the cargo;
5 – traverse; 6 – lower chord; 7 – fuse

Full-scale prototype of the steel and concrete composite cable space frame testing was carried out in several stages. For measuring strain in the cross-sections of the prototype were used wire strain gauges resistance were used, which had the backing paper. They were made of glue BF-2. For measuring strain of steel elements of the prototype, strain gauges with a base of 30 mm (type 2PKB-30-200HB) were used, which have tensosensibility factor of 2.19 and resistance 201–201.49 Ω. For measuring strain of concrete elements of the prototype, strain gauges with a base of 50 mm (type PKB-50), which have tensosensibility factor of 2.21 and resistance 310.3 Ω ± 0.3 Ω were used. Strain gauges of each type were from the same batch that had been tested for suitability for using accordance to national code. Wire strain gauges were attached to the surface of the prototype on certain places with glue BF-2 (Fig. 4).



a) b)

Figure 4 – A general view of placing strain gauges on steel (a) and concrete elements (b)

To quality joining strain gauges to the surface of steel and concrete elements of the prototype and for their correct work during testing, these areas were cleaned to smooth state by grinding equipment and devices with varying degrees of abrasiveness, then skim alcohol solution, and coated with several thin glue layers.

Indications of strain gauges were taken with the equipment «AYD-4». Connection between strain gauges and the device was provided via copper multicore wires of cross-section 0,2 mm² and length up to 6 m. Considering that there were a lot of strain gauges that were used to study of stress-strain state of the prototype, in this case there was the notion «quantitative strain gauges», so for measuring of strain from all strain gauges, single pole switch was used. Except the measurement of the prototype elements strain of steel and concrete composite cable space frame, measurement of nodes displacement by mechanical and photogrammetric method was done (Fig. 5).

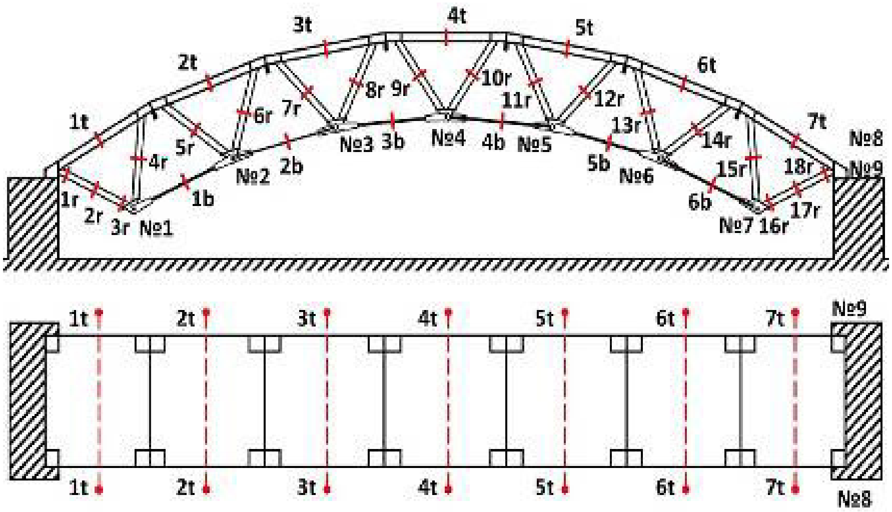


Figure 5 – Number of nodes and sections of the prototype

For measurement vertical displacements of the prototype, the equipment was set on each node of the bottom chord (Fig. 6).

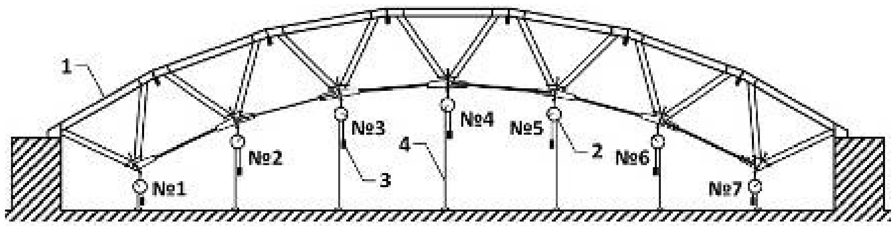


Figure 6 – Placement of instruments for measuring displacements:
№1, №2, ..., №7 – numbering devices; 1 – research design; 2 – device 6PAO;
3 – the cargo; 4 – steel wire Ø 0,25 mm

In addition, horizontal displacement was determined. In particular, horizontal displacement of nodes №8 and №9 were measured (Fig. 7).

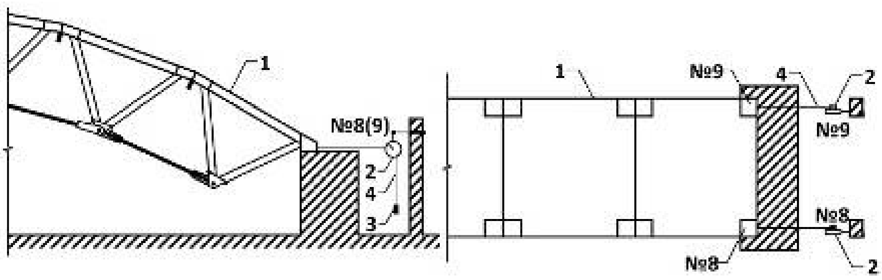


Figure 7 – Placement of instruments for measuring displacement of support nodes:
№1, №2 – numbers of devices; 1 – the prototype; 2 - device 6PAO;
3 – the cargo; 4 – steel wire Ø 0.25 mm

In addition, displacement of nodes №1 and №7 were measured with the devices 6PAO. For this at node №1 on the cap of bolt was set hinging to which, steel wire was attached, and at node №7 block via it wire was moved and attached to device. For a more detailed study of the prototype strain state, optical recording method of movement was used as photogrammetric method (Fig. 8) [8].

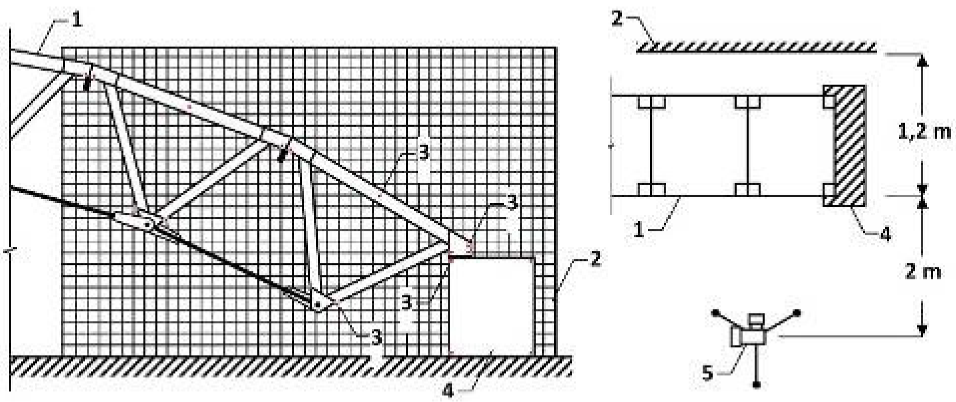
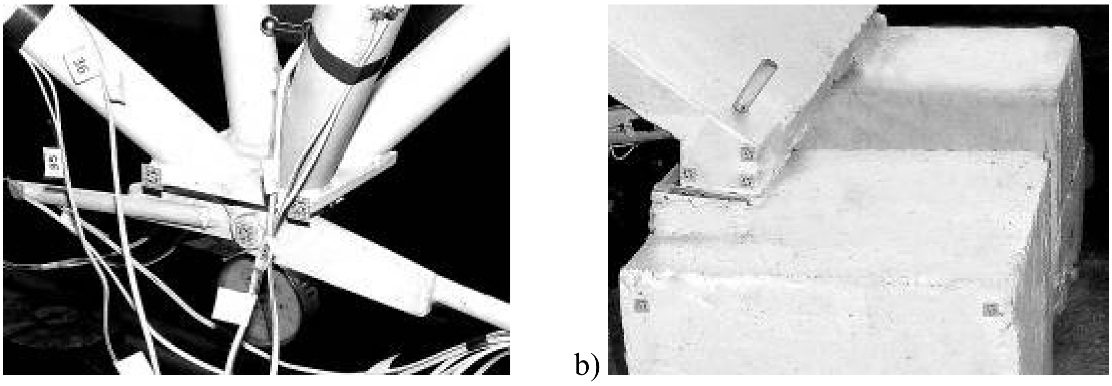


Figure 8 – Placement of instruments for photogrammetric measurement of displacement:
1 – the prototype; 2 – sheet with marks; 3 – marks; 4 – support; 5 – digital camera

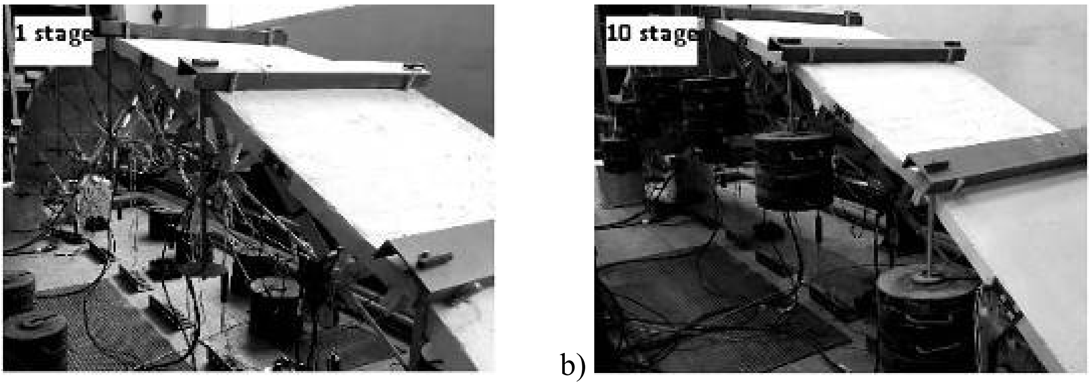
This method is based on digital pictures analysis at each stage of loading. To perform this, a special sheet that had a control grid was made. The sheet was arranged in front of the camera on the other side from the prototype. Also, on the prototype special labels were set. Special labels were set on both the top and bottom chords of the prototype (Fig. 9).



a) b)

Figure 9 – Special labels on the nodes of the bottom (a) and support (b) chord

In addition, marks were set on the support nodes; this was done to learn about their movements more widely. Also, it was enable to obtain the total value of the prototype nodes horizontal displacement and execute correct assessment of stress-strain state of the prototype. After setting up all the equipment and devices, tests were conducted in temporary load (Fig. 10).



a) b)

Figure 10 – Prototype of steel and concrete composite cable space frame prototype testing

To produce lattice and bottom chord of the prototype, seamless steel tubes, hot rolled round steel bars, rolled universal steel sheet were used. To produce the top chord, concrete mixture was used (tab. 1).

Table 1 – The composition of concrete mixture

Class for durability	Water-cement ratio	Consumption of material, kg/m³		
		Sand	Broken stone	Cement
C25/30	0,5	610	1217	383

Determination of the physical and mechanical properties of steel and concrete that were used to make the prototype, it was performed in accordance with applicable rules and regulations of national codes. Determination of the physical and mechanical properties of concrete, testing standard concrete samples of cubes and prisms with a hydraulic press PMM-250 was carried by compression.

The samples were stored in the same premises under identical conditions as the prototype. Testing standard concrete samples of cubes and prisms were performed simultaneously with the experiment. Before the samples test was conducted, visual inspection to detect external damage and defects was conducted. When the samples were viewed, damages that exceeding the permissible were not found. However, minor defects of geometry were identified, which was characterized by change in cross-sectional area to height. Overall quality of cubes and prisms standard concrete samples was satisfactory.

Conclusions. Experimental research methodology of full-scale prototype of the steel and concrete composite cable space frame was designed to obtain data that accurately describe the stress-strain state and prototype conduct under the action of symmetric load. Physical and mechanical properties of steel and concrete that were used to manufacture the prototype were standard and similar to those that commonly are used for the manufacture steel and concrete structures. Manufacturing technology of the prototype is similar to existing, but collecting and assembling methods are original in its execution. Instruments and accessories used in experimental research, allowed to obtain data that objectively describe the conduct peculiarities of the steel and concrete composite cable space frames full-scale prototype under load.

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ASSURANCE OF STRUCTURAL STEEL SURVIVABILITY BASED ON RESERVATION OF PRODUCTION FACILITIES CORROSION PROOFNESS

The paper deals with the task of selecting quality and reliability indices of means and methods of corrosion protection considering structural strength requirements. Systematized description of standard (basic, characteristic) impacts and representative values of negative corrosiveness factors is provided. For corrosion protection design, classification features of steel structures and their protective coatings based on criticality rating are specified. Design indices of structural steel durability are discussed. The developed methodology involves an analytical–experimental estimate of reliability and availability factors of corrosion protection. Logistical system has been generated for reserve planning of survivability of structure on the basis of corrosion proofness signs. A method is proposed for calculating compensation for corrosion losses when comparing competitive advantages of corrosion protection systems. The index of corrosion protection level is specified for managing process safety on the basis of risk reduction.

***Keywords:** reliability, structural and process safety, survivability, corrosion hazard, risk assessment.*

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ЗАБЕЗПЕЧЕННЯ ЖИВУЧОСТІ СТАЛЕВИХ КОНСТРУКЦІЙ НА ОСНОВІ РЕЗЕРВУВАННЯ КОРОЗІЙНОЇ ЗАХИЩЕНОСТІ ПРОМИСЛОВИХ ОБ'ЄКТІВ

В статті розглянуто питання обґрунтування показників якості й надійності систем протикорозійного захисту з урахуванням вимог резервування міцності конструкцій. Здійснено систематизоване описування нормативних (базових, характеристичних) впливів і репрезентативних значень корозійної агресивності. Для проектування захисту від корозії встановлені класифікаційні ознаки сталевих конструкцій та їх захисних покриттів за категорією відповідальності. Обґрунтовані проектні показники довговічності сталевих конструкцій. Розроблена методика включає розрахунково-експериментальну оцінку коефіцієнтів надійності та готовності. Сформована логістична система резервування живучості конструкцій за ознаками корозійної захищеності. Запропоновано метод розрахунку компенсаційних корозійних втрат при порівнянні конкурентних переваг систем протикорозійного захисту. Встановлено індекс рівня корозійної захищеності для управління технологічною небезпекою на основі зниження ризиків.

***Ключові слова:** надійність, конструктивна і технологічна безпека, готовність, живучість, корозійної небезпеки, оцінка ризиків.*

Introduction. Nowadays metal use in construction industry, building facility architectural expressiveness means improvement and methods of corrosion protection should support the long-term service life of structures. It should be noted that in present there are no effective mechanisms that would satisfy the economic approach to increasing the corrosion protection measures efficiency when assuring reliability and process safety of metal structures [1 -- 3].

Review of the latest research sources and publications. In the field of construction, process safety is linked with the regulation of approaches to preventing emergency situations on the basis of buildings and installations reliability program-oriented management methods. The concept of preventing progressive ageing of fixed assets is based on the process-based approach to the problems of construction facilities ensuring technological safety [4]. Technological safety is an important structural component of enterprise safety, which characterizes the system of measures to maintain serviceability, improve the performance of building, installation and utility structures, which have completely or largely exhausted their service life and which are a source of potential hazard with respect to facility technological functions in the course of modernization, refurbishment, and life extension. According to this approach the concept of corrosion risk hazard includes a specified certain condition or situation (threat) where the probability of damage increases due to the fact that given corrosion state or deviation from normal operation are a potential cause (threat) of hazard [5].

Definition of unsolved aspects of the problem. The developed standard is DSTU BV.2.6-193:2013 «Corrosion protection of metal structures. General specifications» is aimed to update the national legal and regulatory framework in accordance with modern objectives of building industry and harmonize this framework with European Union regulations in particular related to implementation and introduction of Directive 89/106/EU. Improvement of operational requirements considering modern scientific developments and provisions of international standards is aimed at corrosion protection of metal structures in civil and industrial engineering. The project was developed for the purpose of intelligent design of corrosion protection at all stages of metal structure life cycles.

According to the technical assignment, the objects of standardization are corrosion attacks, means and methods of metal structure corrosion protection, implementation of the process approach to reliability and quality control on the basis of ISO 9001. Draft standard specifies common design criteria and methods for assessing the indices of corrosion resistance and durability of metal structures and their protective coatings.

Improvement of corrosion protection quality involves the use of primary protection measures (requirements for corrosion resistance of materials for bearing and enclosing structures) and secondary protection (requirements for durability of protective coatings and special equipment of electrochemical protection) in accordance with operational rules [6]. Operating conditions of building units are defined as the effects of impacts or provided conditions on the specified level of facility specifications. Regulatory requirements are confirmed based on the limit state concept using computational models of corresponding range of characteristics of mechanical strength and resistance of steel structures and protective coatings in corrosive environments. Provisions of DSTU BV.2.6-193: 2013 are aimed to meet the industrial safety requirements prevent environmental and technogenic threats, reduce economic risks due to corrosion damages [7].

Problem statement. In the course of facility design reliability and structural safety are ensured via improvement of design codes, including the development of the basis, principles and methods of limit state design. Trouble–free operation of buildings and installations of structures (hereinafter referred to as structures) is linked to assuring process safety based on the methodology of total quality control.

Fixed assets depreciation high level and limited life span of structures pose a significant thread to process safety. Process safety is an important structural component of enterprise safety, which characterizes the system of measures for maintaining serviceability and improving performance of structures that have completely or largely exhausted their design life. Such facilities are viewed as a source of potential hazard in the course of their modernization (refurbishment), revamp, and service life extension.

The purpose of this paper is to substantiate the composition and structure of indicators determining corrosion state (*IDCS*), for program–oriented management of industrial facilities process safety.

The methodology of *IDCS*–based safety management comprises a process approach to choosing means and methods of corrosion protection (*MMCP*). Assurance of process safety is viewed based on considerations of «soft» conflict of the system survivability theory. The essence of «soft» approach in developing programs of reliability assurance (*PRA*) and analysis of survivability is in justification of solutions (subsystem *U*) considering information parameters of technical conditions (subsystem I) and material flows (subsystem *Q*) providing conditions for trouble–free operation (Fig. 1).

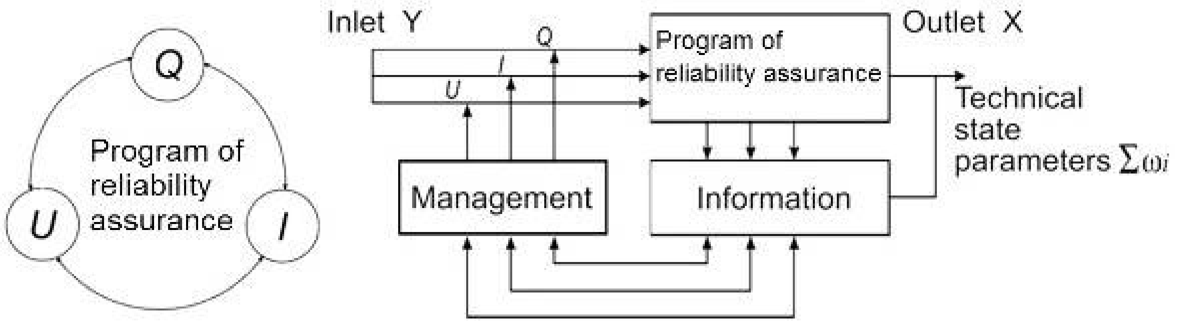

Figure 1 – Structural and organizational model of material (Q), information (I) and management (U) components for assuring process safety

Survivability is an important characteristic of structural capability to maintain a partial operable state under negative impacts in the presence of defects and damage. Thus, assurance of technological safety provides with formation of survivability assessment structural and organizational model considering *IDCS* of structures.

The concept of corrosion hazard includes *IDCS* or situation (threat) where the probability of damage increases. Thus, conditions are being created for logistical management and analysis of *PRA* structural solutions risks extending industrial facility life.

Works provisions development [5-9] has allowed proposing classification of technical and process risks signs under restoration of structure corrosion protection while maintaining and repairing facilities in their actual state (Table 1).

Upon characteristics of secondary protection (durability of protective coatings), structure service life is set on the basis of SCPSS quality indices analysis. Structural steel calculations for corrosion resistance, durability and repairability are carried out based on the limit states of the first and second groups considering *IDCS* (Table 2).

Corrosion protection reliability factors ($\gamma_{z\kappa}$, γ_{zn}) specify permissible deviations of strength, deformation and performance properties of structural members determined for a fixed design case and specified service life ($T_{n\gamma}$, year).

Block diagrams of reliability indices are represented as flow graphs describing changes in the corrosion state of structural steel considering structural and process alternatives of corrosion protection during facility's specified service life.

Table 1 – Signs classification of technical and process risks under restoration of corrosive structure serviceability

Conditions of *SCPSC*	Class of risk	Description of risk	Characteristic of losses	Risk (R_i, point)	Extent of potential damages, *MWA**
Corrosion hazard	1	Catastrophe	Partial or complete failure of structures and installations	9–10	> 72500
	2	Critical	Losses exceed the estimated amount of gross income in the case of facility restoration	7–8	25000–72500
Degree of corrosion protection	3	Allowable	Losses do not exceed the estimate value of returns in the case of service life extension and engineering modernization of facilities	5–6	2500 – 25000
	4	Acceptable	Losses do not exceed the quality-related expenses during facility service life.	1–4	< 2500

**MWA - minimum wage amount*

Steel structure availability factor (K_g) is an important logistic characteristic of structural safety:

$$K_g = \frac{T_{zk} + T_{z\gamma}}{T_{zk} + nT_{z\gamma}}, \tag{1}$$

where n – number of repair cycles for renewal of corrosion protection at facility specified service life.

Table 2 – Parameters of corrosive structures serviceability based on risk classes of SCPSS

Designation of criticality category	Redundancy conditions / Condition for compliance with criticality category	Risk class	Interval estimate of failure based on technical criterion:		Reliability factor	
			A_z	h_k, mkm	γ_{zn}	γ_{zk}
C1	*SR* / Allows degradation of decorative features of secondary protection	3	0,85	-	0,99	0,95
		4	0,90		1,00	0,99
C2	*SR* / Does not allow degradation of protective features of secondary protection	3	0,55	-	0,95	0,9
		4	0,60		0,99	0,95
C3	*PR* / Allows degradation of protective features of secondary protection	3	0,40	50	0,90	0,85
		4	0,45	30	0,95	0,9
C4	*PR* / Allows degradation of features of primary protection	3	0,30	100	0,85	0,8
		4	0,35	70	0,90	0,85

Categories of criticality of steel structures based on the level of corrosion protection are specified depending on partial (within acceptable limits) degradation of primary protection efficiency (reliability factor γ_{zk}) parameters and secondary protection (reliability factor γ_{zn}, composite index of protective properties A_z, thickness of corrosion products under the paint coat h_k, corresponding to failure criterion of protective properties).

Functional reliability reserve under the terms of primary (*PR*) and secondary (*SR*) protection is achieved via setting a required level of corrosion protection (ZI – ZIV) or corrosion hazard (KI – KV) considering data presented in Table 3.

Process safety is achieved with the time reservation of the load capacity of structural steel considering functional survivability of corrosion protection systems under an acceptable risk of industrial facilities a stress-corrosion fracture consequences . Process-approach to ensuring process safety of structures reflects *DMAIC* action strategy (define, measure, analyze, improve, control).

Table 3 – Generalized matrix of choosing SCPSS reliability index level

Degree of exposure corrosiveness K, mm per year	Range estimates of corrosion protection availability factor, K_g				
	$0<K_g\le0,1$	$0,1<K_g\le0,3$	$0,3<K_g\le0,5$	$0,5<K_g\le0,7$	$0,7<K_g\le1,0$
Weak-level corrosive environment, $0,01<K\le0,05$	KI	ZIV	ZIII	ZII	ZI
Low-level corrosive environment, $0,05<K\le0,15$	KII	KI	ZIV	ZIII	ZII
Average-level corrosive environment, $0,15<K\le0,30$	KIII	KII	KI	ZIV	ZIII
High-level corrosive environment, $0,30<K\le0,50$	KIV	KIII	KII	KI	ZIV
Strong-level corrosive environment, $K>0,50$	KV	KIV	KIII	KII	KI

Reservation as a universal method of reliability assurance is used upon established procedure of corrosion monitoring (define, measure) and *SCPSS* diagnostics (analyze, improve, control).

At the stage of corrosion monitoring, the basis for decision making is the level of process safety risk (*LPSR*) with the use of 10–point scale according to the following equation:

$$R_i = \sum_{i=1}^{i=N} Q_i P_i , \qquad (2)$$

where Q_i – weight characteristic of *IDCS* significance;

P_i – detected survivability changes by the value of i-th sign.

It is proposed to manage process safety risks (R_i, point) based on the monitoring data depending on a specified class of hazard, level of threats and vulnerability of corroding structures (Table 4).

The basis for decision making upon the diagnostic data is the assessment of survivability criteria using the data of «Resource» information-analytical system (Fig.2).

Table 4 – Process safety risk (R_i, point) depending on class of hazard of SCPSS, level of threats and vulnerability of buildings and installations

Level of *SCPSS*	Level of threat (category of technical conditions)														
	Low (I)			Limited (II)			Average (III)			High (IV)			Ultimate (V)		
	Vulnerability assessment (category of responsibility)														
	V	B	A	V	B	A	V	B	A	V	B	A	V	B	A
ZI	1	1	2	2	2	3	3	3	4	4	5	5	6	6	7
ZII	1	2	2	2	3	3	3	4	4	5	5	6	6	7	7
ZIII	2	2	3	3	3	4	4	4	5	5	6	6	7	7	7
ZIV	2	2	3	3	4	4	4	5	5	6	6	7	7	7	8
KI	2	3	3	4	4	4	5	5	6	6	7	7	7	8	8
KII	3	3	4	4	4	5	5	6	6	7	7	7	8	8	9
KIII	3	4	4	4	5	5	6	6	7	7	7	8	8	9	9
KIV	4	4	4	5	5	6	6	7	7	7	8	8	9	9	10
KV	4	4	5	5	6	6	7	7	7	8	8	9	9	10	10

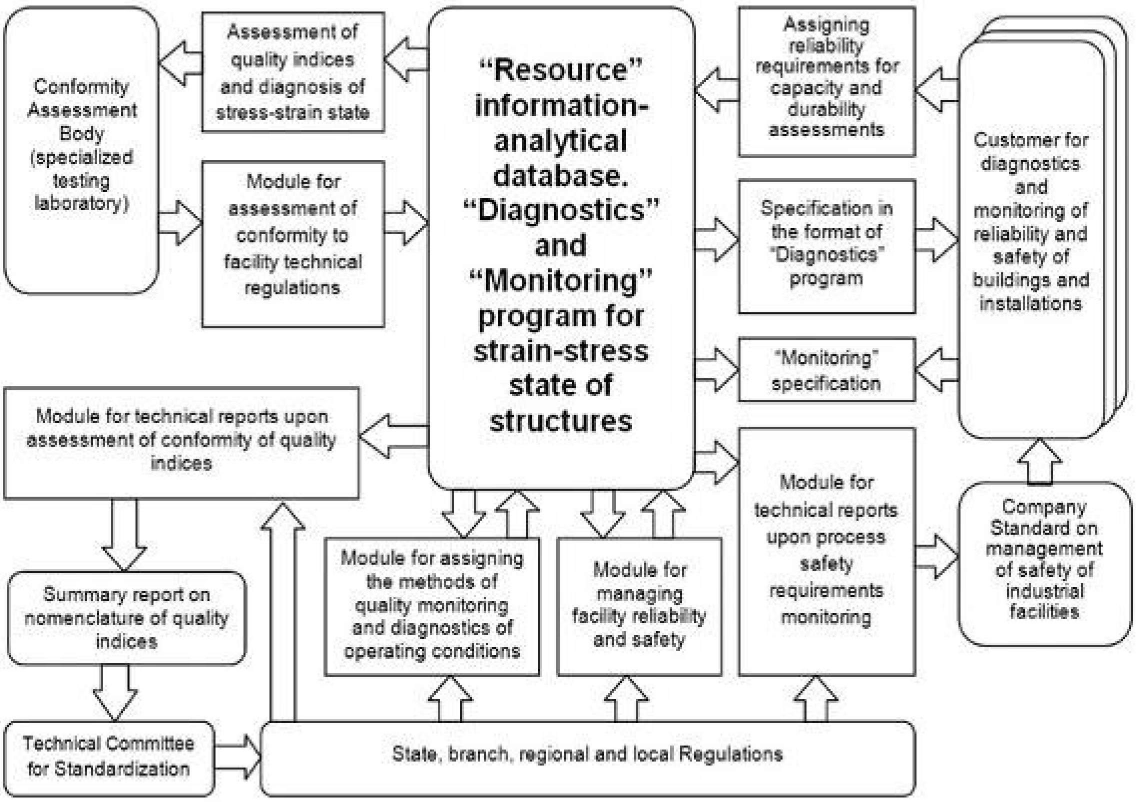

Figure 2 – Diagram of «Resource» information–analytical database for setting survivability indices of metal structures based on corrosion hazard level

The data of monitoring corrosion state of facilities (parameters of system «output») are used for reliability analysis («input» parameters) and justification of the requirements for maintainability, survivability and post-repair load capacity.

Process approach to assuring process safety of structures reflects *DMAIC* action strategy (define, measure, analyze, improve, control).

The data of facility corrosion state monitoring (parameters of system «output») allows carrying out reliability analysis (parameters of system «input») for setting process safety requirements considering indices of repairability, survivability and post–repair load capacity.

The limit state criteria in assessing *IDCS* in their actual state are set with the use of a feedback factor of structure operating conditions (ψ), using equation:

$$N = \frac{\Phi_{cr}}{\Gamma - \psi} \ , \tag{3}$$

where N – the largest rated force in the structural member, *kN*;

Φ_{cr} – limit force, *kN*, which can be withstood by the element of damageability Θ_f;

Γ – reliability margin ratio.

By feedback it is meant results of *IDCS* monitoring for assessing, controlling and correcting *PRA*. With accumulation of damages Θ_f the feedback factor (ψ) defines the degradation of structural performance under the specified design value of reliability margin ratio (Γ). Proposed feedback factor (ψ) provides for realization of the analytical approach to managing process safety. Process safety depends on the integral index of survivability (η), which determines reduction of service life regulation capacity at structural in–service degradation:

$$\eta = \Gamma - \frac{1}{\Gamma - \psi} \ . \tag{4}$$

Perturbing actions of negative internal impacts result in corrosive damages and occurrence of structural limit states signs. Capacity of service life regulation defines the permissible deviations of the reliability margin ratio design value (Γ) for assuring serviceability.

Conclusions. Logistical management in the course of structural survivability upon corrosion hazards planning allows eliminating uncertainty and subjectivity choosing design solutions of *SCPSS*. For real business, assessment of industrial facilities process safety means reducing the risks due to corrosion hazard. Thus, the theory of potential effectiveness is realized to assess risks, considering structure reliability, durability and safety.

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LIGHT CONCRETE COMPOSITE SLAB WITH FLEXIBLE REINFORCEMENT STRESS-STRAIN STATE MODELING BY FINITE ELEMENT METHOD

The light concrete (polystyrene concrete) composite slab with profiled steel sheeting was considered. In this slab, the flexible reinforcement (PMA-2) was used as strengthening. Previous experimental studies have shown that this slab has increased carrying capacity in 2.4 times compared with a similar slab without additional anchoring means. The finite element method (FEM) was used for a detailed study of the PMA-2 slab work and its components. The calculations of light concrete composite slab allowed to investigate the work of profiled steel sheeting, material contact, their bundles, as well as the work of the reinforcement and its influence on the stress-strain state parameters of polystyrene concrete and profiled steel sheeting. Comparison by FEM calculations with experimental data confirmed the accuracy and adequacy of the developed model for PMA-2 slab.

Keywords: *light concrete composite slab, profiled steel sheeting, polystyrene concrete, reinforcement, deformation property, load, finite element method (FEM).*

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МОДЕЛЮВАННЯ НАПРУЖЕНО-ДЕФОРМОВАНОГО СТАНУ ЛЕГКОБЕТОННОЇ КОМБІНОВАНОЇ ПЛИТИ З ГНУЧКИМ АРМУВАННЯМ МЕТОДОМ СКІНЧЕННИХ ЕЛЕМЕНТІВ

Розглянуто легкобетонну (полістиролбетонну) комбіновану плиту з профільованим настилом, у якій як підсилення використовувалося гнучке армування (ПМА-2). Виявлено з попередніх експериментальних досліджень, що така плита має збільшену у 2,4 раза несучу здатність порівняно з аналогічною плитою без додаткових засобів анкерування. Застосовано метод скінченних елементів (МСЕ) для детального вивчення роботи плити ПМА-2 та її компонентів. Досліджено роботу профнастилу, контакту матеріалів, їх розшарування, а також роботу арматури та її вплив на параметри напружено-деформованого стану полістиролбетону і профнастилу з проведених розрахунків легкобетонної комбінованої плити. Підтверджено точність та адекватність побудованої моделі для плити ПМА-2 порівняннями розрахунків за МСЕ з експериментальними даними.

Ключові слова: *легкобетонна комбінована плита, профільований настил, полістиролбетон, армування, деформативність, навантаження, метод скінченних елементів (МСЕ).*

Problem statement. The question of using efficiently structures in construction is particularly relevant for Ukraine and for any another country now. Considering this factor, light concrete composite slabs with profiled steel sheeting [10] are taken as structures that have not only high design qualities, but also thermal insulation. Ideally, the slabs of the coating and the ceiling should be light, but along with it they are strong, have low deformability and a low coefficient of thermal conductivity. Light concrete composite slabs where light concrete will work together with profiled steel sheeting can combine all of the qualities mentioned above. The question is the next – what type of connection is better to use in such slabs and how to carry out analytical calculations that can describe the work of different types of light concrete composite slabs with profiled steel sheeting under load.

Analysis of recent research and publications. Scientists in the field of construction work carried out on the application of various types of anchoring and reinforcement in beam steel reinforced concrete structures. But mainly heavy concrete was used in these constructions. Such studies were conducted by Skyba O., Belyaeva S., Darienko V. and others [1 – 4]. Polystyrene concrete samples depending on the concrete strength class investigation, considering the stability of the profile in the steel reinforced concrete samples, are given in the works of Avramenko Yu. and Semko O. [5, 15].

A question that has not been considered by previous scientists, but that is relevant in connection with the expansion of energy efficient structures use, is in the investigation of the stress-strain state of light concrete (polystyrene concrete) composite slabs with profiled steel sheeting by the finite element method under applying as heat-insulating and structural building element.

It can be assumed that the analytical methods for calculating steel-reinforced concrete slabs [10] do not fully describe the work of the considered slabs. It is due to the fact that different assumptions or hypotheses about the slabs deformation nature are used to obtain finite equations. In addition, engineering methods do not allow to investigate the stress-strain state of both reinforcement and concrete, since they are intended only for strength and deformability verification.

The numerical methods deprived most of these deficiencies and based on direct solution of theory of elasticity. Depending on the discussed tasks, these methods can be used both for the slabs stress-strain state study and for their simplified engineering calculations. Composite slabs and profiled steel sheeting modeling using finite element method is involved by many foreign scientists [14, 17]. In their studies, they solve a variety of specific tasks, such as the development of finite elements, approaches to applying boundary conditions, overlay nets depending on the materials type, loading type etc. [7, 8, 11, 12, 18].

There are many software complexes implementing structures calculations by the finite element method. The most common ones among them are «ANSYS» by «ANSYS Corporation», «FEMAP» (NASTRAN) by «Siemens PLM Software», «ABAQUS» and «COSMOS» by «Dassault Systèmes Simulia Corporation», «LIRA» by «LIRA soft», «LS-DYNA» by «Livermore Software Technology» and others. Each complex has its own set of finite elements that simulate the structure stress-strain state depending on the material, load, deformation type etc. FEMAP software package is one of the most convenient in use. It has intuitive interface, pre- and post-processing, a large number of finite elements that allow to create models of high degree of complexity. Therefore, it is well suited for scientific research. In most cases, these software systems are used for calculating and investigating the stress-strain state of existing building structures types and new ones [13, 16].

Selection of unsolved question. The question that is relevant in the study of the stress-strain state of light concrete (polystyrene concrete) composite slabs with profiled steel sheeting by finite elements method is the creation of an adequate model for slab type calculating. This model must have finite elements that correspond to the properties of the construction materials (concrete, steel). The contact layer among the materials, which is responsible for modeling the fracture at the time of material bundle, should be correctly displayed in the model. The construction of finite elements grid, the application of loads and boundary conditions should consider all the conditions where the investigated structure is located.

Purpose and objectives of the research. The main purpose of the research is to develop approaches to modeling the stress-strain state of composite steel-reinforced concrete slabs (CSRS) by finite element method (FEM).

The set goal requires the solution of the following tasks: creation of three-dimensional geometric models of profiled steel sheeting, polystyrene concrete and contact; definition of physical and mechanical models of materials and contact work; building of finite elements grid; applying loads and boundary conditions; determination the calculation type and obtained data analysis.

The main part of the research. In previous experimental studies, two rational types of anchoring were identified - the anchoring in the form of flexible (PMA-2 slab) and horizontal (PMA-3 slab) reinforcement. The flexible reinforcement use (PMA-2) as anchoring makes it possible to increase the carrying capacity in 2.4 times compared to the slab without any means of anchoring, and horizontal reinforcement (PMA-3) makes it possible to reduce deformation for 8% under a load of $0.6M_{ult,\ exp.}$ The studies by finite element method are presented exactly for the PMA-2 slab with flexible anchoring.

An experimental and analytical study of a PM-1 slab without anchoring and reinforcing has been carried out earlier and are described in details in [6, 9].

All types of FE were used to calculate the slabs - linear, flat and volumetric. Geometric models for the composite slabs were constructed according to the FE type. Three-dimensional bodies were used for a concrete array and contact; the extreme lower planes of the already constructed contact - for profiled steel sheeting; the lines formed at the intersection of the jagged planes - for reinforcement. The profile of profiled steel sheeting was used as basis for the construction where the necessary planes of slab cross-section were formed. After that, the plane «pulled» in the base bodies, which were the basis for the slab models construction. Further transformations of the base geometric bodies are in their partition into pentagons and hexagons in accordance with the shape and location of the specific slabs reinforcement.

Modeling of materials and contact was based on the results of experimental determination of physical and mechanical characteristics and on the material behavior during loading. Work model was considered for each material and depended on the expected conditions of its work. Profiled steel sheeting and reinforcement were set as isotropic nonlinear elastic-plastic material; polystyrene concrete and contact - as non-linear elastic material. Diagrams of materials deformation, that was accepted for the calculation by FEM, were given in the form of two-linear functions $\sigma - \varepsilon$, the horizontal section of which corresponded to the limit of strength or durability of the materials.

The types of FE and their properties were determined before the beginning of the finite element (FE) grids construction. Thus, the linear, flat, voluminous and completely rigid FE was used for the slabs modeling. The main condition for the correct calculation was the connectivity of the grids in the nodes that combine different materials and types of FE. The grids were imposing on all geometric body of model determining its size and configuration. Figure 1 shows the complete FE model of the PMA-2 slab.

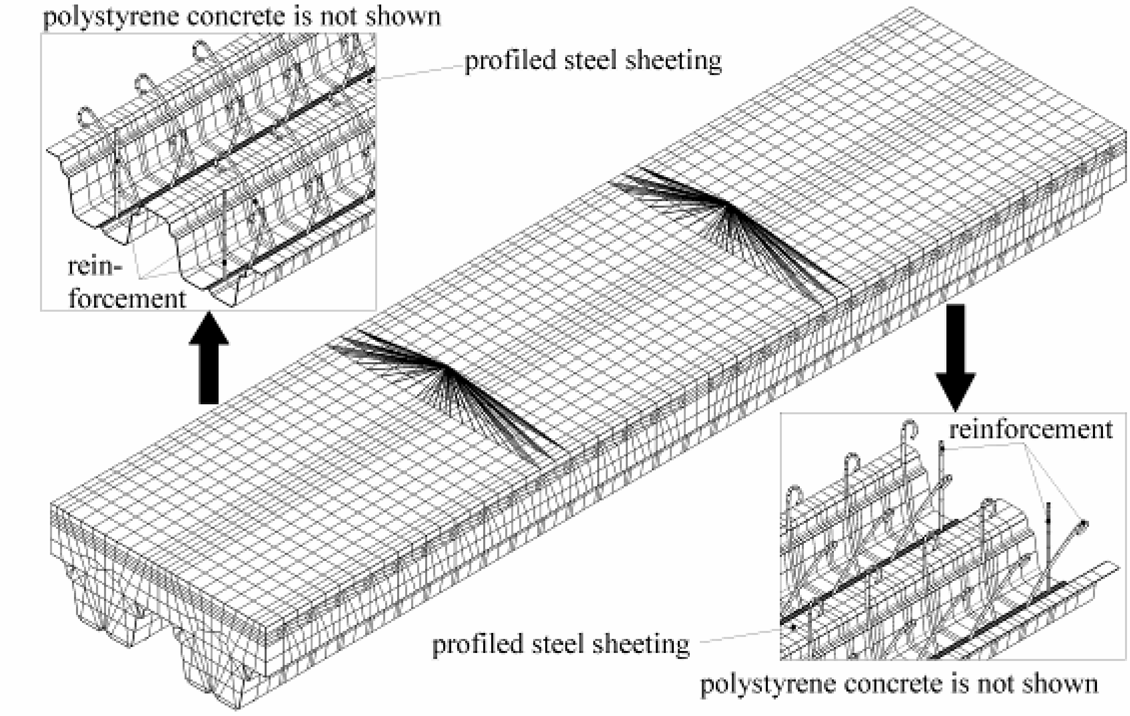

Figure 1 – Complete FE model of the PMA-2 slab

Placements of loading loads, their size and configuration corresponded to the experimental data. Concentrated loads through boards located on the upper surface of polystyrene concrete were applied in the experiment for PMA-1...PMA-5 slabs. The load was applied through complete rigid FE (Figure 1) for adequate modeling of these conditions. The total load value for the PMA-2 slab was equal to 21480 H.

Support nodes of all slabs and profiled steel sheeting were modeled using completely rigid FE. In all cases, the following degrees of freedom are accepted: on the one site of model the support bracket forbade linear displacement along the axes x, y, z, and the angular displacement around the z axis; on the other site the support bracket forbade displacement along the axes x and y and angular displacement around the z axis. Additionally, the nodes of vertical plane were forbidden to move in the direction of the vertical axis in areas of support slabs.

Linear and nonlinear static calculations were performed for all models. During the linear static calculation, the nonlinear properties of materials and the geometric nonlinearity of the construction were ignored. The calculation is based on the initial modules of elasticity. Nonlinear calculation provided applying the load in a few steps with increasing from zero to its maximum value. The values of intersecting elastic module were calculated at every step of loading according to the charts of materials deformation. This process ended when the maximum load was reached, or when the structure was transformed into a mechanism – appearance of plastic hinges, loss of local or general stability, etc. There were 20 steps of load in all nonlinear calculations, but not all slabs reached the maximum load value. The maximum experimental load was not achieved during calculating for both the PMA-2 and PMA-1 slabs. Calculation was ended on value $0,8F_{max}$. The results of nonlinear calculation of PMA-2 slab at maximum load are shown in Figures 2-7. The maximum deflections determined in the middle of the span at the location of the defibrillator are given in Table 1.

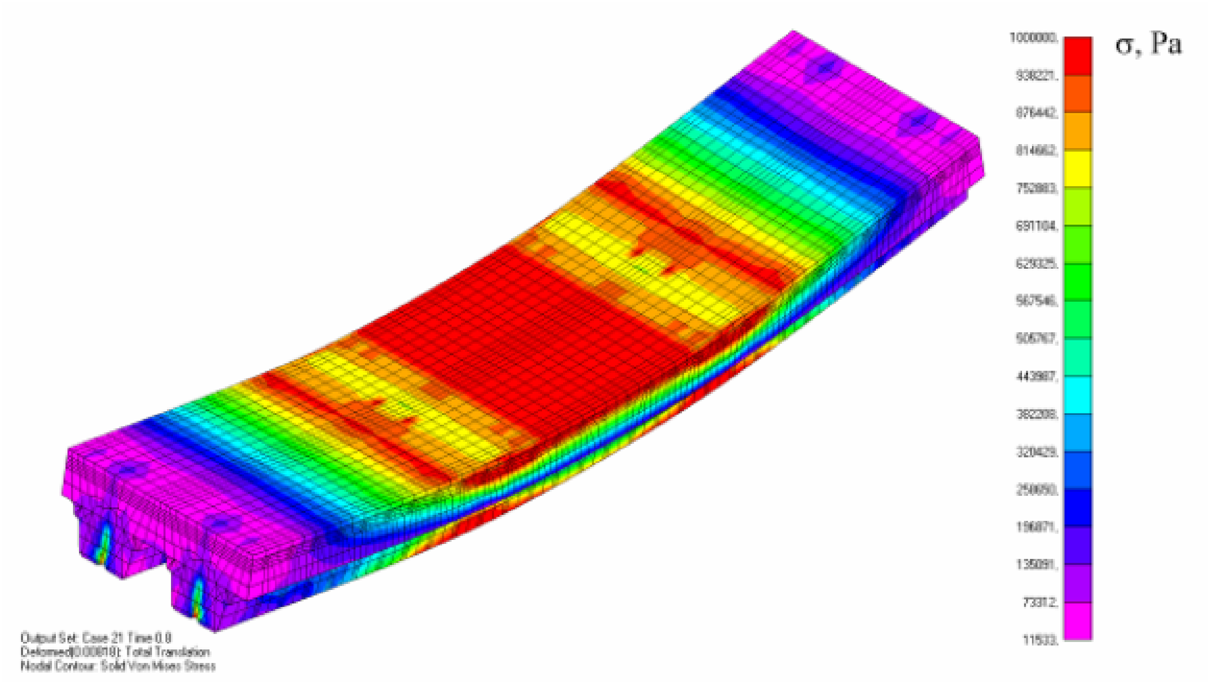

Figure 2 – Chart of Mises stress distribution and deformation of polystyrene concrete when nonlinear calculating PMA-2 slab

The chart of Mises stress distribution and deformation of polystyrene concrete with nonlinear calculating PMA-2 slab is shown in Figure 2. Small concentrations of stresses are observed on the upper surface of the polystyrene concrete in the locations of the reinforcement. There are also concentration stresses from the effects of loads through complete rigid FE. The polystyrene concrete is pressed by reinforcement at the ends of the PMA 2 slab. Maximal stresses arise in the middle of span on the upper part of slabs, and under concentrated forces in lower part of the slab. This distribution is explained by the action of concentrated forces and distribution of reinforcement on all-area of the slab.

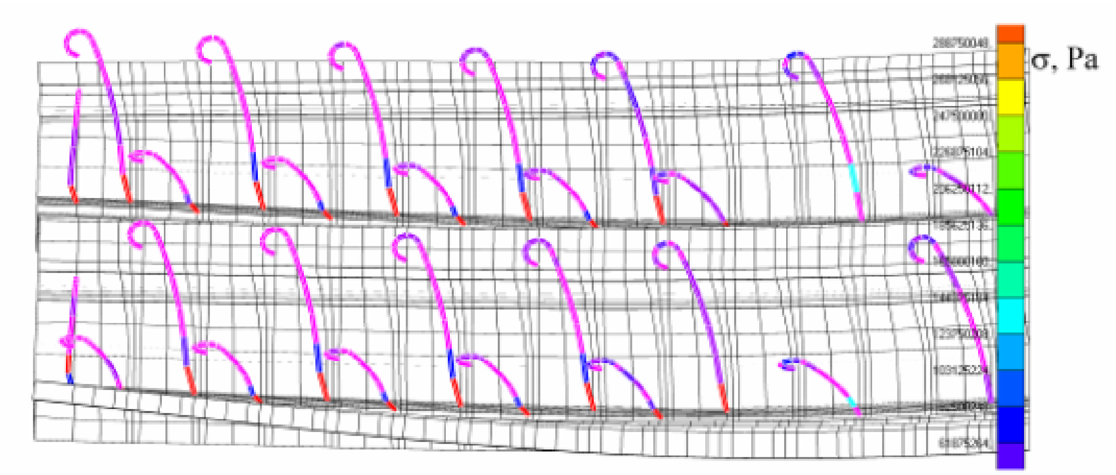

Figure 3 – Chart of Mises stress distribution and deformation of reinforcement with nonlinear calculating PMA-2 slab

Reinforcement work is shown in Figure 3. As it is shown, all reinforcement is reached with yield strength in the contact zone of profiled steel sheeting at maximum load. It undergoes the greatest deformation in bearing zones.

The effect of reinforcement on the stress state of polystyrene concrete is shown in Figures 4-6. The location of reinforcement at different angles causes a chain change in the stresses in the lower central depression of polystyrene concrete that depends on the angle of the reinforcement. The reinforcement located at different angles helps to avoid the occurrence of a longitudinal crack (Figure 4).

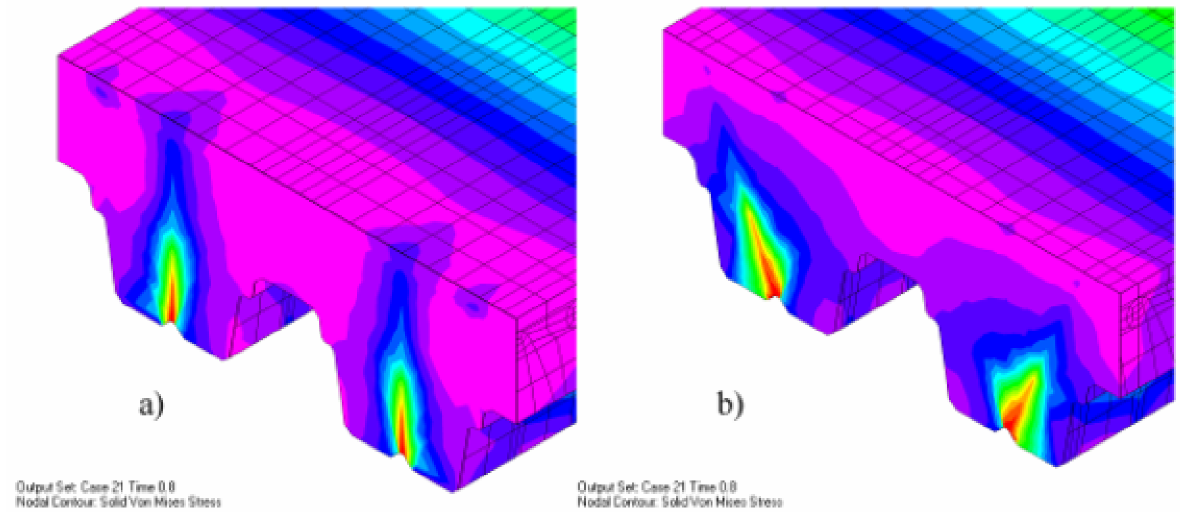

Figure 4 – Effect of vertical and inclined reinforcement at stress state of the PMA-2 slab: a) cross-section at vertical reinforcement; b) cross-section at inclined reinforcement.

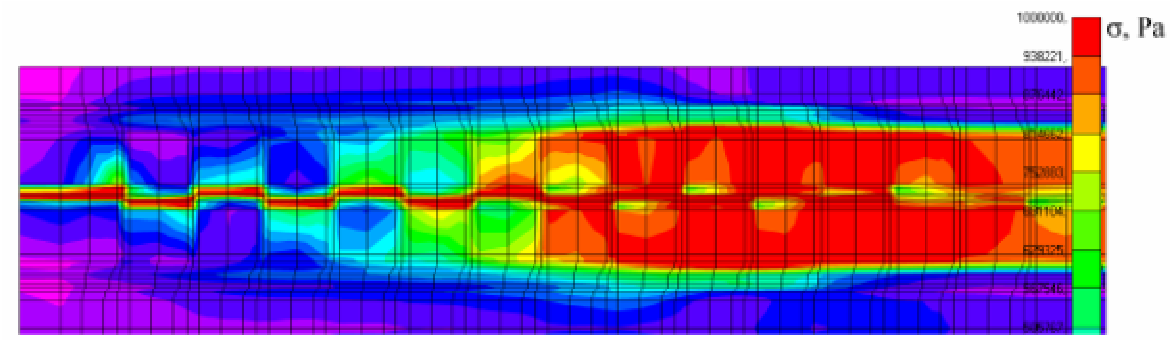

Figure 5 – Effect of the reinforcement on the stress state of polystyrene concrete (bottom view) for the PMA-2 slab

Figure 6 a) shows the distribution of stresses along a longitudinal vertical section through vertical reinforcement. The zones of boundary stress is clearly observed in places of its location that caused by bending of the reinforcement and subsequent bulging of concrete up to the ends of the slab.

Figure 6 b) shows the distribution of stresses along the longitudinal slope section through the inclined reinforcement. In this case, the location of the reinforcement does not cause a continuous zone of boundary stresses in lower part of polystyrene concrete, which improves the mutual work of reinforcement with polystyrene concrete. Distribution of stresses in the zones of inclined reinforcement location is the same but a little bit changes when approaching the middle of the slab. Small circular stress reduction areas are observed in the upper part of the inclined cross-section corresponding to the corner radius at the ends of the inclined reinforcement. The reinforcement works only in the lower part of slab height, causing the corresponding perturbations. Stress distribution does not change in the middle and upper slab parts in zones of reinforcement location.

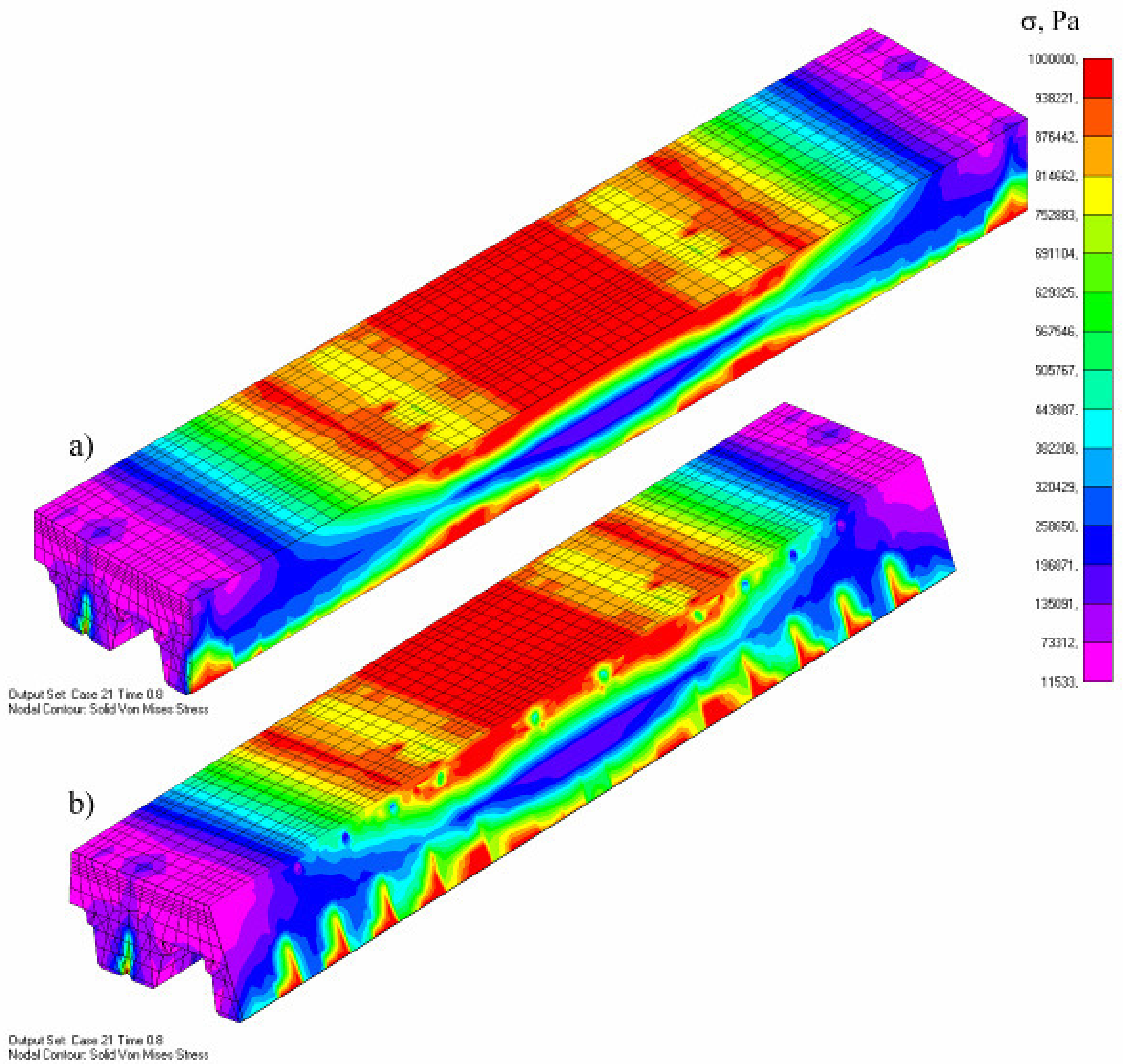

Figure 6 – Effect of vertical and inclined reinforcement on the stress state of the PMA-2 slab: a) longitudinal vertical section on reinforcement; b) longitudinal inclined cross-section along the inclined reinforcement

Contact area between polystyrene concrete and profiled steel sheeting decreased with load increase, ranging from supporting and central zones and ending with lateral surfaces of profiled steel sheeting in the middle of slab span. Full separation of materials occurs at a load of about 5000 H. Further deformation of slab components occurred differently for different zones of the slab. For example, the separation of profiled steel sheeting from concrete increased in support areas, while both materials are equally deformed in the middle span, although they are not connected. Their final separation occurred only when load is 12000 H. In this case, the left shelf of profiled steel sheeting began to lose local stability.

In Figure 7, according to Table 1, PMA-2 slab deformation graph was constructed. The main difference slabs with other types of anchoring research results is that the process of separation was practically free of deformation jumps. This is due to the increased slab stiffness. Separation of profiled steel sheeting from polystyrene concrete begins at load of 12000 H, and the distance between adjacent nodes begins to increase according to increase of load.

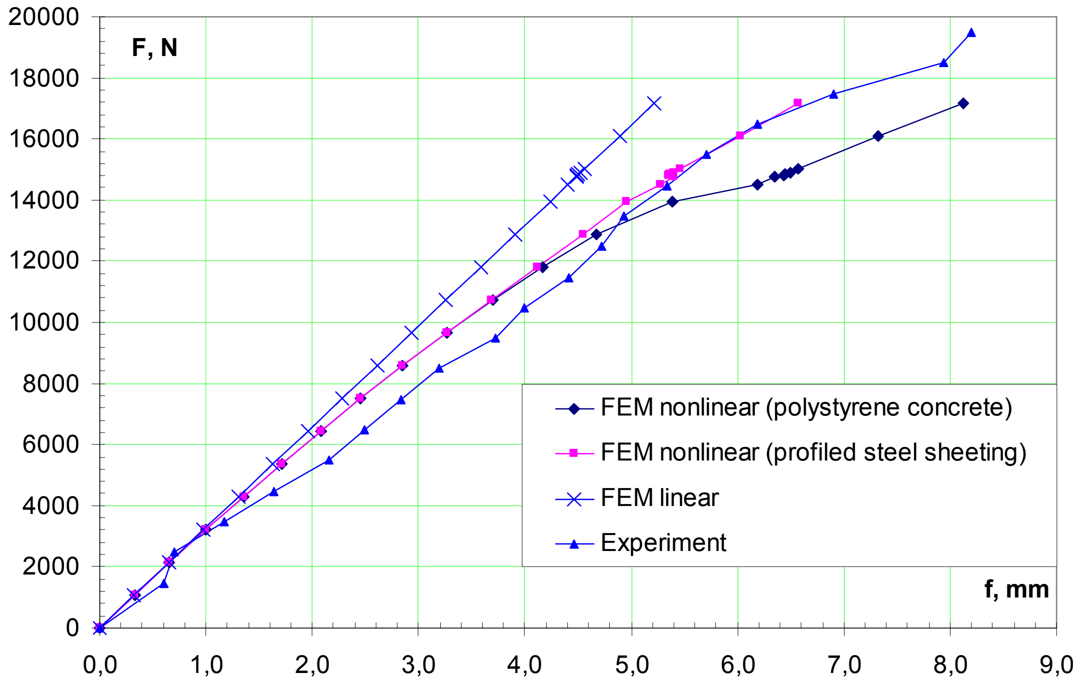

Figure 7 – Comparison of the deflections for the PMA-2 slab by FEM, with experimental data

Table 1 – Results of deflection calculations for PMA-2 slab

Load step	Load, N	Deflection, mm		
		Polystyrene concrete	Profiled steel sheeting	Linear calculation
0	0	0,00	0,00	0,00
1	1074	0,33	0,33	0,33
2	2148	0,66	0,66	0,65
3	3222	1,00	1,00	0,98
4	4296	1,35	1,35	1,30
5	5370	1,72	1,72	1,63
6	6444	2,08	2,08	1,96
7	7518	2,46	2,46	2,28
8	8592	2,85	2,85	2,61
9	9666	3,26	3,26	2,93
10	10740	3,70	3,69	3,26
11	11814	4,17	4,12	3,59
12	12888	4,68	4,55	3,91
13	13962	5,38	4,96	4,24
14	14499	6,19	5,28	4,40
15	14768	6,35	5,40	4,48
16	14801	6,43	5,35	4,49
17	14835	6,45	5,37	4,50
18	14902	6,49	5,40	4,52
19	15036	6,57	5,46	4,57
20	16110	7,33	6,03	4,89
21	17184	8,12	6,57	5,22
–	21480	–	–	6,52

Conclusions. The calculations of the profiled steel sheeting and the slabs by finite element method allowed to investigate the work of the profiled steel sheeting, the contact of the materials, their bundles, as well as the work of the reinforcement and its influence on the parameters of the stress-strain state of polystyrene concrete and profiled steel sheeting. The error in experimental data and calculation by FEM does not exceed 8.4% for nonlinear calculations and 21% for linear ones. The anchoring method used in the PMA-2 slab can be recommended for use in cases where it is necessary to increase bearing capacity significantly due to a slight decrease in deformation properties of the slab. Modeling methods for PMA-2 slab and its components, used in calculations by FEM, can be recommended for application in the finite-element models of steel-reinforced concrete structures development.

Practical recommendations for light concrete slabs with profiled steel sheeting by FEM modeling and calculation, including the foundation for choosing FE type for the rofiled steel sheeting modeling is occurred according to the research.

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APPLICATION OF HYDROPHOBIC CEMENT SLURRIES «RAN-M» OF «RAMSINKS-2M» GROUP TO AVOID FLUID KICK

Laboratory studies of cement slurry and cement stone is established that hydrophobic cement slurry «RAN-M» consists of NTPha additives for well cements PTC-1-100 and «Ramsinks-2M». In the laboratory confirmed the technical parameters of the newest hydrophobic cement slurries (mobility, density, separation, pumpability, etc.) according to the standard requirements in the respective devices. Done such works as: implementation of the selection of formulations of cement slurries with different rate of strength development for different temperature integrals.

***Keywords**: permeability, well, technical condition, cement, Water-repellent, behand column overflows.*

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ЗАСТОСУВАННЯ ТАМПОНАЖНИХ ГІДРОФОБНИХ РОЗЧИНІВ «RAN-M» ГРУПИ «RAMSINKS-2M» ДЛЯ УНИКНЕННЯ ФЛЮЇДОПРОЯВІВ

Лабораторними дослідженнями тампонажного розчину і цементного каменю встановлено, що тампонажний гідрофобний розчин «RAN-M» складається з добавки НТФк до тампонажного цементу ПЦТ-1-100 і «Ramsinks-2M». У лабораторних умовах підтверджено технічні параметри новітніх гідрофобних тампонажних розчинів (рухливість, густина, водовідділення, прокачуваність тощо) згідно зі стандартними вимогами на відповідних приладах.Виконано таку роботу, як здійснення підбору рецептур тампонажних розчинів з диференційованим темпом набору міцності для різних температурних інтегралів.

***Ключові слова**: проникність, свердловина, технічний стан, гідрофобізатор, міжколонні перетоки.*

Introduction. The urgency of the creation of newest cement slurries is due to the need to improve the quality of isolation of the formation at different stages of the completion and operation of wells. With the growth of the depths of oil and gas wells, reservoir temperatures and pressure are increased, resulting in complicated work on the separation of layers in the wells.

An analysis of the industrial statistical material on gas, oil and water inflow in the fields of the Dnipro-Donetsk depression (DDd) in the course of the completion and operation of wells shows that at the fields from 10% to 50% of oil and up to 60% of gas wells have interplanetary flows and in the connection to this well is partially or completely ineffective for efficient operation.

Cementing of wells, especially deep ones, is the most crucial phase of their construction. The value of cementing works is due to the fact that they are the final process, and failures in their execution can minimize the success of the previous work up to the loss of the well, so to improve the quality of wells construction, and especially at the final stage of work – well casing. One of the main causes of these phenomena is poor-quality boreholes, in particular in the zone of production formation.

An overview of the latest sources of research and publications. Scientists from Russia, Ukraine, the USA, England and other countries, whose names are known in oil companies around the world, made a significant contribution to the development of modern ideas about cementation in general and its various aspects [1]. Domestic scientists and engineers are more familiar with famous works of scientists working in this field in Russia, Ukraine and Azerbarijan. A significant contribution to the development of such issues as accident prevention, fastening, cementing, cement slurries were made by V.F. Abubakirov, M.O. Ashrafyan, V.S. Bakshutov, A.I. Bulatov, Yu.M. Basarigin, V.P. Detkov, Ya.S. Rybchicha, M.A. Myslyuk, Ya.S. Kotskulich, Ye.M. Solovyov, V.M. Svetlitsky and others [2].

Analysis of literary sources and production and construction data indicates that the success of cementing wells is determined by the technique and technology of the processes of cementing, the quality of preparatory work, the cement material and the completeness of replacement of the mud with cement slurry [3].

Selection of previously unsolved parts of the general problem. The analysis of special literary sources and production and construction data shows that the materials used today for the separation of layers do not always correspond to reservoir conditions of deep wells and do not provide reliable sealing of the cement ring [4]. The success of cementing wells is determined by the technique and technology of the processes of cementing, the quality of preparatory work, the cement material and the completeness of replacement of the mud with cement slurry [5].

Existing cement slurries (for example, PTC 1-100) according to their physical, mechanical and physical and chemical properties do not fully meet the requirements of the quality of cementation of operation wells [6]. With prolonged storage, especially in the autumn-winter period, the properties of the portland cement deteriorate: the cement breaks down, the timing of seizing the cement slurry is extended, the viscosity is increased, the mechanical strength of the cement stone is decreased. Prolonged storage in wet conditions sometimes leads to the conversion of all cement, which is in a normal paper bag, into a stone-like body [7].

The purpose of this work is to create the latest hydrophobic cement slurries with a differentiated rate of strength to prevent fluid kick in the waiting on cement (WOC) in the temperature range from 0° C to 180° C, which will ensure a high quality of isolation of formations during the operation of wells.

Main material and results. Experimental and industrial testing of cement slurries with the addition of NTKa was carried out in relation to verification of the method of introducing a reagent into astringent, identifying the optimal amounts of the reagent, rational technology of preparation and application of cement slurry with the addition of NTKa, as well as identifying the possibility of transition to industrial use of cement slurries with the additive NTFK. The works were carried out in three programs: for bottom hole temperatures 50-65 °C; 75-90 °C and 100-130 °C.

The method of selection of correlations proposed by the authors of the latest cement slurries, consisting of cement PTC 1-100 and hydrophobic material «Ramsinks-2M» is developed. The ratio of PTC 1-100 cement and hydrophobic material «Ramsinks-2M» in laboratory conditions was 1: 0.001; 1: 0.002; 1: 0.003; 1: 0.005; 1: 0.008.

The use of PTC cement 1-100 is widely known. In more detail, we will consider the hydrophobic material «Ramsinks-2M».

The problem of steady stream flows today exists both in the wells of OJSC Ukrnafta and in the wells of the UkrhazvydobuvannyaSC, NJSCNaftogaz Ukraine.

The latest cement slurries will allow the use of such cement mixtures for the cementing of oil and gas wells in the zones of ANPT, which meets the criterion of industrial use.

Technological properties of the proposed materials are following:

– NTKa is nitrile trimethyl phosphonic acid which is a white crystalline powder, well soluble in water at any temperature, as well as in acids and alkalis. It is widely used in cementing wells for the purpose of regulating the tensile strength of cement slurries;

– «Ramsinks-2M» is a hydrophobic additive (water repellent), a complex silicone hydrophobic compound. The application of this additive in the manufacture of cement mixture «RAN-M» (hydrophobic additive «Ramsinks-2M» + cement PTC-1-100 + NTKa) increases the elasticity of the mixture, prevents the uneven concentration of fillers, as well as prevents the bundle of the mixture and increases the resistance to impact aggressive factors and increases their longevity. The high water resistance of the products with the additive is achieved with the appropriate composition of the cement mixture «RAN-M» by a thin decomposition of hydrophobic particles in the mixer SMN-20.

Test conditions:

– air temperature in the room 20 °C;

– atmospheric pressure 742 mmof mercury column

– humidity 78%;

– pressure in the autoclave installation A-2.00.000.IE 450 atm .;

– temperature in the autoclave installation A-2.00.000.IE 75 ° C.

The main indicators of the quality and effective use of the additive «Ramsinks-2M» are: hydrophobic effect (degree); water absorption of cements and slurries; strength; waterproof; plasticity, etc.

The autoclave unit A-2.00.000.IE in the complex with a special device for the installation of metal forms with samples, whose function is to prevent the destruction of samples, was used to form a cement stone from the cement mixture «RAN-M».

To determine the coefficient of open porosity, the Preobrazhensky method was used - the method of weighing samples after full saturation of pores with a liquid, chemically neutral with cement generating minerals. Experimental studies were conducted using kerosene. For this purpose, in a specially made metal forms, cylindrical samples of a cement mixture «RAN-M» 39.5 ~ 1.0 mm in length and 26 ~ 1.0 mm in diameter were formed for this purpose in the autoclave installation.

The method of determining the coefficient of open porosity is indicated in the following:

– preparation of samples formed in autoclave;

– blowing samples with air;

– drying of samples to constant mass in a drying cabinet at a temperature of 30-40 °C;

– weighing of dried samples in the air;

– saturation with samples of kerosene using a vacuum system;
– weighing of samples saturated with kerosene in the air;
– weighing of kerosene-saturated samples;
– determination of the coefficient of open porosity;
– processing of research results.

Selection of optimal ratios of PTC 1-100 and hydrophobic material «Ramsinks-2M» provides the required density of the cement slurry, the rate of strength at high operational parameters of the stone.

According to the authors, hydrophobic cement slurry due to the mechanical interaction of the hydrophobic material «Ramsinks-2M» with the structure of the cement PTC 1-100 will significantly improve the physical and mechanical and physical and chemical properties of standard cement slurry, which will ultimately lead to a significant improvement in the isolation of the productive formation on stages of completion of wells and their operation.

The determination of the hydrophobic effect was carried out by a laboratory test on the degree of hydrophobicity of the cement PTC 1-100 with the addition of «Ramsinks-2M» a sample of 200 grams of cement was taken, which was filled with water to obtain a normal density of the cement paste, leaving it at rest and marking the time absorption of water by cement.

The data obtained when tested with different values of «Ramsinks-2» as a percentage of cement weight (0.2, 0.25, 0.3%) are given in Table 1 below.

Table 1 – Effect of the amount of hydrophobic additive on the property of cement

Brand and type of cement	Mass of cement sample	Name of additive	Additive ratio (% of the mass of cement)	NDCP-normal densityof c/paste, ml	Degree of cement hydro-phobicity, min
PTC 1-100	200 g			95 ml	8
PTC 1-100	200 g	«Ramsinks-2M»	0,02	95 ml	11
PTC 1-100	200 g	«Ramsinks-2M»	0,025	95 ml	14
PTC 1-100	200 g	«Ramsinks-2M»	0,03	95 ml	17

According to the results of the laboratory test of the degree of hydrophobicity of the cement PTC 1-100 with hydrophobic additive «Ramsinks-2M» it was established that the degree of hydrophobicity of cement depends on the amount of additive «Ramsinks-2M» in percent (%) to the mass of cement.

The cement mixture «RAN-M» is used directly when performing cement works in the well. When mixing, an even-fitting cement slurry is formed.

In laboratory conditions, the following works were performed: the selection of formulations of cement slurrywith a differentiated rate of strength set for different temperature integrals. It is necessary to study their technological properties; study of the physical and mechanical properties of the cement stone in the temperature range from 20 to 80 °C. It is necessary to study the properties in the temperature range of 80-180 ° C; study of thermal stability of cement mixtures at temperatures up to 80 °C. It is necessary to continue the study of thermal stability at temperatures up to 180 °C.

The scheme of the selection of recipes with the necessary parameters and study of the physical and mechanical properties of the cement stone is standard and performed at temperatures of 70 °C, 100 °C, 130 °C, 160 °C and the corresponding pressures by aligning the ratio of PTC cement 1-100 and the hydrophobic material «Ramsinks -2M» for these conditions. Samples are stored in hydro-bacterial conditions for 1, 7 and 28 days.

Table 2 – Results of determination of absolute gas permeability on samples of cement stone from PTC-1-100 and hydrophobic additive «Ramsinks-2M»

№ of laboratory sample	Recipe of the sample	Gas permeability, a $\times 10^{-15}$м^2
40443	Cement stone with PTC-1-100	0,15
40444	Cement stone with PTC -1-100, 0,2% additive «Ramsinks-2M»	0,15
40445	Cement stone with PTC -1-100, 0,25% additive «Ramsinks-2M»	0,10
40446	Cement stone with PTC -1-100, 0,3% additive «Ramsinks-2M»	0,05
40447	Cement stone with PTC -1-100, 0,35% additive «Ramsinks-2M»	0,04
40448	Cement stone with PTC -1-100, 0,4% additive «Ramsinks-2M»	0,04

In the laboratory, i have been conducting researches on the determination of absolute gas permeability according to samples of cement stone from PTC-1-100 and hydrophobic additive «Ramsinks-2M». The tests are performed according to GOST 26450.0-85 – GOST 26450.2-85.

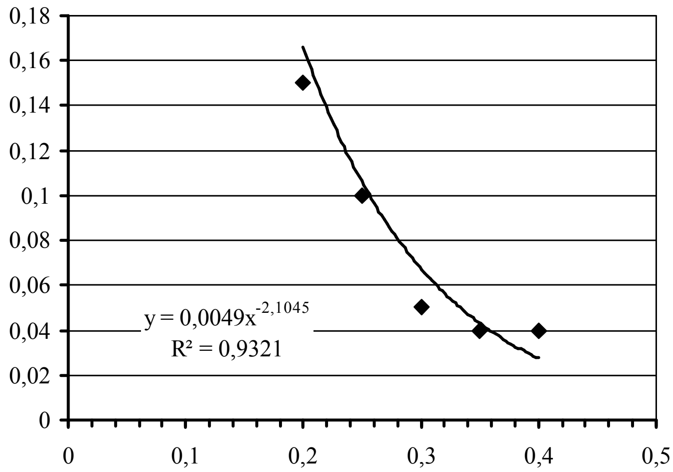

Figure 1 – Determination of absolute gas permeability by samples

This function proves that the gas permeability decreases with the application of different compositions of hydrophobic additives, but the best performance is achieved with a 0.3% additive of «Ramsinks-2M» in PTC-1-100 cement. A further increase in the percentage of quantity leads to a deterioration in the results.

The scientific novelty of the obtained results is that, due to the conducted research:

– the proposed technical solution in comparison with the existing ones will allow to get hydrophobic cement slurries with lower ranges of density of the cement slurry, high stability, good pumpability and high strength of hardened stone, guarantees the reliability of isolation of productive formation;

– to ensure the high quality of the separation of water and gas layers by improving the technology of cementing the operational wells in the «Ukrburgaz», the selection of hydrophobic cement slurries that best meet the mining and geological conditions of the wells of the Starosambirskfield;

– in particular, it is suggested to take hydrophobic materials from the group «RAN-M» to study.

Conclusion. After carrying out laboratory tests on the development of improved cement slurry for increasing their strength, according to the results of examinations of samples installed:

– cement = composition A - the best result with the admixture of NTFa to the cement of USCS 120 at B/C = 0,5 was equal to 5.4 MPa;

– oil-well cement = composition B – cement PTC-1-100 + 0,006 NTFa + 0,25% «Ramsinks-2M» (at B/C = 0,33) = 9,4 MPa;

– oil-well cement = composition C – cement USC 120 + 1.5% Stinol + 0.2% «Ramsinks-2M» + 0.3% foam gun Defoam (at B/C = 0.33) = 12.93 MPa.

The obtained results show that we have developed two newest compositions of oil-well cement B and V.

The use of «RAN-M» hydrophobic cement slurry will significantly improve the properties of cement materials, which in general should lead to the avoidance of overflow and make it possible to significantly reduce the migration of formation fluids.

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PREPARATION OF MULTIFUNCTIONAL LAYERED OXIDE REE-CONTAINING MATERIALS

The aim of this work was to study the processes of cooperative interactions among the structural components during the formation of the layered cation-structured perovskite-like oxide phases of rare earth and transition elements at the preparatory stages using the nitrates of elements with different electronic structure and thermal activation. Stages of such transformations and patterns of phase formation were established; factors of influence and their determinant ability were clarified; a number of physical and chemical properties of the formed intermediate phases (coordination of lanthanides nitrates) were studied.

Keywords: *rare earth elements, alkaline metals, ammonium, alkaline earth metals, nitrates, complex formation, water-salt systems, properties.*

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ПРИГОТУВАННЯ БАГАТОФУНКЦІОНАЛЬНИХ ШАРУВАТИХ ОКСИДНИХ РЗЕ-ВМІСНИХ МАТЕРІАЛІВ

Метою проведення цієї роботи було дослідження кооперативних процесів взаємодії між структурними компонентами у ході формування шаруватих катіоновпорядкованих перовскітоподібних оксидних фаз рідкісноземельних і перехідних елементів у підготовчих стадіях з використанням нітратів елементів різної електронної структури і тепловою активацією. Встановлено стадійність таких перетворень і закономірності фазоутворення, з'ясовано фактори впливу та їх визначальність, вивчено низку фізико-хімічних властивостей утворюваних проміжних фаз – координаційних нітратів лантаноїдів.

Ключові слова: *рідкісноземельні елементи, лужні метали, амоній, лужноземельні метали, нітрати, комплексоутворення, водно-сольові системи, властивості.*

Introduction. Complex layered perovskite-like oxides $ALnTiO_4$, $A_2Ln_2Ti_3O_{10}$, $ALnNb_2O_7$, $ALnTa_2O_7$ (A – Li, Na, K, Rb, Cs, H; Ln – La, Nd) are crystalline compounds where layers of the perovskite structure are alternating with layers having different type of structure. Recently, the mentioned phases have attracted significant attention due to their photo-catalytic properties [1, 2]. They can also be used as precursors to obtain other perovskite-like compounds by means of ion exchange or certain topochemical transformations. It was found that alkaline forms of such perovskites in the aqueous medium can perform replacement of cations by protons and hydration (rooting of water molecules in the interlayer space). These effects lead to substantial changes in physical and chemical properties of photo-catalysts.

The composition of layered perovskite oxides may include several different metals [1–5], and depending on their nature and stoichiometry, these objects can display a wide range of physical and chemical properties. Normally, perovskite-like oxides are semiconducting materials.

It was found that composition systems based on TiO_2 due to internal photoelectric effect (in two ways: either on the electrode surface or in the suspension form) are able to perform:

– photolysis of water

$H_2O + h\nu \rightarrow H_2 + O_2$, that can be used to accumulate (to converse) solar energy in the form of hydrogen, environmentally friendly fuel;

– decomposition of toxic organic impurities in solutions, gaseous media

CxHyOz + hν → CO2 + H2O, that can be taken as a basis for purification of the latter.

The main goals of current research are the spectral action shift from the ultraviolet to the visible region and improving the quantum efficiency due to suppressing undesirable secondary processes. Besides the new materials synthesis, among the main fields of research, modification of the known photo-catalysts by means of doping and formulation of composites should also be called.

Analysis of recent sources of research and publications. Dependencies of photo-induced hydrogen release from suspensions of layered perovskite-like oxides A2Ln2Ti3O10 and ALnTiO4 (A – H, Li, Na, K; Ln – La, Nd), ANdTa2O7 (A – H, Li, Na, K, Rb, Cs) and ANdNb2O7 (A – Rb, Cs) rate on cationic composition were determined [2]. It was found that the maximum activity is displayed by tantalate RbNdTa2O7, referring to Dion-Jacobson phases.

These layered oxides are nano-structured objects. High mobility of their interlayer cations permits wide variations in the composition of such phases and thus affects their optical properties, electronic structure and photo-catalytic activity. Some layered oxides are capable of reversible intercalation of water molecules into the interlayer space that can both cause an increase in the efficient specific surface of photo-catalyst and facilitate spatial separation of reduction-oxidation centers.

There are several ways of forming perovskites [1]. By means of the high-temperature solid phase synthesis method only those phases can be obtained, whose formation is thermodynamically advantageous at a given temperature. Many layered oxides can be synthesized by means of ion exchange reactions and applying molecular and coordination precursors, including nitrate-based ones by means of «soft» chemistry methods.

Unsolved aspects of the problem statement. The authors are studying the possibility of forming layered perovskite-like oxides that relate to the Ruddlesden-Popper phases ($A_2Ln_2Ti_3O_{10}$ and $ALnTiO_4$, where A – H, Li, Na, K, Rb, Cs; Ln – La, Nd) and Dion-Jacobson ($ANdTa_2O_7$, where A – H, Li, Na, K, Rb, Cs and $ANdNb_2O_7$, where A – Rb, Cs), using nitrate coordination REE-containing precursors, whose synthesis is a difficult and extremely urgent task as of today both in the scientific and in the applied respect.

Photo-catalytic activity of the samples is under study in terms of their composition, method of obtaining, structure, nature of their interaction with water based on the results of the available scientific information and a number of features and patterns of behavior of structural elements identified by the authors in multi-component systems at different stages of the preparatory process, in different states of aggregation, in a variety of concentrations and temperature ranges.

In the process of forming the above multifunctional materials using nitrate REE-containing precursors at the stages of preparation, performance, monitoring, at the technological schemes improvement, problems arise, related to the lack of generalized, systemic information on the rare earth elements' complexing ability. It reduces the possibility of intrinsic comprehension, interpretation of the respective chemical transformations mechanisms.

The available information is mostly relating to research of lanthanum and cerium nitrates in triple water-salt systems at low temperature (10 – 30 °C); diagrams describing $Ln(NO_3)_3$ – $MeNO_3$ state within the concentration range of 30 mol. % of Ln $(NO_3)_3$; studying the possibility of extracting Ln^{3+} ions from molted nitrates of elements belonging to IA, IIA groups of the periodic system, their eutectic mixtures. Preparative methods of their obtaining have acquired the most extensiveuse, the methods have been tested under the conditions different from those of target products synthesis. These data do not provide complete answers about the laws and features of phase formation in such systems and do not indicate ways of solving the above problem tasks.

Aim and objectives of the study. To assess the possibility of managing the above processes and to obtain materials with reproducible properties, it is necessary to study $Me(NO_3)_x$ – $Ln(NO_3)_3$ – H_2O (25 – 100 °C) as a model system, where Ln – Y^{3+}, La^{3+} – Lu^{3+}; Me – Li^+ – Cs^+, NH_4^+, x = 1; Mg^{2+} – Ba^{2+}, x = 2 – are system components that define technical specifications of the synthesis product or are used as additives to modify its physical properties.

Experimental procedure. The systems study is performed isothermally (at 25, 50, 65, 100 °C) using additives according to the procedure described in [6, 7]. Phase equilibrium was obtained within 1–2 days. As the source salts, hydrated and anhydrous nitrates of these elements of PA («pure for analysis») mark were used. The temperature interval selection was determined by the existence intervals of the initial components' crystalhydrate forms.

Chemical analysis of liquid, solid phases and «residues» was performed to determine the content of Ln^{3+}, Mg^{2+}, Ca^{2+} – Ba^{2+}, nitrogen ions [8]. Content of Ln^{3+} was determined by means of trilonometry; Mg^{2+} – by the volumetric method; Ca^{2+} – Ba^{2+} – using complexometry titration of the substituent in the filtrate freed from Ln^{3+} by means of ammonium buffer; nitrogen - by stripping; Me^+, NH_4^+ ions – by calculation of the difference, based on the total content of nitrates and partly on the dry residue.

The data obtained was offset against the salt content for individual ions and according to the correspondence principle was applied to the solubility diagram. Graphical display of solid phases, formed in the system, was performed according to Schreinemakers [7]. Their identity was confirmed by chemical, crystal optical, X-ray phase, X-ray structural, infrared spectroscopic, laser SHG, thermographic analysis and other methods.

Results and discussion. Experimental data of the studied systems are generalized and summarized in table 1. They permitted to build the relevant polythermal spatial solubility diagrams for ternary systems. On their basis, the following was established: nature of interaction between the structural components of the systems; quantity, composition, solubility nature, temperature and concentration limits of starting materials and new phases formation; eutonic and transition points structure; in the areas of phase coexistence the positions of divariant equilibria lines were defined; the choice of optimal conditions for coordination nitrates synthesis was made, their crystals growth forms were studied. The obtained data permit to identify phases, to make quantitative calculations in the processes of evaporation and crystallization for similar objects.

Most systems at 25 – 100 °C (under conditions of the solutions existence) are characterized by the formation of new coordination compounds of REE. The compositions of non-variant points of the relevant solubility isotherms correspond to concentration limits of saturated solutions which complex nitrates are extracted from. Difficulty of transformations in nitrate precursors of cerium subgroup elements (La – Sm), Li^+ – Cs^+, NH_4^+, Mg^{2+} during thermal activation was found in our previous studies [8–10, 5] and by the results of X-ray diffraction study of REE coordination nitrates [11, 12] and, as an example, are vividly illustrated by the data of rubidium solubility polytherm (Fig. 1) and by the data of X-R lanthanide complex compounds (table 2).

All the newly-detected phases are synthesized in the mono-crystal form. They have an isometric shape, sizing 4 – 30 mm. Their composition, atomic crystal structure, forms of coordination Ln polyhedrons, types of ligands coordination, a number of their properties are investigated using the set of the above physical and chemical methods. The experimental data are consistent with the results of the previous X-ray studies of potassium lanthanum nitrates, as well as praseodymium, lanthanum-magnesium nitrate [13] studied by means of neutron diffraction analysis.

The performed study permitted to integrate data on the nature and patterns of structural components' chemical interaction, heterogeneous equilibrium (25 – 100 °C) in water-salt systems of lanthanide nitrates, yttrium and elements belonging to IA, IIA groups of the periodic system, ammonium, existing types of compounds, conditions of their formation, it permitted to submit their formulas in an integrated form; to set limits of their iso-stoichiometry and schemes of composition and structure transfer in natural series Y^{3+}, La^{3+} – Lu^{3+}; Li^+ – Cs^+, NH_4^+; Mg^{2+} – Ba^{2+}; the decisive role of the central Ln atom's nonmonotonic change of Ln properties in the process of complexing was defined.

The greatest number of compounds is formed by cerium subgroup elements $Na_2[Ln(NO_3)_5]·H_2O$ (Ln – La – Sm), $Me_2[Ln(NO_3)_5(H_2O)_2]·nH_2O$ (Me – K Ln – La – Nd n=0; Me – Rb Ln – La, Ce n=0; Me – Cs Ln – La – Nd n=0; Me – NH_4^+ Ln – La n=1, 2), $Rb_5[Ln_2(NO_3)_{11}]·H_2O$ (Ln – Pr – Sm), $Me_3[Ln_2(NO_3)_9]·nH_2O$ (Me – Li n=3, Me – K, Rb, NH_4^+ n=0, 1 Ln – La – Sm), $Cs[Ln(NO_3)_4(H_2O)_3]$ (Ln – Pr – Sm).

In the temperature interval 25 – 100 °C yttrium subgroup elements only form compounds with KNO_3, $RbNO_3$, $CsNO_3$, NH_4NO_3 – $K[Ln(NO_3)_4(H_2O)_2]$ (Ln –Y, Gd – Lu), $M[Ln(NO_3)_4(H_2O)_2]·H_2O$ (Me – Rb, Cs, NH_4^+; Ln – Y, Gd – Lu.

Information on the nature of the interaction in the systems of cerium subgroup elements nitrates and Mg, Ca, Sr, Ba indicates that only in magnesium systems in the range of the studied temperatures congruently soluble $[Mg(H_2O)_6]_3[Ln(NO_3)_6]_2·6H_2O$ [5] are formed. In other systems new solid phases are not formed (eutonic type systems).

In the studied water-salt systems complexing mechanism can be explained from the standpoint of competitive substitution of water molecules in the immediate vicinity of Ln^{3+} for NO_3^--groups. The degree of substitution completeness depends on the nature of Ln^{3+}, the impact of disordering effect on these processes, on the solutions structure of the available single- and double-charged cations Li^+ – Cs^+, NH_4^+; Mg^{2+}, Ca^{2+}, Sr^{2+}, Ba^{2+}, the nature of the structural components' thermal motion, properties of electron donor oxygen atoms and spatial structure of the ligand, concentration of anions, amount of solvent. Significant influence of heat factor on these processes and their stages is revealed. The presence of certain temperature values at the beginning of complex compounds evolution into the solid phase indicates the existence of an energy barrier and the necessity for some extra energy to make such transformations possible.

Table 1 – Phase balance and coordination compounds in ternary nitrate water-salt systems of alkaline metals and neodymium at 25 – 100 °C

Systems of $MeNO_3 – Nd(NO_3)_3 – H_2O$			
Me – alkaline metals	Complex nitrate composition	Temperature interval of crystallization, °C	Nature of complex nitrate solubility
Li	Eutonic type system at 25 – 50 °C		
	$Li_3[Nd_2(NO_3)_9]·3H_2O$	65	incongruent
		100	congruent
Na	Eutonic type system at 25 °C		
	$Na_2[Nd(NO_3)_5]·H_2O$	50–100	incongruent
K	Eutonic type system at 25 °C		
	$K_2[Nd(NO_3)_5(H_2O)_2]$	50	incongruent
	$K_3[Nd_2(NO_3)_9]·H_2O$	50	incongruent
	$K_3[Nd_2(NO_3)_9]·H_2O$	65–100	congruent
Rb	$Rb_5[Nd_2(NO_3)_{11}]·H_2O$	25	incongruent
		50	congruent
	$Rb_3[Nd_2(NO_3)_9]·H_2O$	50	incongruent
	$Rb_3[Nd_2(NO_3)_9]·H_2O$	65–100	congruent
Cs	$Cs_2[Nd(NO_3)_5(H_2O)_2]$	25–65	incongruent
	$Cs_2[Nd(NO_3)_5(H_2O)_2]$	100	incongruent
	$Cs[Nd(NO_3)_4(H_2O)_3]$	50–65	incongruent

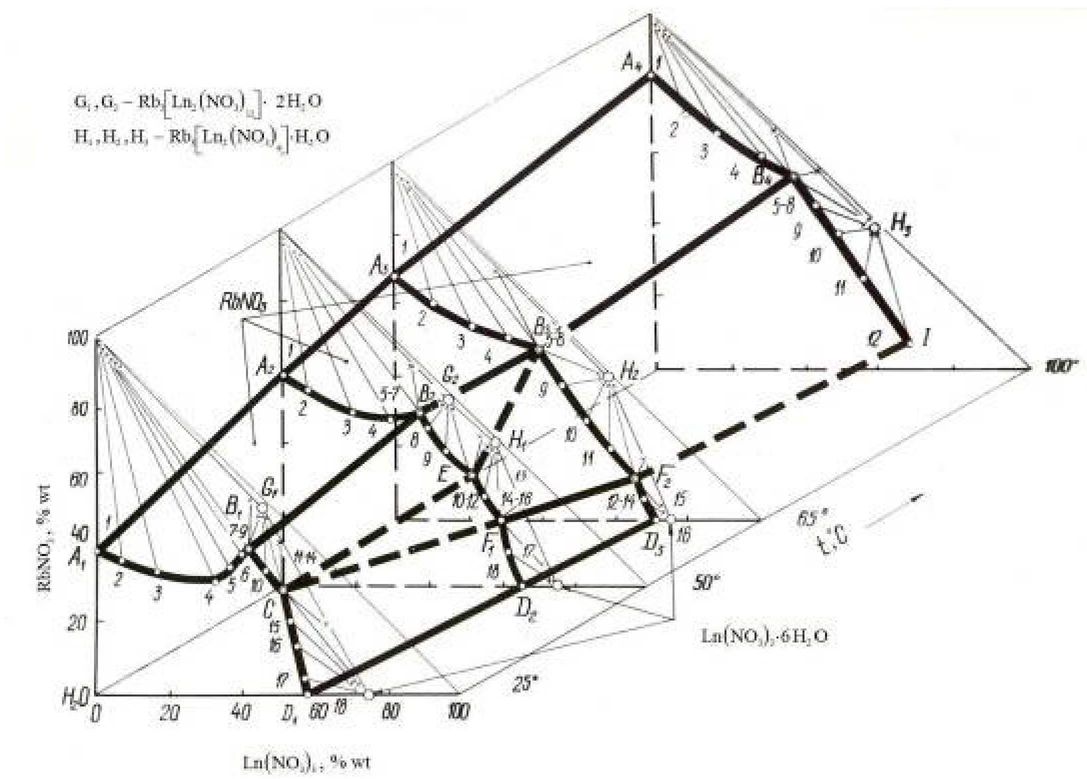


Figure 1– Solubility polytherm of $RbNO_3 – Ln(NO_3)_3 – H_2O$ (Ln – Pr ÷ Sm) system

Table 2 – X-ray data on the structure of the newly formed coordination alkaline nitrates of Nd

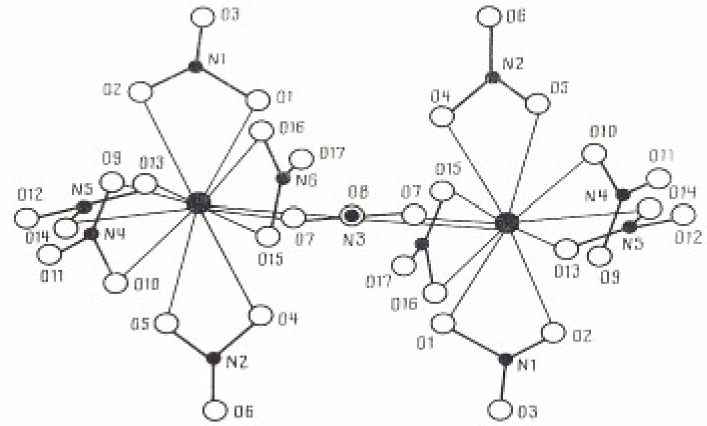


Figure 2 – Binuclear complexes $[Nd_2(NO_3)_{11}]^{5-}$ in the structure of $Rb_5[Nd_2(NO_3)_{11}]\cdot H_2O$

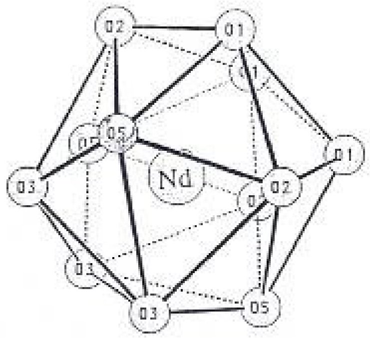
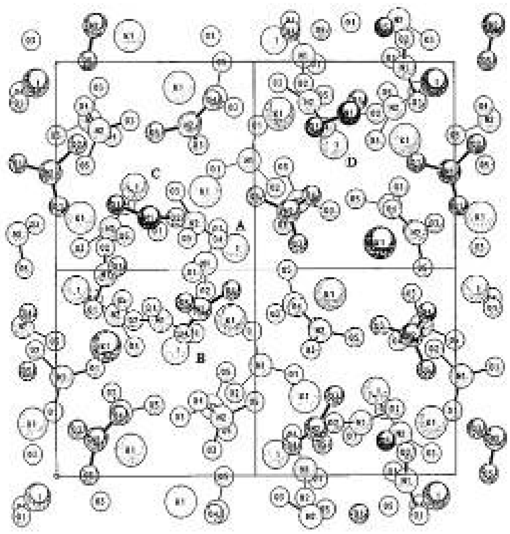
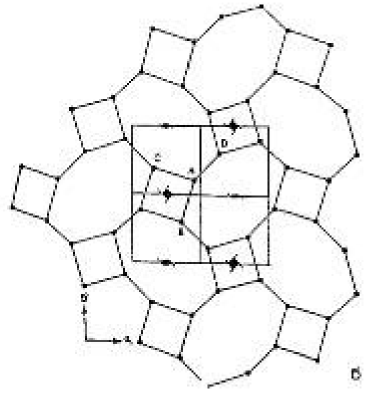


Figure 3 – Coordination Nd polyhedron in the structure of $Rb_3[Nd_2(NO_3)_9]\cdot H_2O$



a)



b)

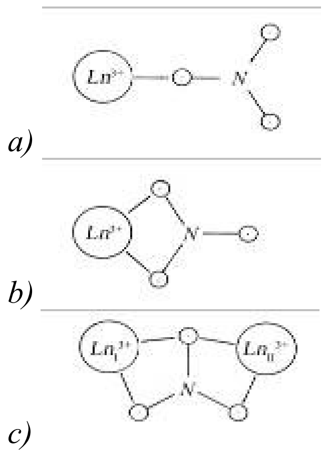


Figure 4 – Three types of coordinating by nitrate groups lanthanides in coordination nitrates structures:
a – monodentate;
b – symmetrical bidentate;
c – symmetrical bridge bidentate

Figure 5 – Structure projection of $Me_3[Nd_2(NO_3)_9]\cdot H_2O$ (Me – K^+, NH_4^+, Rb^+) in xy plane (*a*) and its schematic diagram (*b*)

Differences are determined in the complexing ability of cerium and yttrium subgroups elements, Y, and REE in the middle of the first subgroup. In the formation of nitrate complexes the requirements of symmetry are largely carried out, and small sized planar ligand NO_3^- is «convenient» for the formation of high-symmetric environment of Ln^{3+} ions (Fig. 2). The basis of the compounds structure is presented by rare earth coordination polyhedrons (Fig. 3) that are somehow or other linked in space [11, 12]. Water plays an important role, coordinatedly saturating complexing ions, and providing additional contacts between the complexes in the structure due to hydrogen bonds. For Ln^{3+} complexing ions the tendency is revealed to form a limited number of coordination polyhedrons types, three types of NO_3^--ligands coordination (Fig. 4). This leads to the formation of both isolated complexes and their polymerization into the dual-core ones, chains, frames (Fig. 5).

The benefits of using this type of precursors are indicated by the existence of sufficiently representative segment (a whole class) of coordination REE-containing nitrate compounds of alkaline metals [8] and magnesium [5], identifying among them compounds groups, isotype by composition and structure, that are relevant representatives of lanthanides series, a series of alkaline metals , manifesting a set of technologically valuable properties: a) high solubility and compatibility with most components; b) sufficiently broad temperature range of the complex nitrates existence; c) congruent nature of transformations for most compounds of Li^+, Na^+, K^+, NH_4^+, Rb^+, Mg^{2+} both in solution and in the molten state; d) detection of their high activity by reactive particles obtained by means of the solvent thermolysis, and also nanoscale and uniform in size and morphology; e) existence of a wide range of ways, methods and technical means to activate these processes. Attention should be paid to the fact that now combined transformation methods with special requirements have acquired more widespread together with rapid synthesis methods with combined ways of activating systems and mass production.

Conclusion. Comprehensive study provides a reliable picture of tendencies in compatible behavior of nitrate REE precursors' components, alkaline, alkaline-earth metals at the preparatory stages of layered perovskite-like phase formation with thermal activation. Stages of such transformations and patterns of phase formation were revealed, factors of influence and their paramount importance were clarified, a number of physical and chemical properties of the intermediate phases formed, coordination lanthanide nitrates, was studied. The integrated data allow finding out the mechanisms, kinetics of structural components transformations in similar technological objects, and enabling translating the obtained set of knowledge into the procedure of controlled synthesis of new schemes for obtaining oxide REE-containing multifunctional materials with renewable structure-sensitive performance.

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