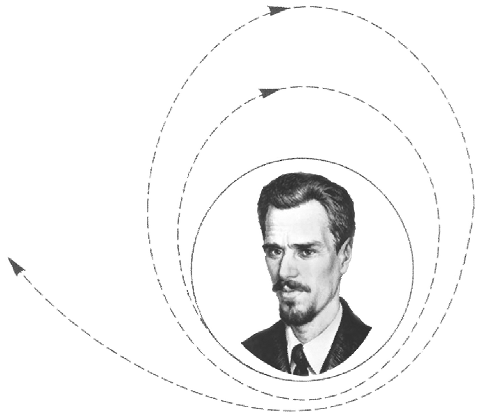
ISSN 2409-9074



ЗБІРНИК НАУКОВИХ ПРАЦЬ

Серія: ГАЛУЗЕВЕ МАШИНОБУДУВАННЯ, БУДІВНИЦТВО

Випуск 1 (48)′ 2017

ACADEMIC JOURNAL

Series: INDUSTRIAL MACHINE BUILDING, CIVIL ENGINEERING

Issue 1 (48)′ 2017

Міністерство освіти і науки України
Полтавський національний технічний університет
імені Юрія Кондратюка

Ministry of Education and Science of Ukraine
Poltava National Technical Yuri Kondratyuk University

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Полтава – 2017

Poltava - 2017

Збірник наукових праць. Серія: Галузеве машинобудування, будівництво / Полтавський національний технічний університет імені Юрія Кондратюка

Збірник наукових праць видається з 1999 р., періодичність – двічі на рік.
Засновник і видавець – Полтавський національний технічний університет імені Юрія Кондратюка.
Свідоцтво про державну реєстрацію КВ 8974 від 15.07.2004 р.
Збірник наукових праць уключений до переліку наукових фахових видань, у яких можуть публікуватися результати дисертаційних робіт (Наказ МОН України №1279 від 6.11.2014 року).
Збірник наукових праць рекомендовано до опублікування вченою радою Полтавського національного технічного університету імені Юрія Кондратюка, протокол № 10 від 24.03.2017 р.

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

Адреса видавця та редакції – Полтавський національний технічний університет імені Юрія Кондратюка
Науково-дослідницька частина, к. 320Ф, Першотравневий проспект, 24, м. Полтава, 36011.
тел.: (05322) 29875;   e-mail: v171@pntu.edu.ua;   www.pntu.edu.ua
Макет та тиражування виконано у поліграфічному центрі
Полтавського національного технічного університету імені Юрія Кондратюка,
Першотравневий проспект, 24, м. Полтава, 36011.
Свідоцтво про внесення суб'єкта видавничої справи до державного реєстру видавців,
виготівників і розповсюджувачів видавничої продукції. Серія ДК, № 3130 від 06.03.2008 р.
Комп'ютерна верстка – В.В. Ільченко. Коректори – Я.В. Новічкова, М.В. Москаленко.
Підписано до друку 27.03.2017 р.
Папір ксерокс. Друк різограф. Формат 60х80 1/8. Ум. друк. арк. – 34,88.
Тираж 300 прим.

Academic journal. Series: Industrial Machine Building, Civil Engineering /
Poltava National Technical Yuri Kondratyuk University

Academic journal was founded in 1999, the publication frequency of the journal is twice a year.
Founder and Publisher is Poltava National Technical Yuri Kondratyuk University.
State Registration Certificate КВ № 8974 dated 15.07.2004.
Academic journal is included into the list of specialized academic publications where graduated thesis results could be presented (Order of Department of Education and Science of Ukraine № 1279 dated 6.11.2014).
Academic journal was recommended for publication by the Academic Board of Poltava National Technical Yuri Kondratyuk University, transactions № 10 of 24.03.2017.
The results of scientific and scientific-technical developments in the sphere of mechanical engineering, automobile transport and mechanization of construction works; designing, erection, operation and reconstruction of structural steels, buildings and structures; its bases and foundations; building physics and energy efficiency of buildings and structures are presented in the collection.
Academic journal is designed for researchers and technologists, postgraduates and senior students.

Address of Publisher and Editorial Board -- Poltava National Technical Yuri Kondratyuk University,
Research Centre, room 320-F, Pershotravnevyi Avenue, 24, Poltava, 36011, Ukraine.
тел.: (05322) 29875; e-mail: v171@pntu.edu.ua; www.pntu.edu.ua
Layout and printing made in the printing center of Poltava National Technical Yuri Kondratyuk University,
Pershotravnevyi Avenue, 24, Poltava, 36011, Ukraine.
State accreditation certificate of publication media, issue ДК № 3130, dated 06.03.2008,
granted by the State Committee for television and radio broadcasting of Ukraine.
Registration certificate of publishing subject in the State Register of Publishers
Manufacturers and Distributors of publishing products. DK Series, № 3130 from 06.03.2008.
Desktop Publishing – V.V. Ilchenko. Corrections – Y.V. Novichkova, M.V. Moskalenko.
Authorize for printing 27.03.2017.
Paper copier. Print rizograf. Format 60x80 1/8. Conventionally printed sheets – 34,88.
Circulation 300 copies.

Devoted to the 120th Birth and to the 75th Death Anniversaries

Kocherga N.K., *PhD, Associate Professor*
Perederiy I.H., *DSc, Professor*
Poltava National Technical Yuriy Kondratyuk University

YURIY KONDRATYUK (OLEXANDR SHARGEY): KNOWN AND UNKNOWN

At the beginning of the 21 century it became clear that the «heavenly» cosmic philosophy of Tsiolkovsky and so called the «planetary» cosmic presented by F.A. Tsander are both to some extend premature, and the «earth» or «ground» cosmic philosophy of Yu.V. Kondratyuk is the most urgent for the Earth because it could realize some peaceful space programs for the benefit of mankind. «The whole universe space is developing not in accordance with Tsiolkovsky's or Tsander's theories but with that of Kondratyuk», said B.I. Romanenko, the scientist, who has been studying the interplanetary space for many years and who was Yuriy Kondratyuk's colleague and biographer. In the history of national science it is difficult to find more dramatic fortune than the life and career of the outstanding self-taught scholar from Poltava.

One of the cosmism pioneers and space era forerunners, Olexandr Ignatovych Shargey, lived and worked for many years under another name: Yuriy Vasyliovych Kondratyuk. During the Civil War in Russian Empire he had to take someone's name though he did not commit any crime against people or country, and he had never taken his real name back, as it was the period of Stalin's repressions.

The name of Yuriy Kondratyuk had only gained the worldwide recognition in the period of American Moon exploration. On March, 31, 1969, American magazine «Life» published the article by John Hubolt, the NASA research center employee. He wrote: «When I was watching the Apollo-11 launch and the first manned flight to the Moon, I recalled the engineer, whose dream had been broken by people's skepticism». Hubolt wrote about the Ukrainian self-taught engineer Yuriy Kondratyuk, who had calculated the LOR scheme 50 years before and it became the best one for reaching the Moon's surface [1].

Soviet people had known nothing about Yu.V. Kondratyuk's outstanding contribution to the space exploration development till the end of the 70s, when his name returned from oblivion… [2]. Not knowing about the ideas of Tsiolkovsky and other founders of the space flights theory, he answered the questions already solved by them, but he was often doing it in his own way, thus confirming their results. However, he went further, particularly in the problems of developing the economizing means of launching spacecraft from the Earth's surface, original rocket engine and interplanetary spacecraft design. He tried to achieve reliable control and stability of a space flight. In his Petrograd manuscript Shargey went on developing the idea of solar power usage which he had started in Poltava. He offered to utilize thin-walled reflectors of various shapes, as means for installation near a spacecraft together with concentrated solar heat receivers connected with water tubes. According to Shargey, water in heated receivers is to be split into oxygen and hydrogen, i.e. fuel, (oxidizer and fuel) for the rocket engine. He had designed reflector structure and estimated the sunlight intensity.

By the autumn of 1919 he had finished the second manuscript, which was called «To Those Who Will Read for the Purpose of Building». This was his most multi-aspect work dated with 1918-1919. The manuscript was prepared for publication, but it was first published only in 1964 in the book «Pioneers of Rocket Techniques»: Kibalchich, Tsyolkovsky, Tsander, Kondratyuk. Selected Works. («Science», Moscow) [3].

In this work Shargey further developed economizing methods of launching a «shell» from the Earth; he made his suggestions concerning flights stabilization with the help of gyroscopes; shell control; multi-usage of solar energy with the help of light, reflectors which can be mounted in the space both for the interplanetary spacecraft's purposes and for «earth» recycling. Here the author expresses the idea of using reflectors for «wireless telegraphy», i.e. the idea is forecasting placement of beam reception and radiation antennas. The design issues of the «shell» and its engine get their further development; an airlock is offered to contact with the open space, and it is recommended, that one should get out of the shell's chamber in the suit, more or less identical to diving suits, having some air reserve» i.e. the idea of a space suit is expressed. The safest crew location in the «shell» at departure is suggested (when heavy acceleration is taking place) depending on the movement direction: their location in separate «forms» – lodgments (cradles) perpendicularly to the movement direction. Complications, which occur when flying at high speed through the atmosphere, are considred; and the ways of avoiding «shell» overheating and using atmosphere for the «aerodynamic descent» are indicated.

In his «Theory of Flight» Olexandr Shargey brilliantly predicted stages of space exploration both for the present day and for the future. The most important of them was the flight scheme from Earth to the solar system planets, suggested by the scientist, using small take-off and landing module, that scheme is now called the «snail route of Kondratyuk», which has been successfully mastered by earthmen.

The young scientist was far ahead of foreign scholars of that time, working in the field of rockets and interplanetary flights. He had made a new step in rocket science and technology development. This research was carried out after the first works of Tsiolkovsky, but not overlapping them and made independently of them, and finally confirmed the complete priority of Russian science and engineering in the space branch.

The deep content of these manuscripts and the original study metho, used in them, have presented us with the undisputed fact, that we are dealing with historical documents that should be forever inscribed into the glorious chronicles of science.

In January, 1929, Yu.V. Kondratyuk published (on his own money) the main book of his life, which was to reach the grateful descendants, «The Interplanetary Space Conquest» [4] edited with a run of 2000 copies, where he had presented the study of interplanetary flights in the best way, comparing to all available at the time of native and foreign literature. Professor Vietchinkin said in his review that those papers contained a number of new issues of paramount importance which other authors did not even mention about. They include:

1. Suggestion to take advantage of different substances burning in ozone, but not in oxygen, which raises the combustion temperature;

2. Suggestion to use solid fuel (lithium, boron, aluminum, magnesium, silicon) in addition to gases, both to raise the combustion temperature and to burn tanks after emptying them from the liquid fuel process themselves and placing into the oven;

3. Kondratyuk was the first to give the formula considering that fuel and oxygen cells weight effects the total rocket weight, and proved that the rocket that does not reset and burn its cells on the move, can not overcome the Earth's gravity;

4. Yu. V. Kondratyuk not only pointed out the necessity of applying wings on the rocket, but he had also performed detailed study of the conditions under which the wings would be useful, what would be in that case the oblique angles of the rocket's trajectory towards the horizon, and indicated the best rocket reaction force at flying in the air; it appeared to be equal to the initial rocket weight [5].

In June, 1930, as a result false denunciation, Kondratyuk together with Horchakov and other co-workers were arrested, accused of wrecking and sentenced to three years of labor camps, and later he was exiled to the place of mining equipment plant construction near Novosibirsk, where he was working until August, 1932, having managed to get a patent and a copyright certificate in the field of mining equipment.

Judicial Division for Criminal Cases at the Supreme Council of RSFSR, by its decision number CB-70-8 on March, 26, 1970, rehabilitated Kondratyuk for absence of crime in the act. In 1932 Yu.V. Kondratyuk moved to Moscow for working in the wind energy field. By this time he had already possessed nine patents and copyright certificates for his inventions.

On the 5th of July, 1941, Yu. V. Kondratyuk joined Moscow people's emergency volunteer corps. Private telephone operator Yuriy Kondratyuk participated in the combat actions in the area of Kaluga, Smolensk region, defended Kaluga, the hometown of Tsiolkovsky, Maloyaroslavets, fought near Tula. In the early of December, 1941, Yu. V. Kondratyuk was the communication officer of the 1st Squadron, belonging to the 49th Army of the Western Front. During the unsuccessful offensive at Bolkhov, while seizing springboard on the west bank of the Oka river, near the village of Krivtsovo, in the interval between 23 and 25, February, 1942, Yuri Kondratyuk died, providing communication among the eastern bank and the springboard.

The decision of the International Astronomical Union was to name a crater on the dark side of the Moon in honour of Yuriy Kondratyuk. One of the minor planets, asteroid under the number 3084, opened by M.S. Chernykh, a Crimean astronomer, is also named in his honour. The International Academy of Astronautics recommended to inscribe the name of Yu.V. Kondratyuk into the list of 78 world scientists, submitted for inclusion in the International Space Glory Hall [6].

The name of Yuriy Kondratyuk (Olexandr Shargey) is included in the Memory Book of Ukraine.

By the decision of the Ukrainian Government, on June, 21, 1997, on the occasion of the 100th anniversary of the prominent space theorist, brilliant engineer - inventor and innovator, his name was granted to Poltava State Technical University, whose scientists were developing their research projects on improving the construction durability and reliability, thus implementing the ideas of Yuriy Kondratyuk (Olexandr Shargey) into reality. These lines taken from the University Anthem vividly express the place and role of Yuriy Kondratyuk in the students life:

Kondratyuk, the glorious, has given you his name.
We're proud to belong to that family of fame.
The best-loved University, we'll keep in our hearts
Spirit of the sciences, spirit of the arts!

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UDC 621.43.011

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IMPROVING OF TWO-LEVEL CAR HAULER STABILITY

To improve the car hauler lateral stability indicatin optimal cargo location parameters, elastically mounted on a platform, mathematical model of its motion was developed. At the same time fluctuations in cargo and car hauler were considered. Simulation results determined that for «hauler – cargo» system consideration of cargo elastic properties leads to a significant decrease in the frequency and amplitude of the system vertical oscillations. Therefore, the presence of the cargo can be regarded as a dynamic passive damping (in the case of a correct choice and design of layout parameters). It is proposed to reduce the distance between the cargo and the upper platform by determination of maximum values of the cargo oscillations amplitudes. In turn, the reduction of platform height reduces center gravity system height, improves the stability of car hauler.

***Keywords:** car hauler, dynamic processes, fluctuations, platform, stability.*

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ПІДВИЩЕННЯ СТІЙКОСТІ ДВОРІВНЕВОГО АВТОЕВАКУАТОРА

Для підвищення поперечної стійкості автоевакуатора шляхом визначення оптимальних параметрів розташування вантажу, що пружно закріплений на платформі, розроблено математичну модель його руху. При цьому враховано коливання вантажу та автоевакуатора. За результатами моделювання встановлено, що для системи «евакуатор – вантаж» урахування пружних властивостей вантажу приводить до істотного зменшення частоти та амплітуди вертикальних коливань системи. Отже, з'ясовано, що наявність вантажу можна розглядати як засіб динамічного пасивного гасіння коливань (у разі правильного вибору конструктивних і компонувальних параметрів). Запропоновано зменшувати відстань між вантажем і верхньою платформою завдяки визначенню величин максимальних амплітуд коливання вантажів. У свою чергу зниження висоти платформи зменшує висоту центра ваги системи, що підвищує стійкість автоевакуатора.

***Ключові слова:** автоевакуатор, динамічні процеси, коливання, платформа, стійкість.*

Introduction. Stability – is one of the factors which provides driving safety. It also allows to increase speed.

Electric systems are used to control stability of vehicle with a high center of mass, But it impairs vehicle course stability – deviation from chosen trajectory for decreasing lateral acceleration (which leads to rollover) [9]. The mass center's height significantly impacts on transverse stability of vehicle. This feature also concerns to two-level car hauler with elastically fixed weights, which fluctuate while driving.

Vehicles fluctuations are caused by road defects. Vehicle undergoing are low-freq. (15 – 18 Hz) and high-freq. fluctuations. Hard fixed weights fluctuates with high-freqs and elastically fixed weights fluctuates with low-freqs. Load of soft fixed masses is passing by elastic suspension elements [10].

It was established in research [11] that transverse fluctuations impair the vehicle lateral stability and depend on vehicle’s load. When vehicle drives elastically fixed weight, the height of mass center is changing. It leads to deterioration of lateral stability while performing maneuvers and the platform heeling.

Movable mechanical system with elastic fixed weights is quite common in vehicle building.

Car hauler is also movable mechanical system with elastic fixed weight. So it is essential to create a mathematical model of a car hauler with elastic fixed cars which fluctuates while driving objective research and analysis dynamical processes. This model should reproduce layout features and interaction of individual elements or parts of real vibrating system. The fixing method characterizes lateral stability and driving smoothness on the road. The typical fixing method provides hard fixed wheels by fixing straps. The car with hard fixed wheels generates vibration and fluctuations which impairs car hauler stability and driving smoothness. So the dynamic of vertical fluctuations should be researched considering additional spring-loaded weight.

Analysis of the last research sources and publications. The truck train stability improvements were researched by famous scientists, such as D.A. Antonov, P.V. Aksenov, V.P. Volkov, A.S. Litvinov, M.A. Podryhalo, V.P. Sakhno, A.P. Soltus, G.A. Smirnov [10 –14].

Many scientific works deal with researches of vertical fluctuations dynamic [1–10]. The most part of researches is about cars and trains for general purpose. There are no fundamental investigations of vehicle moving processes with elastically fixed weights, except semi trailer auto transporter [11], where cars are regarded as separate discrete mass points, elastically fixed on a platform.

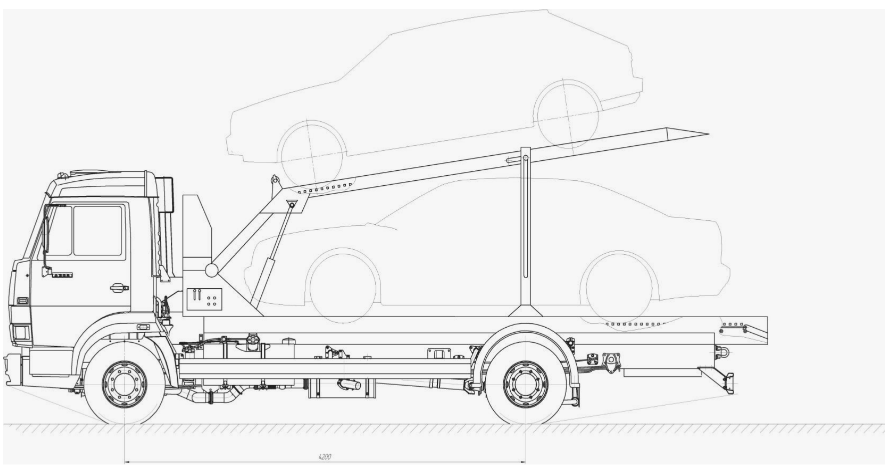

Figure 1 – Doubled car hauler

Car hauler is a complex mechanical system with lots of freedom degree. That's why it is difficult to calculate some parameters of system.

The main problem is optimizing and damping of fluctuations considering the standards ICO 2631-74 and GOST 12.1.012-78, which standardize fluctuations and vibrations and other parameters which provide stability and driving softness.

Specifing unsolved aspects of the problem. Source analisys proves that influence of elastic weight on vertical fluctuations dynamic and two level car hauler stability is not researched.

Objectives setting. The article target is researching of two level car hauler's vertical dynamic (fig. 1); mathematical description of its simplified circuit (fig. 2) for minimizing the amplitude of weight fluctuations by its rational positioning on a platform; improving a lateral stability of vehicle by mass center height decreasing.

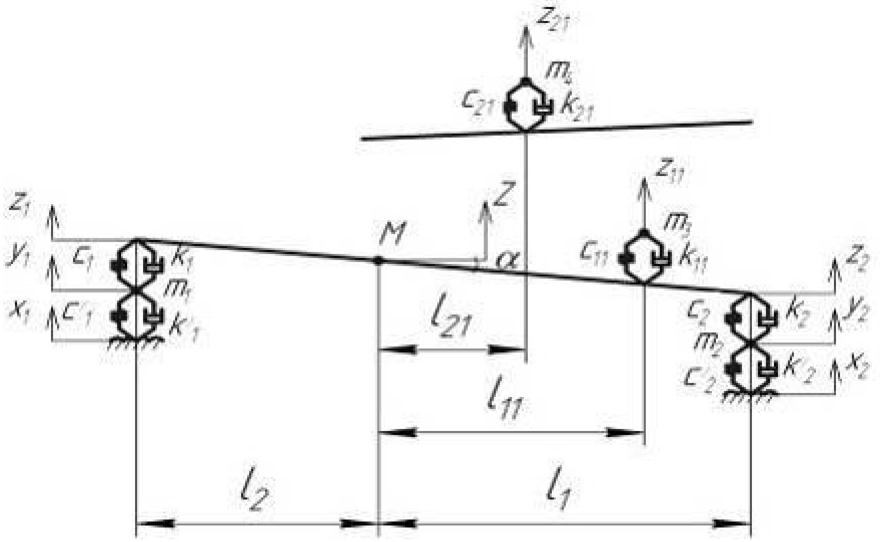

Figure 2 – Calculating circuit of car hauler

The mains and researches. Mathematical model of the fluctuations is described by differential equations based on Lagrange 2nd kind equations. It is based on kinematic and dynamic analysis of car hauler structures:

$$\frac{d}{dt}\frac{\partial L}{\partial \dot{q}_i} = \frac{\partial L}{\partial q_i} - \frac{\partial R}{\partial \dot{q}_i}, \tag{1}$$

where $L = T - U$ – Lagrange function;

T, U, R – in accordance kinematic, potential energy and dissipative function;

q_j – generalized coordinate.

Lagrange 2nd kind equations can be displayed like:

$$\frac{d}{dt}\frac{\partial T}{\partial \dot{q}_i} - \frac{d}{dt}\frac{\partial U}{\partial \dot{q}_i} = \frac{\partial T}{\partial q_i} - \frac{\partial U}{\partial q_i} - \frac{\partial R}{\partial \dot{q}_i}. \tag{2}$$

Shifting is calculated considering relative position of static equilibrium. Then equations type stays the same with(out) the weight force which can be unspecified. Lets take function (1) for car hauler movement. For Z axis elastic vertical movements were chosen and for Y axis inelastic vertical movements were chosen in accordance with fig. 2. Disturbing function X selected profile of single inequality as half sine wave [5]. Individual irregularities vibrations caused by the road vary according to a sinusoidal law. The differences are quantitative and insignificant.

Mark: m_1 , m_2 – inelastic masses of appropriate axles of car hauler;

m_3, m_4 – masses of elastic weights;

M, I – elastic mass of car hauler and its inertia moment of the central axis, that is perpendicular to the plane of the figure;

c_1 , c_2 , c'_1, c'_2 ,c_{11}, c_{21}, k_1 , k_2 , k'_1, k'_2, k_{11}, k_{21} – appropriate equivalent stiffness of suspensions and tires and their coefficient of viscous friction;

l_1 , l_2 , l_{11}, l_{12} – appropriate geometrical parameters of car hauler.

Define the kinetic energy of the system:

$$T = \frac{1}{2}M \cdot \dot{z}^2 + \frac{1}{2}I \cdot \dot{\alpha}^2 + \frac{1}{2}m_1 \cdot \dot{y}_1^2 + \frac{1}{2}m_2 \cdot \dot{y}_2^2 + \frac{1}{2}m_3 \cdot \dot{z}_{11}^2 + \frac{1}{2}m_4 \cdot \dot{z}_{21}^2, \tag{3}$$

where is

$$z = \frac{z_1 \cdot l_2 + z_2 \cdot l_1}{l_1 + l_2}, \quad \alpha = \frac{z_1 - z_2}{l_1 + l_2}.$$

Define the potential energy of the system:

$$\begin{aligned}
U &= \frac{1}{2}c_1\left(z_1 - y_1\right)^2 + \frac{1}{2}c'_1\left(y_1 - x_1\right)^2 + \frac{1}{2}c_2\left(z_2 - y_2\right)^2 + \frac{1}{2}c'_2\left(y_2 - x_2\right)^2 + \\
&+ \frac{1}{2}c_{11}\left(z_{11} + \frac{z_1 - z_2}{l_1 + l_2}\cdot l_{11}\right)^2 + \frac{1}{2}c_{21}\left(z_{21} + \frac{z_1 - z_2}{l_1 + l_2}\cdot l_{21}\right)^2 = \\
&= \frac{1}{2}c_1\left(z_1^2 - 2z_1y_1 + y_1^2\right) + \frac{1}{2}c'_1\left(y_1^2 - 2y_1x_1 + x_1^2\right) + \frac{1}{2}c_2\left(z_2^2 - 2z_2y_2 + y_2^2\right) + \\
&+ \frac{1}{2}c'_2\left(y_2^2 - 2y_2x_2 + x_2^2\right) + \frac{1}{2}c_{11}\left(z_{11}^2 + 2z_{11}\alpha\cdot l_{11} + \alpha^2 l_{11}^2\right) + \\
&+ \frac{1}{2}c_{21}\left(z_{21}^2 + 2z_{21}\alpha\cdot l_{21} + \alpha^2 l_{21}^2\right).
\end{aligned} \tag{4}$$

Rayleigh dissipative function:

$$\begin{aligned}
R &= \frac{1}{2}k_1\left(\dot{z}_1 - \dot{y}_1\right)^2 + \frac{1}{2}k'_1\left(\dot{y}_1 - \dot{x}_1\right)^2 + \frac{1}{2}k_2\left(\dot{z}_2 - \dot{y}_2\right)^2 + \frac{1}{2}k'_2\left(\dot{y}_2 - \dot{x}_2\right)^2 + \\
&+ \frac{1}{2}k_{11}\left(\dot{z}_{11} + \frac{\dot{z}_1 - \dot{z}_2}{l_1 + l_2}\cdot l_{11}\right)^2 + \frac{1}{2}k_{21}\left(\dot{z}_{21} + \frac{\dot{z}_1 - \dot{z}_2}{l_1 + l_2}\cdot l_{21}\right)^2 = \\
&= \frac{1}{2}k_1\left(\dot{z}_1^2 - 2\dot{z}_1\dot{y}_1 + \dot{y}_1^2\right) + \frac{1}{2}k'_1\left(\dot{y}_1^2 - 2\dot{y}_1\dot{x}_1 + \dot{x}_1^2\right) + \frac{1}{2}k_2\left(\dot{z}_2^2 - 2\dot{z}_2\dot{y}_2 + \dot{y}_2^2\right) + \\
&+ \frac{1}{2}k'_2\left(\dot{y}_2^2 - 2\dot{y}_2\dot{x}_2 + \dot{x}_2^2\right) + \frac{1}{2}k_{11}\left(\dot{z}_{11}^2 + 2\dot{z}_{11}\dot{\alpha}\cdot l_{11} + \dot{\alpha}^2 l_{11}^2\right) + \\
&+ \frac{1}{2}k_{21}\left(\dot{z}_{21}^2 + 2\dot{z}_{21}\alpha\cdot l_{21} + \dot{\alpha}^2 l_{21}^2\right).
\end{aligned} \tag{5}$$

Derivatives are needed to prepare for the second kind of Lagrange equation:

$$\begin{aligned}
\frac{\partial T}{\partial \dot{z}_1} &= \frac{M}{2\left(l_1 + l_2\right)^2}\cdot\left(2\dot{z}_1\cdot l_2^2 + 2\dot{z}_2\cdot l_2\cdot l_1\right) + \frac{I\left(2\dot{z}_1 - 2\dot{z}_2\right)}{2\left(l_1 + l_2\right)^2} = \\
&= \frac{M\cdot l_2^2 + I}{\left(l_1 + l_2\right)^2}\cdot\dot{z}_1 + \frac{M\cdot l_2\cdot l_1 - I}{\left(l_1 + l_2\right)^2}\cdot\dot{z}_2,
\end{aligned} \tag{6}$$

$$\frac{d}{dt}\frac{\partial T}{\partial \dot{z}_1} = a_{11}\ddot{z}_1 + a_{12}\ddot{z}_2 \tag{7}$$

where

$$a_{11} = \frac{M \cdot l_2^2 + I}{\left(l_1 + l_2\right)^2},\ a_{12} = \frac{M \cdot l_2 \cdot l_1 - I}{\left(l_1 + l_2\right)^2},$$

$$\begin{aligned} \frac{\partial T}{\partial \dot{z}_2} &= \frac{M}{2\left(l_1 + l_2\right)^2}\cdot\left(2\dot{z}_1 \cdot l_2 \cdot l_1 + 2\dot{z}_2 \cdot l_1^2\right) + \frac{I\left(-2\dot{z}_1 + 2\dot{z}_2\right)}{2\left(l_1 + l_2\right)^2} = \\ &= \frac{M \cdot l_2 \cdot l_1 - I}{\left(l_1 + l_2\right)^2}\cdot \dot{z}_1 + \frac{M \cdot l_1^2 + I}{\left(l_1 + l_2\right)^2}\cdot \dot{z}_2, \end{aligned} \tag{8}$$

$$\frac{d}{dt}\frac{\partial T}{\partial \dot{z}_2} = a_{21}\ddot{z}_1 + a_{22}\ddot{z}_2, \tag{9}$$

where

$$a_{21} = \frac{M \cdot l_2 \cdot l_1 - I}{\left(l_1 + l_2\right)^2},\ a_{22} = \frac{M \cdot l_1^2 + I}{\left(l_1 + l_2\right)^2},$$

$$\frac{\partial T}{\partial \dot{y}_1} = \frac{1}{2}\cdot m_1 \cdot 2\dot{y}_1 = m_1\dot{y}_1, \tag{10}$$

$$\frac{d}{dt}\frac{\partial T}{\partial \dot{y}_1} = m_1\ddot{y}_1. \tag{11}$$

Similarly define derivatives:

$$\frac{d}{dt}\frac{\partial T}{\partial \dot{y}_2} = m_2\ddot{y}_2, \tag{12}$$

$$\frac{d}{dt}\frac{\partial T}{\partial \dot{z}_{11}} = m_3\ddot{z}_{11}, \tag{13}$$

$$\frac{d}{dt}\frac{\partial T}{\partial \dot{z}_{21}} = m_4\ddot{z}_{21}. \tag{14}$$

Find the partial derivative of potential energy (4) for the generalized speed dq_i :

$$\frac{\partial U}{\partial \dot{z}_1} = \frac{\partial U}{\partial \dot{z}_2} = \frac{\partial U}{\partial \dot{y}_1} = \frac{\partial U}{\partial \dot{y}_2} = \frac{\partial U}{\partial \dot{z}_{11}} = \frac{\partial U}{\partial \dot{z}_{21}} = 0, \tag{15}$$

then

$$\frac{d}{dt}\frac{\partial U}{\partial \dot{z}_1} = \frac{d}{dt}\frac{\partial U}{\partial \dot{z}_2} = \frac{d}{dt}\frac{\partial U}{\partial \dot{y}_1} = \frac{d}{dt}\frac{\partial U}{\partial \dot{y}_2} = \frac{d}{dt}\frac{\partial U}{\partial \dot{z}_{11}} = \frac{d}{dt}\frac{\partial U}{\partial \dot{z}_{21}} = 0. \tag{16}$$

To compile the right of Lagrange equations of the second kind makes differentiation.

Partial derivatives of kinetic energy T on generalized coordinates q_i equal to:

$$\frac{\partial T}{\partial z_1}=\frac{\partial T}{\partial z_2}=\frac{\partial T}{\partial y_1}=\frac{\partial T}{\partial y_2}=\frac{\partial T}{\partial z_{11}}=\frac{\partial T}{\partial z_{21}}=0, \tag{17}$$

Define the partial derivative of potential energy for the generalized coordinates q_i :

$$\frac{\partial U}{\partial z_1}=\frac{1}{2}c_1\left(2z_1-2y_1\right)=c_1\left(z_1-y_1\right), \tag{18}$$

$$\frac{\partial U}{\partial z_2}=\frac{1}{2}c_2\left(2z_2-2y_2\right)=c_2\left(z_2-y_2\right), \tag{19}$$

$$\frac{\partial U}{\partial y_1}=\frac{1}{2}c_1'\left(2y_1-2x_1\right)=c_1'\left(y_1-x_1\right), \tag{20}$$

$$\frac{\partial U}{\partial y_2}=\frac{1}{2}c_2'\left(2y_2-2x_2\right)=c_2'\left(y_2-x_2\right), \tag{21}$$

$$\frac{\partial U}{\partial z_{11}}=\frac{1}{2}c_{11}\left(2z_{11}+2\alpha\cdot l_{11}\right)=c_{11}\left(z_{11}+\alpha\cdot l_{11}\right), \tag{22}$$

$$\frac{\partial U}{\partial z_{21}}=\frac{1}{2}c_{21}\left(2z_{21}+2\alpha\cdot l_{21}\right)=c_{21}\left(z_{21}+\alpha\cdot l_{21}\right). \tag{23}$$

Next, find the partial derivatives of Rayleigh dissipative function for the generalized speed ∂q_i

$$\frac{\partial R}{\partial \dot{z}_1}=\frac{1}{2}k_1\left(2\dot{z}_1-2\dot{y}_1\right)=k_1\left(\dot{z}_1-\dot{y}_1\right), \tag{24}$$

$$\frac{\partial R}{\partial \dot{z}_2}=\frac{1}{2}k_1\left(2\dot{z}_2-2\dot{y}_2\right)=k_1\left(\dot{z}_2-\dot{y}_2\right), \tag{25}$$

$$\frac{\partial R}{\partial \dot{y}_1}=\frac{1}{2}k_1'\left(2\dot{y}_1-2\dot{x}_1\right)=k_1'\left(\dot{y}_1-\dot{x}_1\right), \tag{26}$$

$$\frac{\partial R}{\partial \dot{y}_2}=\frac{1}{2}k_2'\left(2\dot{y}_2-2\dot{x}_2\right)=k_2'\left(\dot{y}_2-\dot{x}_2\right), \tag{27}$$

$$\frac{\partial R}{\partial \dot{z}_{11}}=\frac{1}{2}k_{11}\left(2\dot{z}_{11}+2\dot{\alpha}\cdot l_{11}\right)=k_{11}\left(\dot{z}_{11}+\dot{\alpha}\cdot l_{11}\right), \tag{28}$$

$$\frac{\partial R}{\partial \dot{z}_{21}}=\frac{1}{2}k_{21}\left(2\dot{z}_{21}+2\dot{\alpha}\cdot l_{21}\right)=k_{21}\left(\dot{z}_{21}+\dot{\alpha}\cdot l_{21}\right). \tag{29}$$

Substituting fragments (3) – (29) into Lagrange equation (2) to get a system of six second order differential equations (30), that describes the vertical oscillations towing considering the elastic weight properties.

$$\begin{cases} a_{11}\ddot{z}_1 + a_{12}\ddot{z}_2 + k_1(\dot{z}_1 - \dot{y}_1) + c_1(z_1 - y_1) = 0 \\ a_{21}\ddot{z}_1 + a_{22}\ddot{z}_2 + k_2(\dot{z}_2 - \dot{y}_2) + c_2(z_2 - y_2) = 0 \\ m_1\ddot{y}_1 + k_1^{/}(\dot{z}_1 - \dot{x}_1) + c_1^{/}(z_1 - x_1) = 0 \\ m_2\ddot{y}_2 + k_2^{/}(\dot{z}_2 - \dot{x}_2) + c_2^{/}(z_2 - x_2) = 0 \\ m_3\ddot{z}_{11} + k_{11}(\dot{z}_{11} + \dot{\alpha}\cdot l_{11}) + c_{11}(z_{11} + \alpha\cdot l_{11}) = 0 \\ m_4\ddot{z}_{21} + k_{21}(\dot{z}_{21} + \dot{\alpha}\cdot l_{21}) + c_{21}(z_{21} + \alpha\cdot l_{21}) = 0. \end{cases} \quad (30)$$

On the basis of rational equations it is possible to determine optimal coordinates for weight placing on vehicles provided minimum fluctuations on its platform. Set next:

$$\omega_1 = \sqrt{\frac{\mathrm{c}_{11}\cdot\alpha\cdot l_{11}}{m_3}}\ ; \qquad \omega_2 = \sqrt{\frac{\mathrm{c}_{21}\cdot\alpha\cdot l_{21}}{m_4}}\ , \quad (31)$$

where ω_1 and ω_2 is a weight fluctuation freqs.

To reduce the mutual influence of fluctuations and avoid resonance it is essential toprovide the maximum frequency differentiation.

$$\max\left[\sqrt{\frac{\mathrm{c}_{11}\cdot\alpha\cdot l_{11}}{m_3} - \sqrt{\frac{\mathrm{c}_{21}\cdot\alpha\cdot l_{21}}{m_4}}}\right]. \quad (32)$$

In cause, that

$$\frac{m_3\cdot l_{11} + m_4 l_{21}}{l_{11} + l_{21}} = a\,.$$

Chosing l_{11}, l_{21} by method of finding the conditional Lagrange extremum

$$\hat{o}(l_{11}, l_{21}) = \left(\sqrt{\frac{\mathrm{c}_{11}\cdot\alpha\cdot l_{11}}{m_3} - \sqrt{\frac{\mathrm{c}_{21}\cdot\alpha\cdot l_{21}}{m_4}}}\right) + \lambda\left(\frac{m_3\cdot l_{11} + m_4 l_{21}}{l_{11} + l_{21}}\right), \quad (33)$$

where λ – Lagrange multiplier.

To find l_{11}, l_{21} write the Lagrange equation in cause, that:

$$\hat{o}'_{11} = 0\ ; \qquad \hat{o}'_{21} = 0\,;$$

$$\begin{cases} \dfrac{1}{2}\sqrt{c_{11}\cdot\alpha}\cdot\sqrt{\dfrac{1}{m_3\cdot l_{11}} + \lambda\left(\dfrac{m_3\cdot l_{11}}{l_{11} + l_{21}}\right)_{l_{11}}} = 0 \\ \dfrac{1}{2}\sqrt{\dfrac{c_{21}\cdot\alpha}{m_4\cdot l_{21}} + \lambda\left(\dfrac{m_3\cdot l_{21}}{l_{11} + l_{21}}\right)_{l_{21}}} = 0 \\ \dfrac{m_3\cdot l_{11} + m_4 l_{21}}{l_{11} + l_{21}} = a. \end{cases} \quad (34)$$

Find the original

$$\lambda\left(\frac{m_3\cdot l_{11}}{l_{11} + l_{21}}\right)'_{l_{11}} = \lambda m_3\left(\frac{(l_{11} + l_{21}) - l_{11}}{(l_{11} + l_{21})^2}\right) = \frac{\lambda\cdot m_3\cdot l_{21}}{l_{11} + l_{21}}. \quad (35)$$

In cause, that

$$\left[a = \frac{l}{2}\right],$$

$$\begin{cases} \dfrac{1}{2}\sqrt{\dfrac{c_{11}\cdot\alpha}{m_3\cdot l_{11}}} + \dfrac{\lambda\cdot m_3\cdot l_{21}}{(l_{11}+l_{21})^2} = 0 \\ \dfrac{1}{2}\sqrt{\dfrac{c_{21}\cdot\alpha}{m_4\cdot l_{21}}} + \dfrac{\lambda\cdot m_4\cdot l_{11}}{(l_{11}+l_{21})^2} = 0 \\ \dfrac{m_3\cdot l_{11} + m_4\cdot l_{21}}{l_{11}+l_{21}} = a. \end{cases} \quad (36)$$

The result of the decision system – rational coordinates for placing weights on a platform l_{11} and l_{12}.

As a result of calculation adopted by the initial data received schedules fluctuations of car hauler and cars-weights on it. (fig. 3 – 6).

Fluctuations in the rear and front axles of car hauler represent sinusoid that are exponented as damping. The maximum displacement is 0.0045 m in the first seconds of motion when hitting an obstacle.

Moving of weights at both loading platform (Fig. 5, 6) is also sinusoids that is exponented as damping. For the car, placed on top platform the maximum amplitude is 0.0035 m. The smallest displacement from equilibrium gets the car, placed on the bottom platform. It is 0.0025 m. Thus it is possible to reduce the gap to acceptable level between the car roof and upper platform to lower the center of gravity of the system.

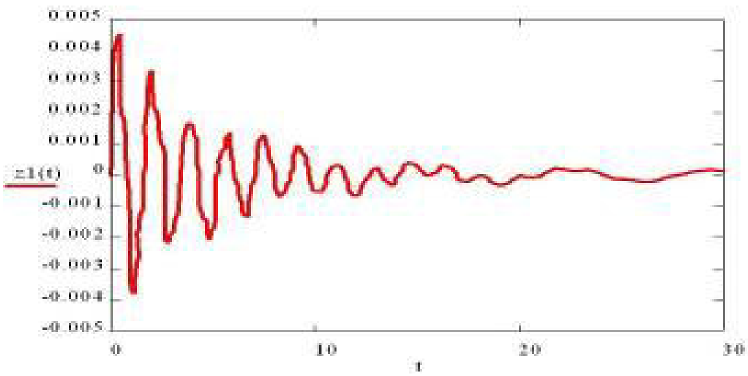

Figure 3 – Car hauler front axle shiftings

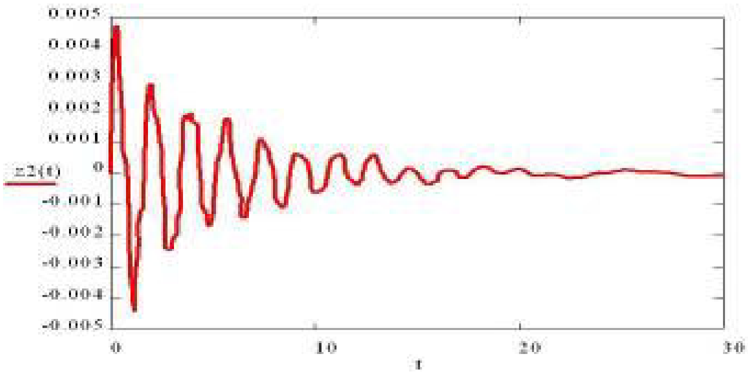

Figure 4 – Car hauler rear axle shiftings

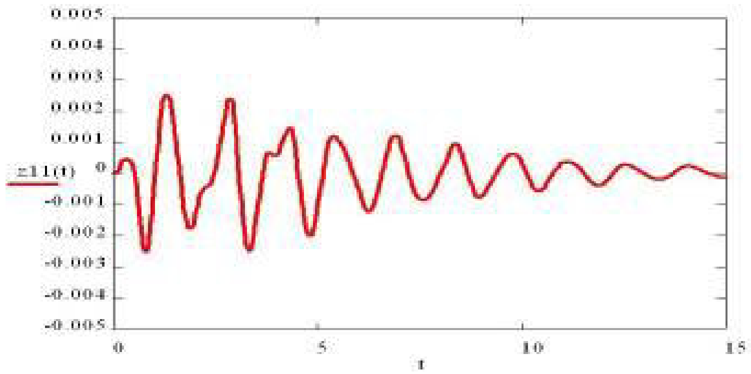

Figure 5 – Car-weight shifting on bottom platform

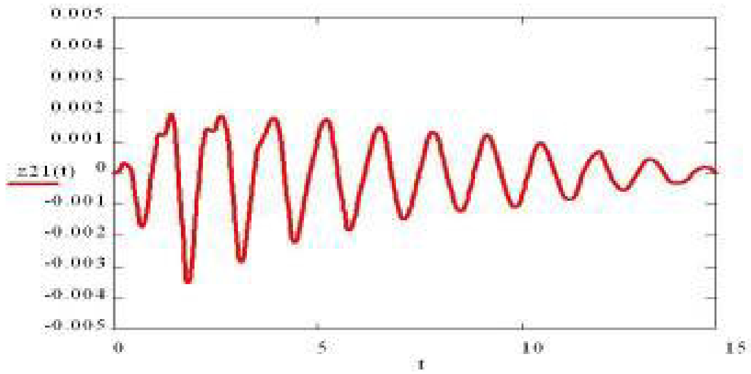

Figure 6 – Car-weight shifting on top platform

Conclusions. The simulation results indicate that considering of weight elastic properties in system «car hauler – car» leads to significant decreasing of the frequency and amplitude of the vertical fluctuations of the system. Thus, the availability of cargo can be viewed as a mean of passive dynamic vibration reduction (in case of correct choice of constructional and layout options). In this case there will be certain resonance speed of the system, which will show this effect as much as possible.

Determination of the maximum amplitude of cargo's fluctuations enables to reduce the distance between the cargo and the upper platform. In its turn lowering the platform reduces the height of gravity center system, which improves stability car hauler.

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Received 30.11.2016

UDC 656.13/73.31.41

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CAR ZAZ-1102 IMPROVEMENT IN FUEL EFFICIENCY AND ENVIRONMENTAL PERFORMANCE IN WARM-UP PHASE AFTER ENGINE COLD START

The article considers the question that relates to optimizing fuel consumption and reducing emissions of harmful substances in the exhaust gases of the vehicle in the following modes cold start of the engine with spark ignition and warm-up. To solve this problem device is proposed for increasing the temperature of the intake air at low temperatures, which will improve the mixture formation, gas exchange and better distribution of the fuel-air mixture in the engine cylinders. The use of this device is one of the promising directions of implementation of energy efficient technologies in road transport.

***Keywords:** engine with spark ignition, cold start engine, heated intake air, improving of gas exchange and mixture formation, increasing the efficiency of internal combustion engines.*

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ПОЛІПШЕННЯ ПАЛИВНОЇ ЕКОНОМІЧНОСТІ І ЕКОЛОГІЧНИХ ПОКАЗНИКІВ АВТОМОБІЛЯ ЗАЗ-1102 В РЕЖИМІ ПРОГРІВУ ПІСЛЯ ПУСКУ ХОЛОДНОГО ДВИГУНА

У статті розглянуте питання, яке пов'язане з оптимізацією витрат палива і зменшенням емісії шкідливих речовин у відпрацьованих газах автомобіля в режимах пуску холодного двигуна з іскровим запалюванням та його прогріву.Для вирішення цієї задачі запропоновано використання пристрою для підвищення температури повітря на впуску в умовах низьких температур, що дозволить поліпшити сумішоутворення, газообмін і більш якісний розподіл паливоповітряної суміші по циліндрах двигуна. Використання даного пристрою є одним з перспективних напрямів реалізації енергоефективних технологій на автомобільному транспорті.

***Ключові слова:** двигун з іскровим запалюванням, пуск холодного двигуна, підігрів повітря на впуску, поліпшення газообміну та сумішоутворення, підвищення енергоефективності ДВЗ.*

Introduction. Currently one of the priority development directions of all sectors of the domestic economy is the creation of energy efficient technologies allowing the efficient use of energy resources. This fully applies to the road transport.

Efficient vehicle operation at low ambient air temperatures is associated with different challenges. The most significant ones are the start of the cold engine and subsequent warming up. These modes are preparatory before operating the engine under load and are some of the most unfavourable engine operating conditions in fuel economy and environmental safety.

According to various sources [1 – 4], emissions of carbon monoxide (CO) and hydrocarbons (HC) with exhaust gases in the operation of vehicles with spark ignition engines in low temperatures increase 6 – 10 times, and in the modes of start-up and warm-up engine emits up to 70...80% of total emissions of incomplete combustion of CO and HC products.

Among the main reasons for the difficulties of cold engine starting in low ambient temperature are:

– the deterioration of fuel evaporation conditions, which leads to non-optimal air-fuel mixture;

– high vacuum in the intake manifold, which leads to deterioration processes of gas exchange;

– the increase in the required starting speed of the crankshaft;

– decrease the real capacity of the battery.

At low ambient temperatures, these causes occur simultaneously, exacerbating and complicating the operation of starting and warming up of cold engine. According to [5], the territory of Ukraine it is the temperate climatic zone. In the territory of the state it is formed by temperate continental climate. Of the year (about 4 – 6 months), the cars are operated at temperatures between +5°C and below.

In this regard, decisions of the tasks related to the operation of vehicles in conditions of low ambient temperatures, is relevant.

Analysis of recent researches and publications. The influence of seasonal conditions on the fuel efficiency of cars, the concentration of harmful substances in waste gases and the reliability of cars are investigated by many authors. Seasonal conditions are the factors that change periodically throughout the year. It is primarily climatic factors.

Climatic factors in different periods of the year are determined by temperature, humidity, atmospheric pressure, rainfall, force and direction of winds, duration of snow cover etc. Low ambient air temperature has significant influence on the temperature regime of the car units, especially engine and systems, supporting its work and, through their changes on fuel consumption. The most comprehensive analysis of the impact of ambient temperature on fuel consumption and emissions of harmful substances in exhaust gases of cars is given in the works [6 – 10].

It was established in [7, 11] that the fuel consumption by lowering the ambient temperature increases by 10...30%. The increase in fuel consumption is associated with increase in the viscosity of the fuel, deterioration of his ability to atomization and evaporation and, consequently, deterioration of mixture formation and combustion efficiency of fuel-air mixture. The increase in viscosity of gasoline may result in unacceptable depletion of the fuel mixture. Viscosity primarily affects the volumetric amount of fuel passing through the jet per unit time, and therefore on its consumption. Leakage of gasoline through the nozzle when changing temperature from plus 40 to minus 40°C is reduced by 20...30%.

Depending on the time of year and temperature in the region, vehicle operation, the percentage ratio of the concentrations of pollutants from «cold start», «further combined heating» and «hot» car can be in various ratios. Climatic area «moderately warm» in the calculations were obtained by the following relationship: in the warm period 2 – 3 – 95% in the cold period 10 – 15 – 75% [12].

Thus, at low temperatures of exploitation, the deterioration of the thermal state of the motor has significant influence on the increase in fuel consumption and, consequently, increasing the emission of harmful substances in the exhaust gases. In addition, the analysis of the research showed that there is some optimal value of the intake air temperature at which fuel consumption is minimal [13, 14].

Minimum fuel consumption of the internal combustion engine is warmed up. It is observed at the inlet temperature at motor +35...+45°C, its change leads to increase in fuel consumption. For the engine ZMZ-53 the decrease in air temperature in the range from plus 48 to minus 28 according to [14], causes increase in fuel consumption from 2.5 to 16%.

One of the possible ways to improve fuel economy of spark-ignition engines and reducing the emission of harmful substances in the exhaust gas at cold start of the engine and its warming up, is the provision of heated air entering the engine and to stabilize its temperature at +35...+45°C in the future.

The selection not resolved before the general problem. To solve the task of reducing fuel consumption and emissions of harmful substances in the exhaust gases in the modes of cold start engine with spark ignition and warm-up, it is necessary to improve the mixture by increasing the temperature of the air entering the engine.

Statement of the problem. The purpose of the experimental research is determination of intake air temperature influence on fuel efficiency and emissions of harmful substances in the exhaust gas of the car engine when it is warming up in idle.

The object of experimental research is the car ZAZ-1102, which has carbureted, four-stroke, four-cylinder in-line engine MeMZ-245.

Experimental investigations were carried out in the laboratory of engine testing of National transport University.

To ensure preheating of the intake air equipment with metal-ceramic heating element was used. Tests were conducted on the gasoline A-95 at the same atmospheric conditions. Engine starting and warming was carried out at an ambient temperature of about +3...+4°C at the engine speed of 1400...1500 rpm (optimal steady frequency heating) to the temperature of the engine coolant +85°C.

During testing of the engine the concentration of harmful substances in exhaust gases was evaluated: carbon monoxide CO, carbon dioxide CO_2, hydrocarbons HC behind the analyzer of the META, and fuel consumption during warm-up the engine using the fuel flow meter ONO SOKKI DF-311.

In addition, the following controlled parameters of the engine are the following: coolant temperature $t_{coolant}$, the temperature of the air to the carburetor $t_{intake\ air}$, the engine speed n (Fig. 1).

Main material and results. According to the conducted researches the following results were achieved:

equipment metal-ceramic heating element provides heating of the air entering the carb around +60...+65°C at an ambient temperature of +3.5°C (Fig. 1).

The temperature of the coolant in the heating air inlet reached +60°C for 360 seconds after starting the engine, without heating for 450 seconds. Time to warm up a cold engine coolant temperature +60°C was reduced to 90 seconds, which is 20%, while the fuel economy was 0,062 kg (about 14%).

Stabilization of the coolant temperature at +85°C was observed when using heated air for 560 seconds, without heating – for 690 seconds after starting the engine. That warm engine (to coolant temperature +85°C) when heating air inlet, is, according to the tests for 130 seconds (about 19%) most of the time warming up the engine with the standard system supply air (Fig.1).

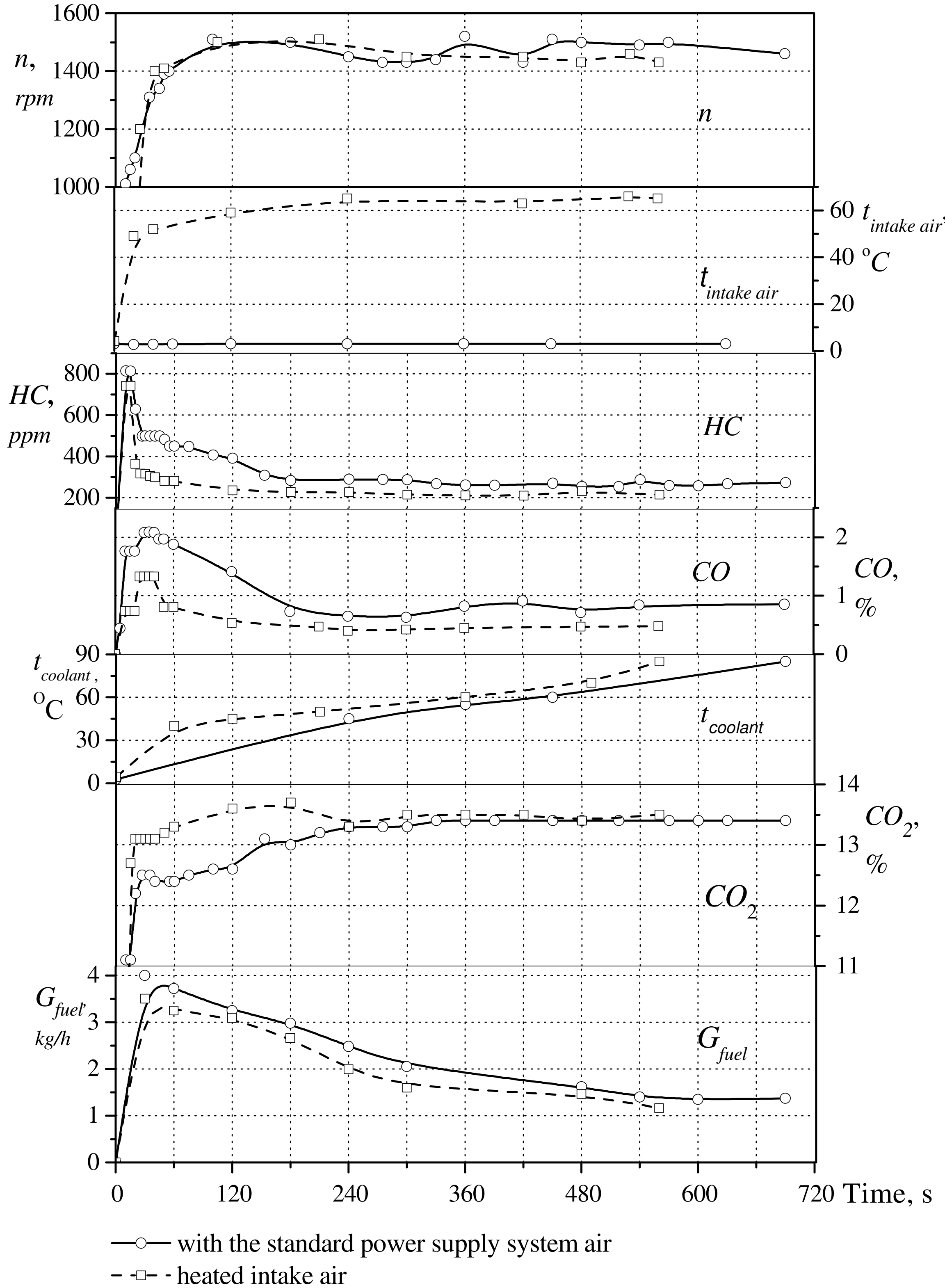

Figure 1 – The change in the performance of the engine in warm-up mode

Analysis were derived from experimental studies, dating on fuel consumption, given the time required for warming up the engine coolant temperature +85°C, to determine its quantity. Heated air inlet, fuel consumption is – 0.464 kg with the standard system – 0.582 kg. That is, during the heating of the engine, the results of tests, fuel economy, while heated air inlet is 0.118 kg (20%).

Analysis of the obtained data on engine harmful substances emissions, exhaust gases warming up in idle are showed the following (Fig. 1).

The concentration of carbon monoxide in the exhaust gas without heating the intake air is stabilized at the level of 0,73% for 180 seconds, heating the air at the level of 0,53% for 120 seconds. The concentration of carbon dioxide in the exhaust gases as heated intake air and with a staff almost are equal at the level of 13,4% after starting the engine in 240 seconds (Fig. 1).

The concentration of hydrocarbons in exhaust gases during engine warm-up heated air intake was observed significantly less stabilization occurred after 120 seconds and to the end of the warm-up amounted to about 219 ppm, whereas when working with the standard system stabilization is observed after 180 seconds until the end of the warm-up was about 271 ppm.

Concentrations of toxic substances in exhaust gases of the engine does not fully characterize the harmful effects of the engine environment, as the amount of harmful substances emitted to the atmosphere depends on the amount of combustion products that are formed in the cylinders of the engine and the time of heating.

The calculation of the mass emissions of harmful substances has shown that when using the heated intake air, their number is reduced compared to the regular system (Fig. 2).

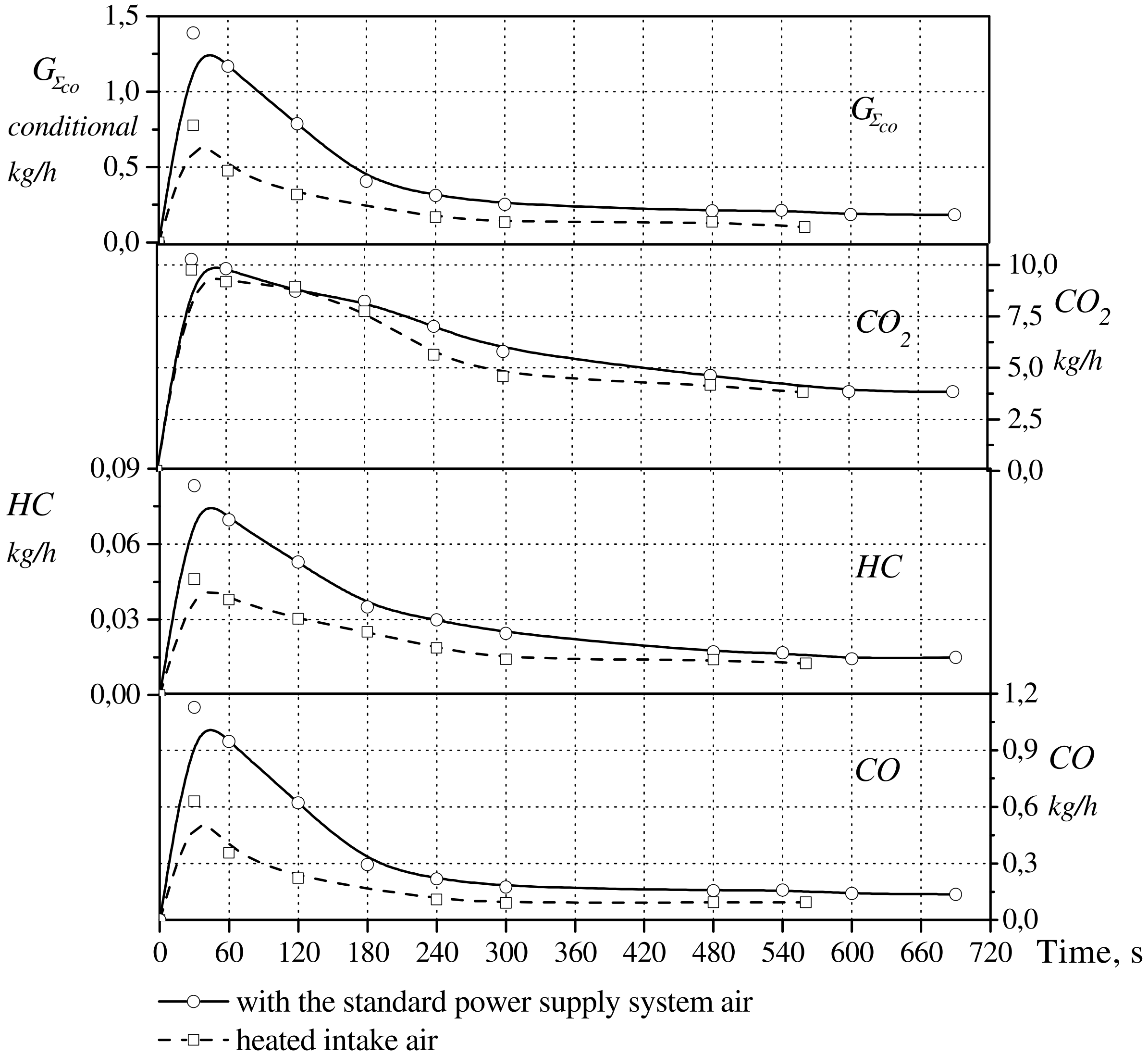

Figure 2 – The change in the mass emissions and total consolidated harmfulness of exhaust gases in the warm-up phase

During the test, the engine mass emissions in the exhaust gases averaged, taking into account the time of heating (heated air intake – 560, with the standard system 690 seconds):

– carbon monoxide with heated air – 0,034 kg/h, with the standard system – 0,076 kg/h.

– hydrocarbons, heated air – 0,0039 kg/h, with the standard system – 0,0069 kg/h.

– carbon dioxide heated air 1,047 kg/h, with the standard system – 1.27 kg/h.

The total mass emissions of harmful substances in exhaust gases averaged, taking into account the time of heating (heated air intake – 560, with the standard system 690 seconds) heated air – 0,046 conditional kg/h, with the standard system – 0,11 conditional kg/h.

Conclusions. Thus, conducted experimental studies determined the effect of air temperature at the inlet to the fuel economy and emissions of engine gasoline exhaust gases with spark ignition MeMZ-245 with its warming idling, allow the following conclusions:

1. Warm-up time to the engine coolant temperature +60°C decreased by 20%, while the fuel economy with about 14%, coolant temperature to +85°C was reduced by 19%, while the fuel economy was about 20%;

2. Mass emissions of harmful substances in exhaust gases heated air inlet in comparison with the standard system decreased on average:

– carbon monoxide by 55%;

– hydrocarbons by 43%;

– carbon dioxide by 18%;

– total mass emission are reduced to CO by 57%.

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Received 28.11.2016

UDC 621.833.65

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TENSION AND DEFORMATIONS OF STAR-SHAPED SPRING VERTICES WITH STRANGULATED ENDS OF THE FLEXIBLE COUPLING

The article examines the structure of overload flexible coupling that contains internal and external hubs, connected by a star-shaped spring with the circular vertices inserted in grooves on the outer surface of internal hub and inner surface of the external hub. Position of the star-shaped spring is fixed on the inner hub by the wedge, which allows the spring to be made either solid or of separate circular vertices with strangulated ends. Geometric synthesis of star-shaped spring with vertices of circular shape, depending on its size was conducted. It is assumed that star spring vertices are the double-hinge arches of circular form. Calculations of analytical solutions using the methods of structural mechanics were done. Analytical expressions proved workability of the star-shaped spring of flexible overload coupling during the torque transmission.

Key words: *flexible overload coupling, star-shaped spring, circular vertices, statics calculation, deformation.*

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НАПРУЖЕННЯ І ДЕФОРМАЦІЇ ВИСТУПУ ЗІРКОПОДІБНОЇ ПРУЖИНИ З ЗАЩЕМЛЕНИМИ КІНЦЯМИ ПРУЖНОЇ МУФТИ

Описана будова і принцип роботи запобіжної муфти, що містить внутрішню і зовнішню півмуфти, з'єднані між собою зіркоподібною пружиною з виступами круглої форми, встановленою в заглибини на зовнішній поверхні внутрішньої та внутрішній поверхні зовнішньої півмуфти. Зіркоподібна пружина може бути виконана суцільною або складена з окремих виступів з защемленими кінцями. Проведено геометричний синтез зірковоподібної пружини з виступами кругової форми, в залежності від необхідних розмірів. Прийнято, що виступ зіркоподібної пружини являє собою арку з защемленими кінцями і для неї, методами будівельної механіки, проведені розрахунки. Отримані аналітичні вирази дозволяють робити висновки про роботоздатність зірковоподібної пружини пружної муфти при передачі нею обертального моменту.

Ключові слова: *муфта запобіжна пружна, зіркоподібна пружина, виступ кругової форми, статичний розрахунок, деформація.*

Introduction. Drives of hoisting, transport, building, road, land reclamation machines and others often include different couplings, which are quite responsible mechanical devices that can determine the reliability and durability of all equipment. The main purposes of couplings are the shafts connection and torque transmission. Besides, elastic safety couplings perform such responsible functions as compensation of the harmful effects of shaft ends geometric axes offset, resulting from inaccurate production, installation or design features and operating conditions of drives; amortization of vibrations, jolts and shocks arising during operation; protection of machine elements from overload; facilitation of the machine start-up. A variety of couplings maintenance functions contributed to the development of a large number of constructions. But in all cases the work of couplings has still many shortcomings that need to be eliminated due to their improvement.

Review of recent sources of research and publications. The modern technical literature, for example [1-4] describes the safety cam couplings with shear pin or others that through the direct contact of their half-sleeves transmit torque toughly, and it negatively affects the work of drives elements and machine in general. The analysis of the shortcomings of known coupling designs enabled to develop the new design of safety elastic couplings with star-shaped springs at the level of patents [5-8].

Specifing problems unsolved earlier. Theoretical and experimental research for new safety elastic couplings has not yet been held. Part of such research has been made for couplings with a parabolic [9] and a circular vertices [10] and with hinged fastening of ends.

Problem statement. To conduct a preliminary analysis of power parameters of safety elastic couplings including star-shaped springs with strangulated ends of vertices.

Main material and results. This article deals with static calculations of star-shaped springs of safety elastic couplings [5-9] in operation, i.e. during the transfer of sustainable torque. One of these couplings is shown on Fig. 1.

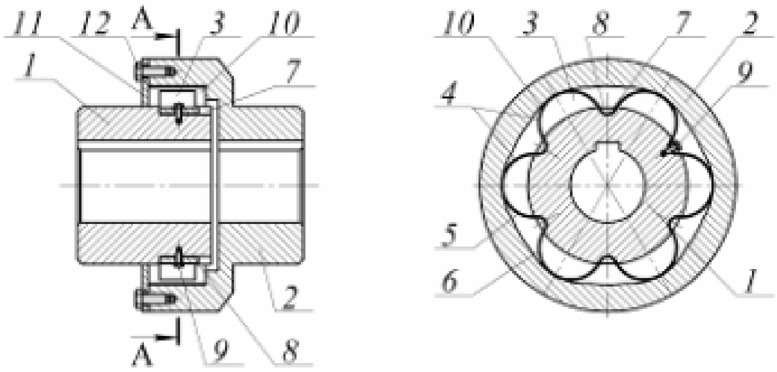

Figure 1 – Scheme of safety elastic couplings with star-shaped spring with strangulated ends

Safety coupling with the star-shaped spring has internal 1 and external 2 half-sleeves, connected by a star-shaped spring 3 with vertices 4. Side surfaces 5 and 6 of the vertices 4 made convex apart from the axis of symmetry, and their internal ends set in recesses 7 on internal half-sleeve 1 and fixed by wedges 8 with some tension and screws 9. The tops of the vertices 4 are located in the recesses 10 of external half-sleeve 2, moreover, those recesses 10 are made with radius greater than the tops rounding radius.

Safety coupling with the star-shaped spring assembles in the following order. First, internal ends of vertices 4 set in recesses 7 on internal half-sleeve 1 and fixed with wedges 8 and screws 9. Next, assembled internal half-sleeve 1 with star-shaped spring 3 set in the external half-sleeve 2 so that the vertices 4 are in contact with the recesses 10. Safety coupling with the star-shaped spring with strangulated ends of vertices is ready.

Safety coupling with composed star-shaped spring works as follows. When rotating internal sleeve 1 transmits torque through star-shaped spring 3 to the external sleeve 2, the overload mode star-shaped spring 3 deforms, decreasing in its diameter along its outer contour. The surfaces 5 and 6 of vertices 4 bent in direction of its convexity, providing deformation of star-shaped spring 3 within the limits of its elastic deformation until they are out of the recesses 10 of external sleeve 2. Rounding radius of recesses 10 are larger than radius of peaks 4 so vertices 4 slide over the cylindrical surface of the outer external 2 until torque is reduced to the nominal value. Star-shaped spring 3 can be solid or composed of separate vertices.

Fig. 2 shows a model of elastic couplings with star-shaped spring, created in the "KOMPAS - 3D", assembled (Fig. 2, a) and disassembled (Fig. 2, b).

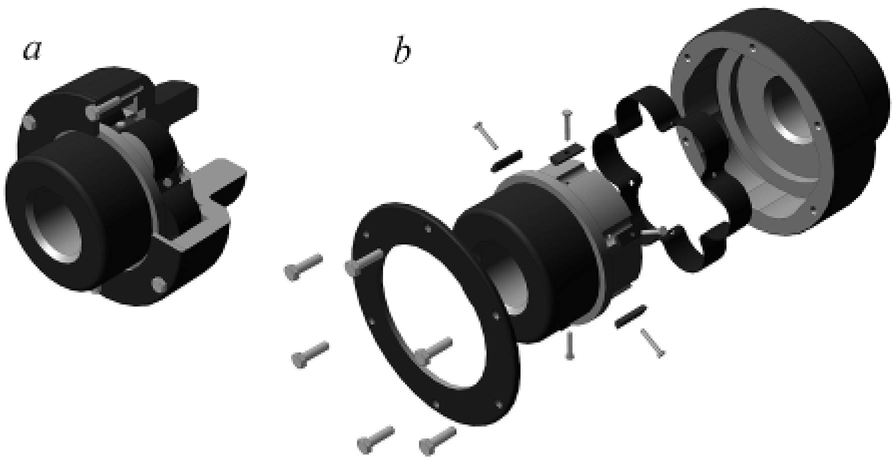

Figure 2 – Models of elastic safety coupling: *a* – assembled; *b* – disassembled

Fig. 3 shows the scheme of the dimensions for the geometric synthesis of star-shaped springs, where: D - inner diameter of external sleeve; D_1 - outer diameter of the internal sleeve; R - radius circular performance; l - pitch (span) of circular vertices; h - the height of segment; r - radius of recesses in the internal sleeve; R_1 - radius of recesses in the external sleeve; δ - the height of recesses in the external sleeve; z - number of vertices.

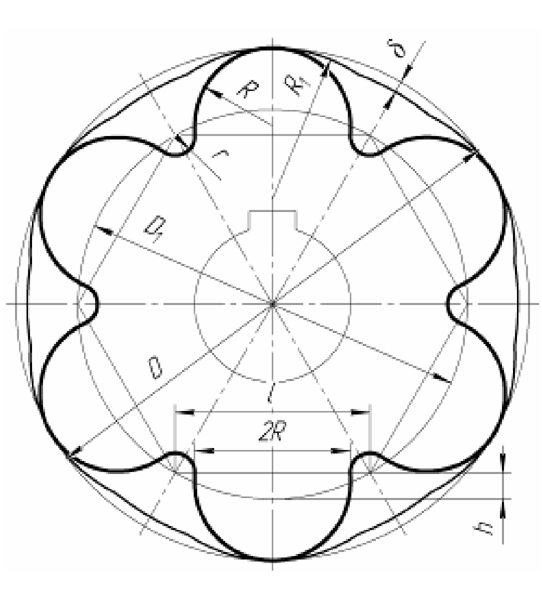

Figure 3 – Scheme for geometric synthesis of star-shaped springs with circular vertices

The relationship between these dimensions can be described by

$$l = D_1 \sin\frac{180°}{z};\tag{1}$$

$$h = R - \sqrt{R^2 - \frac{l^2}{4}};\tag{2}$$

$$r = \frac{l - 2R}{2}; \quad R_1 = (1{,}8 - 2{,}2)R;\tag{3}$$

$$D = D_1 + R - h - \delta.\tag{4}$$

The length of the of the workpiece for manufacturing of the solid star-shaped springs is

$$L_{заг} = \pi(R + r)z.\tag{5}$$

When designing couplings it is recommended to take following parameters construction:

$D_1 \le 1{,}75\ d$,

where d – shaft diameter;

$R = (0{,}4\ldots0{,}45)\ l$;

$z = 6$.

Set problem is solved with the following assumptions: vertices of star-shaped spring deform the same along its axis of symmetry; acting load in vertices lies on this line of symmetry in the plane of star-shaped spring that is perpendicular to rotation axis of coupling, and equals

$$F = \frac{2T_p}{Dzf},\tag{6}$$

where T_p – estimated torque, transmitted by coupling;

D – outer diameter of star-shaped spring;

z – number of star-shaped spring vertices;

f – coefficient of friction.

The vertices deform similarly. All calculations are reduced to calculation of circular arch with strangulated ends using the method of Mohr [11].

Scheme for calculations of star-shaped spring vertex is shown in Fig. 4.

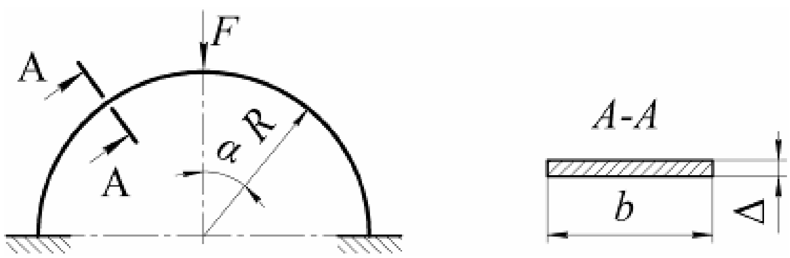

Figure 4 – Scheme for calculations of star-shaped spring vertex

According to [11] this arch with strangulated ends (Fig. 3) is three times statically undefined system. The most suitable equivalent and main systems are shown in Fig. 5.

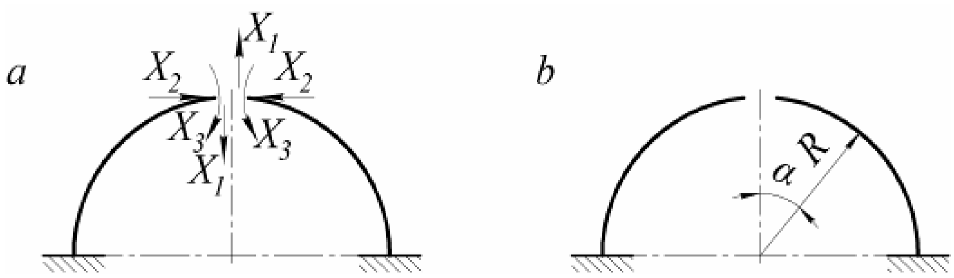

Figure 5 - Equivalent (*a*) and main (*b*) systems of star-shaped spring vertex

Superfluous bonds are taken to unidentify X_1. X_2, X_3. Arch is deformed identically to star-shaped spring vertex. Unidentified forces in the equivalent system are determined from zero equality condition for displacements in directions x_1. x_2, x_3.

For this th following the canonical equation of the forces method is done:

$$\delta_{11}X_1 + \delta_{12}X_2 + \delta_{13}X_3 + \delta_{1F} = 0;$$

$$\delta_{21}X_1 + \delta_{22}X_2 + \delta_{23}X_3 + \delta_{2F} = 0; \tag{7}$$

$$\delta_{31}X_1 + \delta_{32}X_2 + \delta_{33}X_3 + \delta_{3F} = 0,$$

where δ_{11}, δ_{22}, δ_{33} – displacement in the directions x_1. x_2, x_3, caused by forces $X_1 = 1$. $X_2 = 1$, $X_3 = 1$, respectively;

δ_{12}, δ_{13}– displacement in the direction of the X_1, caused by $X_2 = 1$ and $X_3 = 1$, respectively;

δ_{21}, δ_{23} – displacement in the direction of the X_2, caused by $X_1 = 1$ and $X_3 = 1$, respectively;

δ_{31}, δ_{32} – displacement in the direction of the X_3, caused by $X_1 = 1$ and $X_2 = 1$, respectively;

δ_{1F}, δ_{2F}, δ_{3F} – displacement in the direction of X_1. X_2, X_3, respectively, caused by the external load F.

Main system can be derived from the equivalent system after its liberation from external load F and unidentified forces X_1. X_2, X_3, that replace redundant links. Main system is shown in Fig. 4, *b*, where R is the circle radius.

All this displacement are defined using the Mohr integrals

$$\begin{aligned}
&\delta_{11} = \sum\int_0^S \frac{M_1^2 ds}{EJ}; \quad &&\delta_{22} = \sum\int_0^S \frac{M_2^2 ds}{EJ}; \quad &&\delta_{33} = \sum\int_0^S \frac{M_3^2 ds}{EJ};\\
&\delta_{12} = \sum\int_0^S \frac{M_1 M_2 ds}{EJ}; &&\delta_{13} = \sum\int_0^S \frac{M_1 M_3 ds}{EJ}; &&\delta_{21} = \sum\int_0^S \frac{M_2 M_1 ds}{EJ};\\
&\delta_{23} = \sum\int_0^S \frac{M_2 M_3 ds}{EJ}; &&\delta_{31} = \sum\int_0^S \frac{M_3 M_1 ds}{EJ}; &&\delta_{32} = \sum\int_0^S \frac{M_3 M_2 ds}{EJ};\\
&\delta_{1F} = \sum\int_0^S \frac{M_1 M_F ds}{EJ}; &&\delta_{2F} = \sum\int_0^S \frac{M_2 M_F ds}{EJ}; &&\delta_{3F} = \sum\int_0^S \frac{M_3 M_F ds}{EJ},
\end{aligned} \tag{8}$$

where E – elasticity modulus of the first kind for spring material;

J – axial inertia moment of the section (see. Fig. 4), equals $J = b\Delta^3/12$;

M_1, M_2, M_3 and M_F – bending moments from the forces X_1. X_2, X_3 and F, respectively: $M_1 = \pm X_1 R \sin\alpha$; $M_2 = \pm X_2 R (1 - \cos\alpha)$; $M_2 = X_3$; $M_F = F_1 R \sin\alpha$, where $F_1 = F / 2$.

Fig. 6 shows the constructed diagrams of bending moments

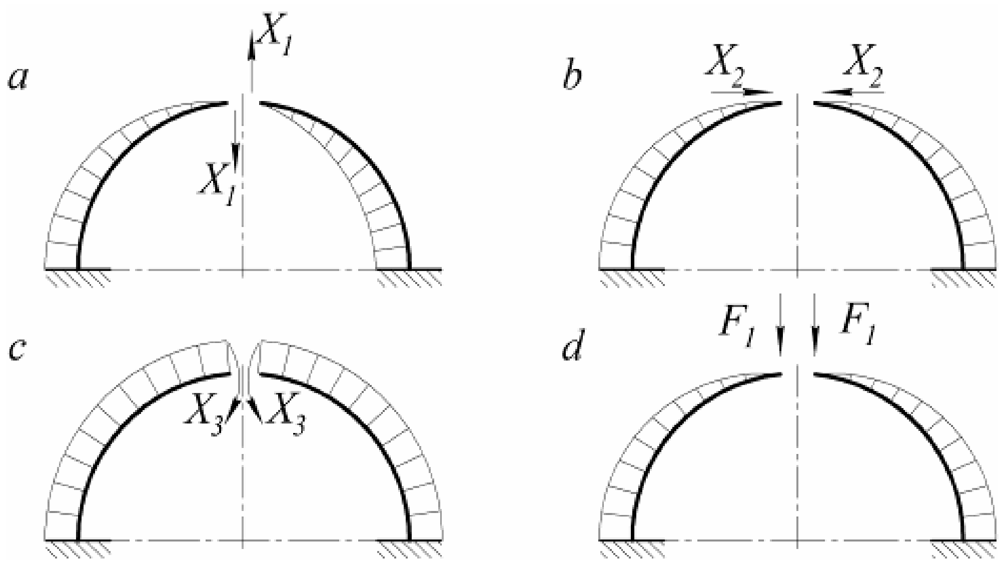

Figure 6 – Diagrams of bending moments from forces X_1. X_2, X_3 and F

From the analysis of integrand values of moments from expressions (7) and diagrams (Fig. 6) we have that $\delta_{11} = 0$, $\delta_{12} = \delta_{21} = 0$, $\delta_{13} = \delta_{31} = 0$ and $\delta_{1F} = 0$. Thus, the system of canonical equations (7) is reduced to

$$\begin{aligned} &\delta_{22}X_2 + \delta_{23}X_3 + \delta_{2F} = 0 \\ &\delta_{32}X_2 + \delta_{33}X_3 + \delta_{3F} = 0 \end{aligned}. \tag{9}$$

Using expressions (3) and bending moment diagrams (see Fig. 6), assuming that limit integration for curved sections is $s = D\,d\,\alpha$, and for angle α that varies from 0 to $\pi / 2$, it is got:

$$\begin{aligned} &\delta_{22} = \frac{R^3}{EJ}\int_0^{\pi/2}(1-\cos\alpha)^2 d\alpha = \frac{(3\pi-8)R^3}{4EJ}; \\ &\delta_{23} = \delta_{32} = \frac{R^2}{EJ}\int_0^{\pi/2}(1-\cos\alpha)d\alpha = \frac{(\pi-2)R^2}{2EJ}; \\ &\delta_{33} = \frac{R}{EJ}\int_0^{\pi/2}d\alpha = \frac{\pi R}{2EJ}; \\ &\delta_{2F} = \frac{FR^3}{EJ}\int_0^{\pi/2}(1-\cos\alpha)\sin\alpha d\alpha = \frac{FR^3}{2EJ}; \\ &\delta_{3F} = \frac{FR^2}{EJ}\int_0^{\pi/2}\sin\alpha d\alpha = \frac{FR^2}{EJ}. \end{aligned} \tag{10}$$

To get a system of equations (4) relatively to unidentified X_2 and X_3 it is used:

$$X_2 = \frac{\delta_{23}(\delta_{22}\delta_{3F} - \delta_{32}\delta_{2F})}{\delta_{22}(\delta_{22}\delta_{33} - \delta_{23}^2)} + \frac{\delta_{2F}}{\delta_{22}}; \qquad X_3 = \frac{\delta_{32}\delta_{2F} - \delta_{22}\delta_{3F}}{\delta_{22}\delta_{33} - \delta_{23}^2}. \tag{11}$$

The final expression for determination of resultant bending moment in the vertex of circular shape with the strangulated ends it is

$$M_\Sigma = M_2 X_2 + M_3 X_3 + M_{2F} + M_{3F}. \tag{12}$$

Fig. 7 shows the diagram of resultant bending moment in circular vertex with strangulated ends

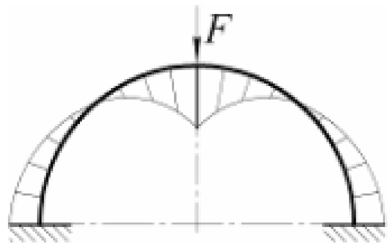

Figure 7 – The resultant diagram of bending moment in circular vertex with strangulated ends

When $\alpha = 0$, it is the maximum bending moment on the axis of symmetry of circular vertex.

To determine the displacement δ it is used Mohr method and the basic system (see. Fig. 5, *b*). In the direction of displacement δ it is exerted power unit ($X = 1$) and from it, bending moment diagram $M_x = R / 2$ is done,

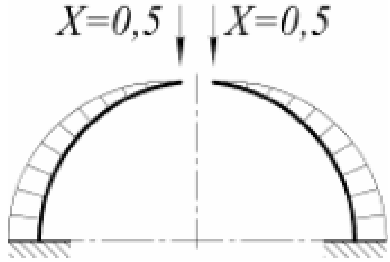

Figure 8 – Diagram of bending moment M_4 from the force X

Then use the moments' value M_Σ and M_4, and obtain:

$$\delta = \frac{R}{2EJ}(M_2 X_2 + M_3 X_3 + M_{2F} + M_{3F}). \quad (13)$$

The nature of the deformation of of vertex with strangulated ends of star-shaped spring is shown in Fig. 9.

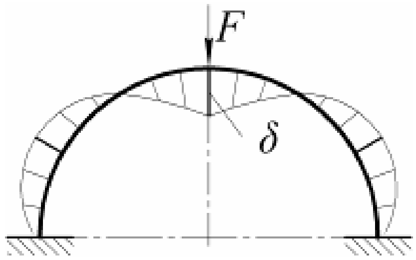

Figure 9 – Diagram of deformation of vertex with strangulated ends of star-shaped spring

Conclusions. Analytical dependence (13) between deformation and the load in star-shaped spring with the circular vertices with strangulated ends may be used in designing of new safety elastic couplings.

Expression (12) allows to determine the maximum value of bending moment for dangerous intersection and to find a tension for it by formulas.

The proposed method of theoretical research of dependence between the load and deformation in star-shaped springs with circular vertices with strangulated ends can be used for springs with different number of vertices and is the basis for further safety elastic couplings studies.

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Received 01.12.2016

UDC 631.052: 62-192

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ANALYSIS OF WORK CONDITIONS AND CAUSES WEAR HOLES SEPARATING SIEVES IN SERVICE

Advantages and disadvantages of sieves for the crusher were considered. The research focuses on changes of the holes of the sieves toroidal shape and establishes common time to failure. The result of experimental research is to establish the durability of the separating sieves with holes design close to the wear in comparison with standard sieves. The result shows the efficiency of using sieves with toroidal form of holes similar to the natural wearing, which persists for the entire lifetime and provides high-quality completion of the separation process. The principal difference in the experimental separation of the sieves is that they are in the process of operation and wear effectively perform the function of sifting the grain mass desired fraction. The new form of holes, close to form normal wearing has led to the possibility of such profiles forming that change their shape.

***Key words:** sieve, exploitation, grinding, durability, shape normal wear.*

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АНАЛІЗ УМОВ РОБОТИ ТА ПРИЧИНИ ЗНОШУВАННЯ ОТВОРІВ СЕПАРУЮЧИХ РЕШІТ В ПРОЦЕСІ ЕКСПЛУАТАЦІЇ

Розглянуто переваги та недоліки решітних класифікаторів при утриманні дробарки в експлуатації. Дослідження спрямовані на вивчення зміни отворів тороїдальної форми решіт та встановлення загального наробітку до відмови. Результатом проведених експериментальних досліджень є встановлення довговічності сепаруючих решіт із конструкцією отворів наближеної до форми природного зношування в порівнянні із серійними решетами. Принциповою різницею в роботі експериментальних сепаруючих решіт є те, що вони в процесі експлуатації і зношування ефективно виконують функцію з просіювання зернової маси потрібної фракції. Застосування нової форми отворів наближеної до форми природного зношування призвело до можливості формування таких профілів, що найменше змінюють свою форму.

***Ключові слова:** решето, експлуатація, подрібнення, довговічність, форма природного зношування.*

Introduction. In the process of shredding, grain material moves in the inner circular perimeter flow chamber. At steady state intensity of the crusher particles passing through the sieve is almost constant and corresponds to a certain number of them that go from the camera to remove.

Fractional composition of grain and milled grain is determined by the size of the holes. Changing these holes during operation is result in the loss of this indicator. At the same time, the quality of grinding grain mix is the main factor that influences consumer properties of feed, and ultimately weight gain in animals.

In the process of deterioration in the operation of crushers, sieves openings separating lose their original geometrical parameters. This profile of the peripheral part of the returned inside the camera takes the shape of January curve that gradually develops may lead to full wipe membranes between holes.

Analysis of basic research. The main disadvantage of mills is that the grinding products during an intensive wear of hammers edges holes in separate sieves. Much work was conducted to improve the life of the hammers. At the same time less attention was paid to disability of sieves as the main separating working parts.

Analysis of literature and patent sources indicates that the prospects for the development of technical means to perform the process of grinding and separation of raw materials in the manufacture of animal feed in the direction of number and quality increasing can be achieved as a result of the working mills improvement [1 - 4].

According to the operation it needs to replace parts and components of separating working bodies after operating time1000-1800 tons. The lowest operating time among them are cylindrical sieve with holes (1070 tons).

A crusher for crushing grain mass often use smooth sieve with holes 3, 4, 6, 8 and 10 mm, made of sheet steel 1 ... 3 mm. Checked uneven wear sieves, which are much faster lose original shape holes in the bottom of the camera.

There is some work to improve sieves, which are aimed at improving productivity crushers. For example, the idea of the possibility of changing the area by adjusting the passage opening sieve displacement presented in [5]. This design consists of two adjacent sieves that are able to move relative to each other. Due to this, is the quality and productivity of grinding mills.

It is proposed to establish a place sieve deck with the working surface which has a corrugated shape, and rounded holes [6] .In this case, energy consumption in the process of crushing is reduced, that is economically justified.

Noteworthy research results [7] increase productivity crusher, crushing process, reduce energy and improve quality of the product through the use of real and personal sieves, rectangular holes. Movable sieve has the ability to guide displaced along the crushing chamber to immovable sieve over distance not exceeding a length of perforated holes.

Setting objectives. The aim of work is to analyze the main types of damage crushers, to find possible ways to eliminate these problems, to continue work of sieves and their reliability in general. Another objective of the study is changes dynamics in the shape of holes in the sieves separating shorts in serial is to develop method removing fingerprints at working surface [8]. Removing the replicas of the studied sites performed at regular operating time (ΔQ = 100 t), followed by a photo obtained profiles.

Profiles, formed in the process of operation determine the dynamics of changes in the shape of the holes of the sieve, which gives the opportunity to build according to changes of the geometric parameters of permissible and ultimate forms. Changing the geometry of the holes indicates the formation of a particular surface in a separation process and can be measured: wear on the thickness and area of worn portions in cross-section.

Quantitative evaluation linear wear lengths of the holes (Δh) depends on the amount of sifted material passing through the holes and it is functionally described by the formula:

$$\Delta h = f\left(Q\right).$$

The specific functional depends on the profile holes of the sieve set according to the results of experimental studies using the mesh wear.

Change of surface shape of the holes in sieve during the process of operation is submitted by relevant curves C, D, E, F, G, H . Distance between the created profiles is layer of material, worn over a period of hours (ΔQ). The thickness of this layer can be determined using the held normal to one of formed surfaces of wear equal to the amount of linear wear in the corresponding points of the openings in each period (ΔQ). Thus, the system of the profile curves and the normal made them form grid of wear. This grid describes the general nature of the shape changes of holes and the distribution of magnitude of wear on friction surfaces in full.

Main material and results. Practical operation of crushers proved that the working part of the sieve is subjected to rapid wear and loss of original form holes, thereby forming a geometric shape that is significantly different from the inherent design. Curved surface formed due to wear form increases the perimeter interacting with the grain weight and ultimately leads to deterioration in the quality of the output product.

The experience of working parts separation indicates their low durability. This leads to the need for further research related to ensuring healthy state, for their longer life.

The physical nature of separating sieves wear practically, as consequence of the complexity of the structure of their working surfaces, has not been studied. The main hypothesis of the wear surfaces are caused by influence of grain mixture flow and small amount of abrasive particles having more hardness than hardness of the separators material, the number of which in feed material is up to 0,5%, and the dust 0,26% of the total weight.

The presence of solid components, causing partial deformation and scratches on the surface during contact with the sieve is destroying gradually. Therefore, the wear of the sieves working surfaces is regarded as natural process of loss form under contact interaction of working body of the material, which is crushed.

The quality of grinding depends on the physic-mechanical properties of the grain material, which determine its destructive characteristics. Thus, the moisture content primarily affects its strength in the future, the increased grinding efficiency. With increasing moisture content of the grain material increases its ductility, which leads to increase intensity of the hammers blows and sticking it to the working parts. When grinding excessively wet grain consumes and deteriorates the quality of the output product. Also, this type of grinding grain increases the intensity of wear, as well as the impact of aggressive environment.

Characteristics separating sieves as perforated details and characteristics of wear holes are given, promising to improve the resource, to increase longevity using all constructive methods. This is due to operating conditions and manufacturing sieves, where the use of hardening coatings or wear resistant materials is not technically feasible and economically justified.

One of approaches is constructive method which provides making sieves with form that is resistant to wear holes (Ukraine patent for utility model number 96341). Form holes are made on the concave surface, close to the toroidal obtained as a result of natural wear and tear.

Characteristic feature of job profiles sieves with holes toroidal shape is that they are fundamentally different from those projections holes for serial sieves [9]. However, the rate of formation of profile curves on the side (A) is intensively opposite side (B).

General view of curves core hole family with image net of depreciation is shown in Figure 1.

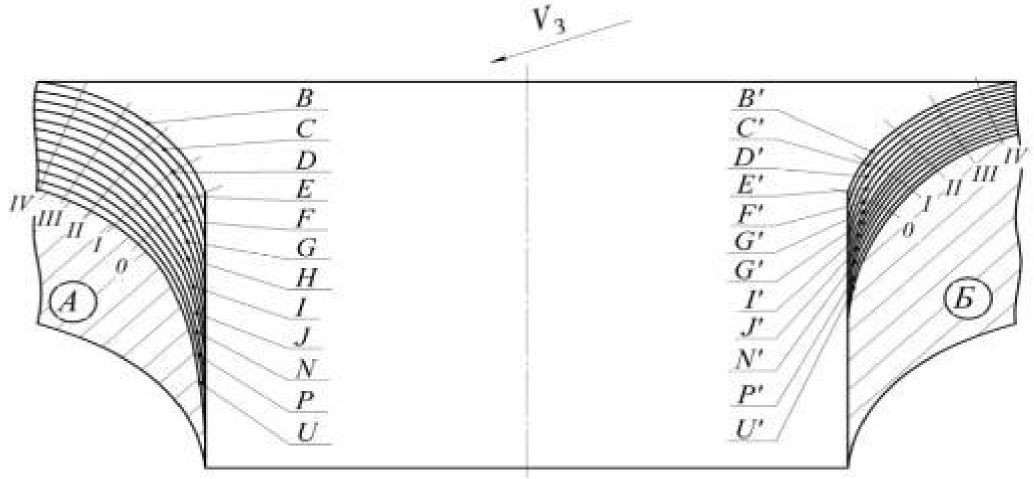
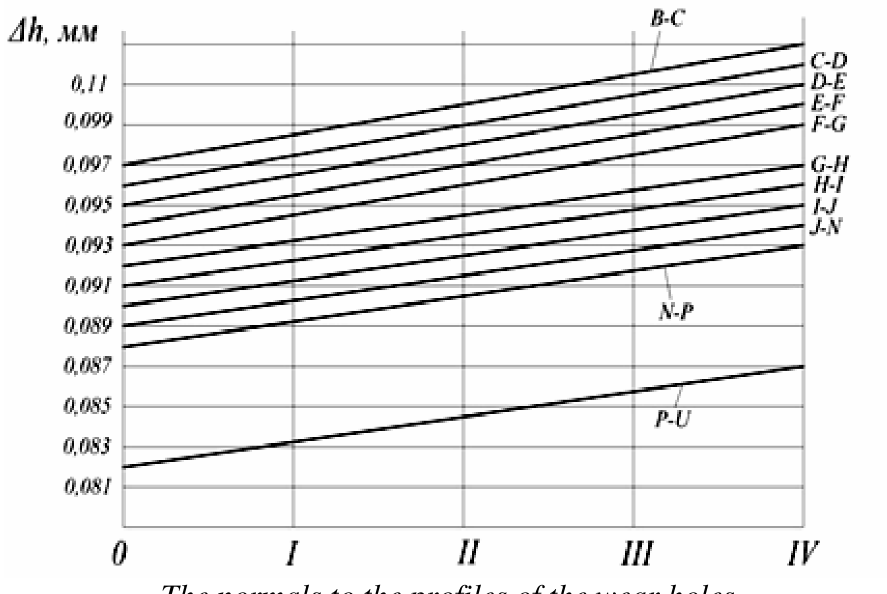


Figure 1 – Net depreciation experimental separating sieves after operating time:
B – new; *C* – 100 t; *D* – 200 t; *E* – 300 t; *F* – 400 t; *G* – 500 t; *H* – 600 t;
I – 700 t; *J* – 800 t; *N* – 900 t; *P* – 1000 t; *U* – 1080 t (wear limit);
0, I, II, III – line depreciation for the entire period of operating time;
V_s – direction of the grain material.

The family received the curves, indicated essential identical nature of the profiles wear. At the same time there is some uneven distribution of values of linear wear normal position. Thus, for the normal "0" is smaller than normal for higher numbers. This trend continues as the inclusion of new areas of wear area of the cylindrical surface of the hole. Detailed analysis of the distribution of linear wear side openings, by determining the thickness of the material lost (Δh) operating time in each period is shown in Figure 2 and Figure 3.

The group of curves obtained to indicate the general nature of the formation of stable structures. At all stages of deterioration pilot sieve after working hours, meaning the thickness of the material decreases lost and moving deep into the profile material sieve. As the wear profiles, the value (Δh) retains their general character even incremental movement (Figure 2 and Figure 3) in the direction of reduction.



The normals to the profiles of the wear holes

Figure 2 – Dependence of thickness loss of material sieve pilot holes on the side (A)

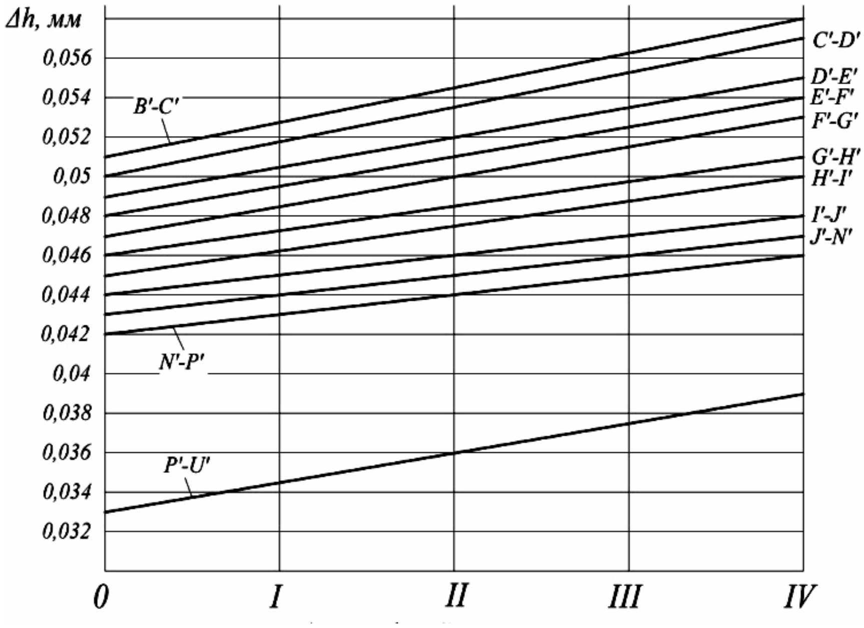


Figure 3 – Ependence of thickness loss of material sieve pilot holes on the side (B)

Reducing the thickness of the material loss along the normal «0» is associated with redistribution pressure fine particles of grain while passing through the separating holes. Normal higher order increases in the thickness loss in each period ΔQ.

From these dependences it is shown that the thickness of the wear in every period of operating time practically is unchanged. This result confirms that the use of separating sieves with holes toroidal shape is effective to increase their durability. Operating hours pilot sieve to wear full is about 1000 tons of crushed grain sieving.

The result shows the efficiency of sieves with holes toroidal shape close to the shape of the natural wear and tear, which is stored in furthering lifetime and provides high-quality separation process of passing. The new design reduces the loss of holes form surface, increasing the life of the sieves and crushers in general.

Studies of wear holes in sieves experimental section shows the uneven shape by losing parties, observed with serial wear sieves. Therefore, to improve the durability and working full potential resource it is necessary to sieve turning it 180 ° (reverse). Performing this operation is advisable for operating time after the crusher 750 tons. Important characteristic that reflects the specific features of forming surfaces during wear, is to determine areas of worn areas that are between two formed profiles wear in every period of operating time ΔQ results of these studies show that the distribution of values in each period ΔQ is the amount equal, and the party (*A*) is 0,084 mm^2, and on the side (*B*) ~ 0,041 mm^2 of the total area.

Because turning sieve (reverse) after an operating time of 750 tones or less worn side (*B*) takes position on the side (*A*), continuing to deteriorate both parameters of the opposite side (Fig. 4, $Б_{пер}$) and vice versa (Fig. 4, $A_{пер}$). As a result, the area of demolition party gradually aligned closer to each other. When the wear limit sieves parties (*A* and *B*) holes lose the same amount of material (Fig. 4, and 5), it exhauses its resources.

Based on the results of experimental studies conducted in actual use it was found that longevity sieves with holes toroidal shape, close to the shape of the natural wear and tear are 1470 tons of recycled materials. Thus, with respect to durability experimental batch sieve with a modified form of openings it is increased 1, 75.

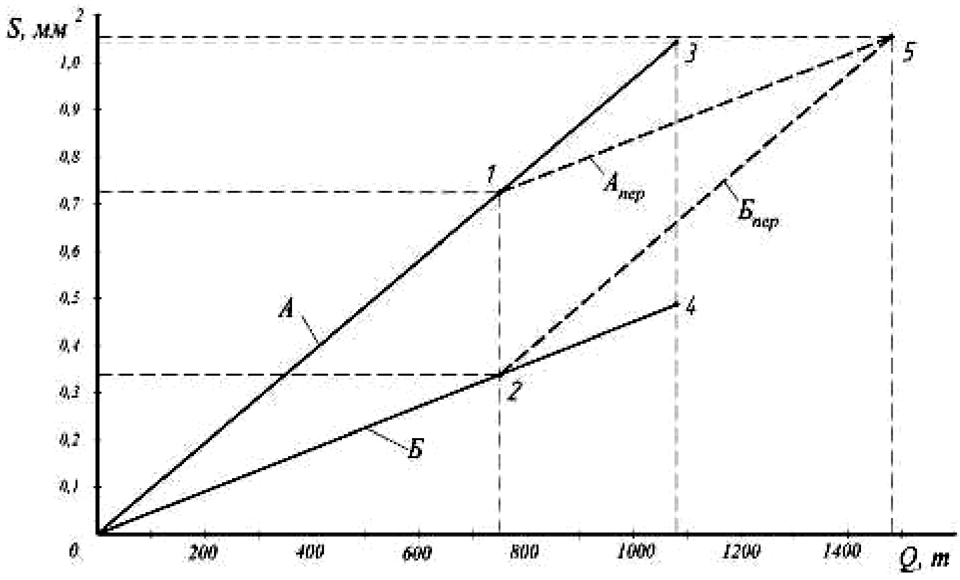


Figure 4 – Dependence of changes in the area of the material lost in the operation of experimental sieves before and after the revolution of 180 ° (reverse):
1-2 – the best party wear holes for turning operation;
3-4 – the maximum wear holes without turning sieve;
5 – maximum values holes extended wear life

Wear sieves with holes toroidal shape leads to a special profile, which increases their efficiency. Thus, the grinding module is always within Me = 1, 58-1, 62 mm. Number of original product that meets the requirements of zootechnical feeding is increased by 5% compared with serial samples. This indicates better quality experimental screening sieve, and it is characterized by round shape of the holes close to the natural shape wear.

Conclusion. It is advisable to increase durability sieves lay in the original structural form apertures that ensures minimal wear of their intensity. Toroidal shape is close to natural shape wear.

The principal difference in the experimental separating sieves is wearing during the operation and function of effectively screening the desired grain mass fraction. The use of new form of holes is close to the natural shape wear. It leads to the possibility such profiles forming and at least changes their shape. The service life of sieves showed the effectiveness of their use in comparison with the serial.

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Received 16.02.2017

UDC 519.63

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APPLICATION OF NEW ALTERNATIVE MODELS OF FINITE ELEMENT METHOD IN PROBLEMS OF ROD TORSION

The work is dedicated to the problem solution of prismatic bars torsion with a rectangular section by the finite element method with the use of standard and alternative serendipian models. By the solving of the inverse problem considering the precise value for maximum shear stress, the new improved models on the biquadratic and bicubic serendipian elements were received. By using new alternative models as well as standard models, the shear stresses in two dangerous points of section and the torque for different ratio of the rectangle's sides were defined. The obtained results allow to solve various application problems of physical fields restoring occuedr in technical systems and objects by developing new mathematical models.

Keywords: *prismatic rod, stress function, torque, shear stresses, finite element, basic function.*

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ЗАСТОСУВАННЯ НОВИХ АЛЬТЕРНАТИВНИХ МОДЕЛЕЙ МЕТОДУ СКІНЧЕННИХ ЕЛЕМЕНТІВ У ЗАДАЧАХ ПРО КРУЧЕННЯ СТЕРЖНІВ

Роботу присвячено розв'язуванню задачі про кручення призматичних стержнів з прямокутним поперечним перерізом методом скінченних елементів з використанням стандартних та альтернативних серендипових моделей. Шляхом розв'язання оберненої задачі з урахуванням точного значення для максимального дотичного напруження було отримано нові вдосконалені моделі на біквадратичному та бікубічному серендипових елементах. З використанням нових альтернативних моделей, а також і стандартних моделей, було визначено дотичні напруження у двох небезпечних точках перерізу та крутний момент для різних відношень сторін прямокутника. Отримані результати дають можливість вирішувати різні прикладні проблеми, пов'язані з відновленням фізичних полів, що виникають в технічних системах та об'єктах, шляхом розробки нових математичних моделей.

Ключові слова: *призматичний стержень, функція напружень, крутний момент, дотичні напруження, скінчений елемент, базисна функція.*

Problem definition. Currently, approximate methods increasingly used in applied problems since the precise solution of differential equations with partial derivatives are quite bulky and it is very difficult or even impossible to find it. Therefore, for example, under bars torsion the approximate method of Ritz [1] is used (which gives the erroneously low result) or Treftz method [2] (which gives the exaggerated result) or two methods together to reduce the error of the approximate solution. But among many engineering problems one of the most popular computational methods is the finite element method (FEM) [3] that is successfully used in solving various problems. However, the problem of FEM schemes improvement in order to decrease the expenses on their implementation is not only relevant in Ukraine but also in the whole world.

Analysis of recent research and publications. The first works using FEM for structural mechanics problems are published in [4]. With the advent of computer engineering it became possible to expedite bulky calculations and to obtain more accurate results. However, work on improving of the universal computing applications continues. So in recent research [5], [6] due to geometric modelling, new improved FEM models used in this article were obtained.

Description of general problem unsolved aspects. The problem of torsion rectangular cross section bars using new improved models was considered in [6]. The torque and maximum shear stress were determined. Then these results were compared to the exact results obtained in [7] and the results for the standard models. But the value of the shear stress in another dangerous point was not determined; also research for rectangular aspect ratio of more than 2.5 was not conducted at all.

The purpose of the study. To implement the FEM to solve the torsion problem of prismatic rectangular cross section bars using standard and alternative models and to analyze the accuracy of the results.

Main research. This work is dedicated to solve torsion problems of prismatic bars with rectangular cross section $2a \times 2b$ and $b \geq a$ (fig. 1).

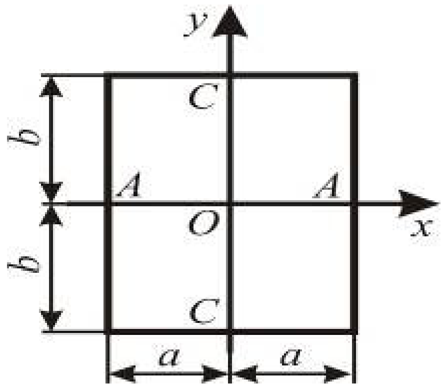

Figure 1 – The bar's cross section

The mathematical model of this problem is the Poisson equation

$$\frac{\partial^2 \varphi}{\partial x^2} + \frac{\partial^2 \varphi}{\partial y^2} = -2G\Theta \tag{1}$$

where φ – the stress function (the Prandtl function) must satisfy the differential equation (1) and be equal to zero along the edge of the cross section;

G – the modulus of elasticity of the second kind (shear modulus),

Θ – the relative angle of twist per unit of length.

The way of solving the equation (1) using the membrane analogy [7] is shown in the theory of elasticity.

The torque, knowing the stress function, is determined by the formula

$$M_t = 2\int_{-a}^{a}\int_{-b}^{b}\varphi(x,y)dxdy, \tag{2}$$

where a, b – half sides of the rectangular section (fig. 1).

The exact calculation of M_t is quite bulky and is reproduced in [7]. For practical purposes the simplified formula for M_t is used [7]

$$M_t = k_1 G\Theta(2b)(2a)^3, \tag{3}$$

where k_1 – numerical coefficient that depends on the ratio of b/a.

It is also argued that the maximum shear stresses $\tau_{\max}$ in the torsion of rectangular cross section bar occur at the surface in the middle of longer sides of rectangular section (in points A) and are equal [7]

$$\tau_{max} = \frac{M_t}{k_2(2b)(2a)^2}, \tag{4}$$

where k_2 – numerical coefficient that depends on b/a.

The shear stresses occured at the surface in the middle of smaller sides of rectangle (in points C) are smaller. They are calculated by the formula

$$\tau = k\tau_{max}, \tag{5}$$

where $k = k_1/k_2$.

Some values of k_1, k_2, k coefficients are listed in the tabl00e in [7].

The same problem was being solved in [6]. A variety of the standard models were used. For the further study, two models from [6], which produced the most accurate results, are used.

Model 1. We are using biquadratic elements of the serendipian family (8 nods) (fig. 2, *a*) [8, 9], covering the bar's cross section with the four 8-nods elements

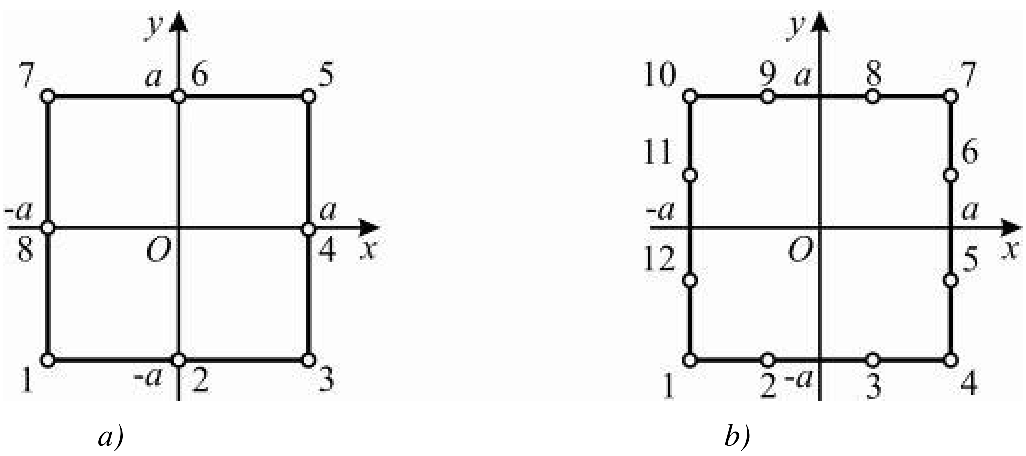

Figure 2 – Biquadratic (*a*) and bicubic (*b*) serendipian elements

The basic functions are [4]

$$N_1^{(1)}(x,y)=\frac{1}{4}(1-x)(1-y)(-1-x-y),$$
$$N_2^{(1)}(x,y)=\frac{1}{2}(1-x^2)(1-y). \tag{6}$$

Model 2. Bicubic serendipian elements (12 nods) (fig. 2, *b*) [8, 9], covering the bar's cross section with the four such elements are used. The basic functions are [4]

$$N_1^2(\xi,\eta)=\frac{1}{32}(1-\xi)(1-\eta)\left[-10+9(\xi^2+\eta^2)\right],$$
$$N_2^2(\xi,\eta)=\frac{9}{32}(1-\xi^2)(1-\eta)(1-3\xi). \tag{7}$$

In addition to standard models (6), (7), due to geometric modelling, new alternative models on these elements are obtained [5], and the unlimited quantity of models with different features through "scaling" them is received. By the solution of the inverse problem with account of the precise meaning for $\tau_{\max}$ for the square cross section, the new models of the biquadratic element were received [6]

$$N_1^{(*)}(x,y)=\frac{1}{2000}(1-x)(1-y)(-569-569x-569y-69xy),$$
$$N_1^{(*)}(x,y)=\frac{1}{2000}(1-x^2)(1-y)(1069+69y). \tag{8}$$

and on the bicubic element

$$N_1^{(**)}(\xi,\eta)=\frac{-1}{32000}(1-\xi)(1-\eta)\left[13789-12789(\xi^2+\eta^2)+3789\xi^2\eta^2\right],$$
$$N_2^{(**)}(\xi,\eta)=\frac{3}{64000}(1-\xi^2)(1-\eta)(7263-21789\xi+842\eta-842\xi\eta-421\eta^2+2947\xi\eta^2). \tag{9}$$

These optimized models were tested for solving problems of torsion of prismatic rectangular cross section bars with ratio sides of $1\le b/a\le 2{,}5$ only. In this article the study is continued. The relative error of calculations for $\tau_{\max}$ and M_t in comparison with accurate [7] for rectangles with the ratio of sides $b/a>2{,}5$ is determined. These results are shown in the table 1. The comparative analysis with the corresponding results for the standard models (6) and (7) are shown in this table as well.

Table 1 – Relative errors for $\tau_{\max}$ and M_t for rectangular cross section bar, [%]

b/a / Model	3,0		5,0		7,0		10,0	
	$\tau_{\max}$	M_t	$\tau_{\max}$	M_t	$\tau_{\max}$	M_t	$\tau_{\max}$	M_t
(6)	13,2	4,1	13,8	4,0	14.0	4,0	14,1	3,9
(7)	18,1	2,8	18,9	2,5	19,2	2,4	19,3	2,4
(8)	8,6	2,8	9,1	2.5	9,3	2,3	9,4	2,3
(9)	1,3	0,8	1,6	0,7	1,7	0,67	1,7	0,67

It has to be noted, that these results were obtained dividing the bar's cross section by only four elements. As shown in [6], covering the bar's square cross section with 16 elements significantly improves the results of calculation for standard models (6) and (7). The same situation occurs on rectangles with different sides ratio. The best result was obtained for the model (7).

However the determination of the shear stress τ in another dangerous point, which is located in the middle of the smaller side of rectangle (point C) using new improved models was not conducted in [6]. Note, that the determination of the value for τ in point C is necessary not only to get more results by different methods and compare them with accurate (although this is important), but also because it opens up the possibility to solve complex problems in which the bar of rectangular cross section works not only in torsion, but, for example, bending with torsion too. In this case it is difficult resistance. As it is known from the materials mechanics, the greatest normal stresses [10] occur in the points of smaller sides of the rectangle, thus in point C too. Therefore, when calculating bar structures for strength for the third or fourth failure theories the value of shear stress τ in point C can't be ignored because this value is also considerable. For the different ratios of rectangle's sides it is within $0{,}743\tau_{\max} \le \tau \le \tau_{\max}$ [7].

The obtained relative errors of calculations for τ in point C using new alternative models in comparison to precise [7] and standard models for different ratio of rectangle's sides b/a are shown in table 2.

Table 2 – Relative errors for τ in rectangular cross section bar, [%]

Model \ b/a	1,0	2,0	3,0	5,0	7,0	10,0
	τ	τ	τ	τ	τ	τ
(6)	2,37	9,12	12,6	13,8	14,6	15,2
(7)	12,06	14,3	16,1	17,6	18,4	18,7
(8)	0	3,0	4,2	5,3	5,4	5,6
(9)	0	0,32	0,81	0,94	1,2	1,24

Conclusions. The new alternative models of the finite element method on the biquadratic and bicubic elements were received. Using these models and dividing the bar's cross section only by four elements the quite precise results were obtained. While using the standard models, the acceptable results were obtained only covering the bar's cross section with 16 elements. But the amount of computing work was essentially increased.

The obtained results allow to solve various application problems of restoring of the physical fields that occur in technical systems and objects by developing new mathematical models.

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Received 20.02.2017

UDC 624.04:41

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INFLUENCE OF LONGITUDINAL FORCES FOR DETERMININGTHE FREQUENCY OF FREE OSCILLATIONS DISCRETE DYNAMIC SYSTEMS

In the research flat and spatial rod system as a part of many engineering and technical structures were considered. A new construction with a simultaneous increase quality of their design requires finding new ways to improve dynamic analysis and calculation of complex rod systems. It was analyzed, that longitudinal forces consideration when determining discrete dynamical systems free oscillations frequency together with use of modern computer hardware and software systems allows to reduce time for such calculations, and use the innovative schemes and methods that were previously inaccessible through large amounts of computing. The problem of road systems free oscillations frequencies with finite number of freedom degrees in bending with considering of longitudinal forces was considered. The examples of quantitative assessment influence of longitudinal forces on the frequencies of free oscillations were shown.

***Keywords:** free oscillations, longitudinal force, frequency, dynamic system, load.*

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УРАХУВАННЯ ПОЗДОВЖНІХ СИЛ ПРИ ВИЗНАЧЕННІ ЧАСТОТ ВІЛЬНИХ КОЛИВАНЬ ДИСКРЕТНИХ ДИНАМІЧНИХ СИСТЕМ

Розглянуто плоскі й просторові стержневі системи, що входять до складу багатьох інженерних і технічних споруд. Скорочення строків розробки нових конструкцій з одночасним підвищенням якості їх проектування потребує знаходження нових шляхів для удосконалення динамічного аналізу й розрахунку складних стержневих систем. Проаналізовано, що урахування поздовжніх сил при визначенні частот вільних коливань дискретних динамічних систем сумісно з використанням сучасної комп'ютерної техніки і програмних комплексів дозволяє зменшити час для проведення таких розрахунків, а також використовувати принципово нові схеми та методи, які були недоступні раніше через великі обсяги обчислень. Розглянуто задачу обчислення спектра частот вільних коливань стержневих систем із кінцевим числом ступенів вільності при згинанні з урахуванням поздовжніх сил. Наведено приклади кількісної оцінки впливу поздовжніх сил на частоти вільних коливань.

***Ключові слова:** вільні коливання, поздовжня сила, частота, динамічна система, навантаження.*

Introduction. The article contains results of theoretical research of longitudinal forces quantitative assessment influence in the spectrum of dynamic rod systems free oscillations frequencies with a finite number of freedom degrees in bending. The results of quantitative assessment impact forces in the longitudinal frequency spectrum free oscillation in bending shown as graphs depending on the free oscillations frequency of the longitudinal forces values.

Analysis of quantitative assessment influence of longitudinal forces in the spectrum of free oscillations frequencies in bending shows that longitudinal forces significantly alter the frequency of free oscillations (compressive – reduce the frequency and stretching – increase them). Longitudinal forces also took position on the frequency spectrum.

It is proved that the calculation of the free oscillations frequency spectrum must consider the influence of longitudinal forces.

Review of recent sources of research and publications. It is known that the physical properties of real structures exactly display discrete-continuous models, for which there is no comprehensive approach that allows with desirable accuracy and minimal expenses perform their full dynamic calculation [1, 2]. Additional difficulties arise when calculating spatial branching structures, they are connected with the necessity of modeling compatible oscillations, considering the density of the frequency spectrum and the influence of different factors: the transverse and longitudinal loads, type of concentrated impurities, shear median surface, deplanation and turn sections, damping, elastic links type and other boundary conditions. Even with modern computing tools and technologies many dynamic problems remain inaccessible for direct solution [3 – 5].

Parts of the common problems unsolved earlier. It is known that when calculating linear deformed rod systems with a finite number of freedom degrees on small free oscillations for calculating frequencies spectrum, frequency equation is solved [2 – 7]. The coefficients of this equation are expressed through relevant elements of matrix stiffness or pliability system. Usually elements of these matrices are determined by undeformed system scheme and depend on its mechanical properties. When calculating system by the deformed scheme manifested mutual influence of certain types deformations; for example, when the rod bent transverse load longitudinal forces causing an additional bending. This leads to the fact that the stiffness will depend on the longitudinal forces, and accordingly, free oscillations frequencies of such a dynamic system also depend on the longitudinal load.

Purpose of the work. To quantify the influence of longitudinal forces in the spectrum of dynamic rod systems free oscillations frequencies with finite number of bending freedom degrees.

Main material and results. Consider free oscillations of the cantilevered beam with a point mass *m* at its end (Figure 1, a). (*EI* = const; *EA* = const).

If the longitudinal force is zero, then the circular free oscillations frequency without considering the resistance forces ω was calculated by the formula

$$\omega = \frac{1}{\sqrt{m\delta_{11}}}, \tag{1}$$

where $\delta_{11} = l^3/3EI$ (Figure 1, b).

In the case where $N \neq 0$, calculate displacement δ_{11}^{c} from $F = 1$ (when $N<0$ – compression, Figure 1, c) and displacement δ_{11}^{p} (when $N > 0$ – stretching).

The corresponding differential equations of curved beam axis shall be:

$$-\textit{for } N<0, \qquad EIy'' = -Ny + \delta_{11}^{c}N + (\ell - x); \tag{2}$$

$$-\textit{for } N>0, \qquad EIy'' = Ny - \delta_{11}^{p}N + (\ell - x). \tag{3}$$

Denote $k^2 = N/EI$ and write the general solution for (2 and 3) in the form:

$$-for\ N<0,\qquad y=-\left(\frac{\ell+N\delta_{11}^{c}}{N}\right)\cos kx+\frac{1}{kN}\sin kx+\frac{(\ell-x)}{N}+\delta_{11}^{c}; \tag{4}$$

$$-for\ N>0,\qquad y=\left(\frac{\ell+N\delta_{11}^{p}}{N}\right)chkx-\frac{1}{kN}shkx-\frac{(\ell-x)}{N}+\delta_{11}^{p}. \tag{5}$$

For determination of the necessary displacements δ_{11}^{c} i δ_{11}^{p} it used use conditions: when $x=\ell$ and $N<0$ $y=\delta_{11}^{c}$; when $x=\ell$ and $N>0$ $y=\delta_{11}^{p}$.

Then from the equations (4) and (5) it is obtained:

– when $N<0$
$$\delta_{11}^{c}=\frac{\ell^{3}}{3EI}\frac{3(tgk\ell-k\ell)}{(k\ell)^{3}}; \tag{6}$$

– when $N>0$
$$\delta_{11}^{p}=\frac{\ell^{3}}{3EI}\frac{3(k\ell-thk\ell)}{(k\ell)^{3}}. \tag{7}$$

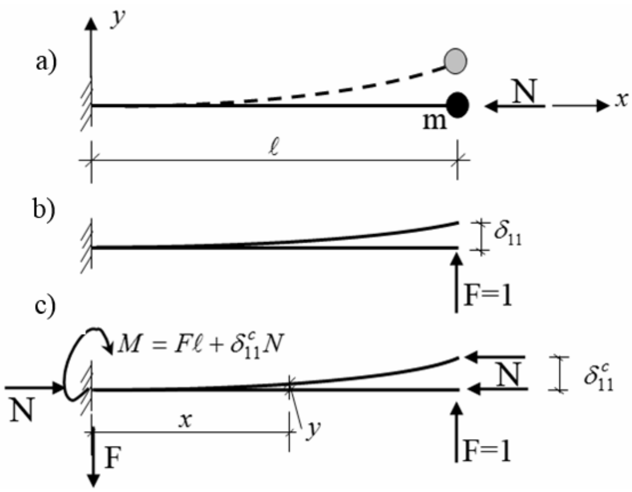

Figure 1 – Design scheme of the beam

Thus, when calculating free oscillations frequency considering longitudinal forces in the formula (1) instead δ_{11} it needs to substitute values calculated by formulas (6) and (7).

The results of displacements calculation δ_{11}^{c}, δ_{11}^{p} and also the frequencies ratio ω^{c}/ω, ω^{p}/ω are shown in figures 2 and 3.

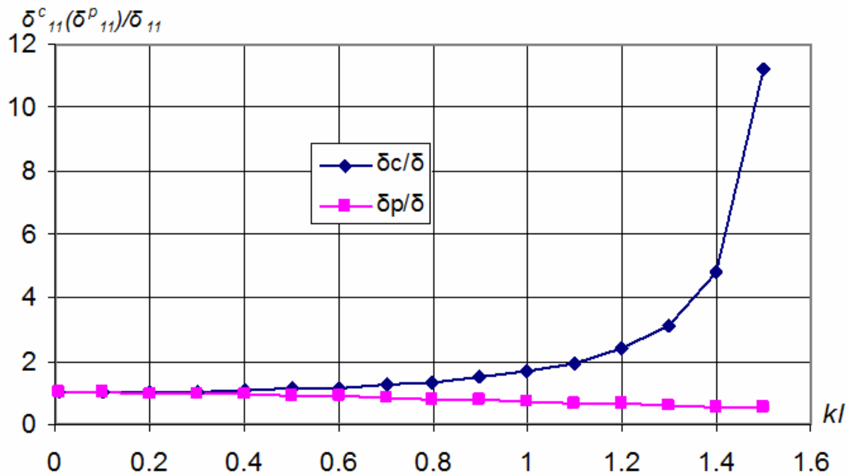

Figure 2 – Displacements δ_{11}^{c} and δ_{11}^{p}, calculated by the formula (6, 7) when $l^3/(3EI)=1$

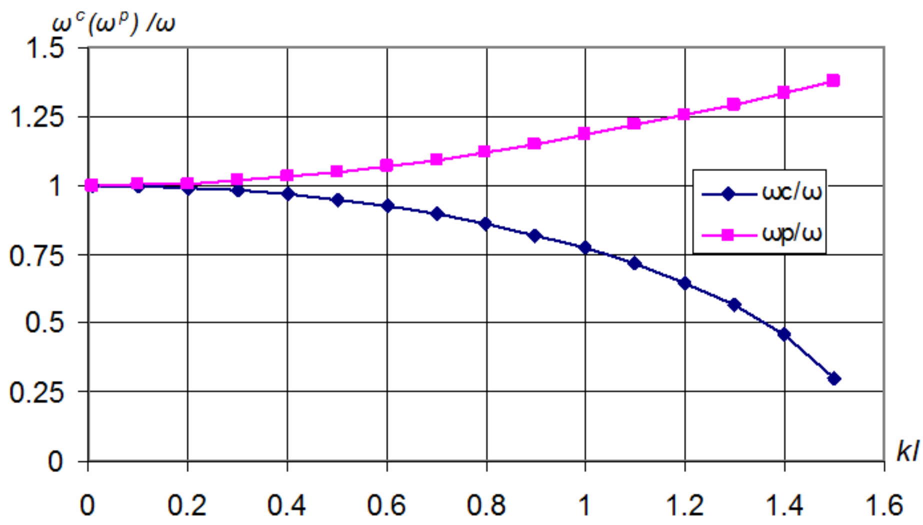

Figure 3 – The dependence of frequencies ratios ω^c/ω and ω^p/ω from *kl*

The dependence of frequency ω^c/ω on the ratio N/N_{cr} isillustrated in figure 4.

As can be seen from the graphics the influence of longitudinal compressing forces on the frequency of free oscillations is less than 5% to the ratio N/N_{cr} = 0,1. With increasing longitudinal forces, frequency reduced significantly.

For example, when N/N_{cr} = 0,58, ω^c/ω=0,65.

Consider the dynamic rod system with two degrees of freedom (figure 5).

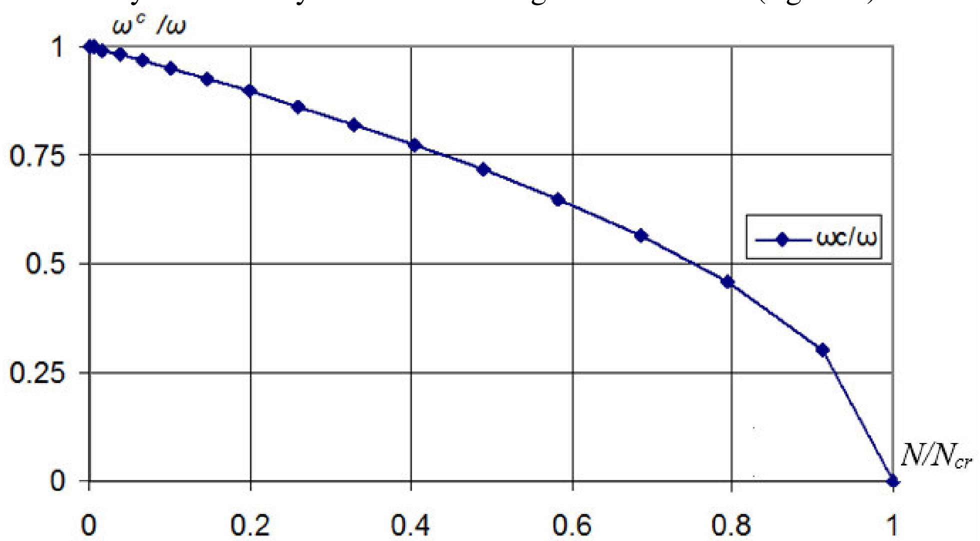

Figure 4 – The dependence of frequencies ratios ω^c/ω on the ratio N/N_{cr}

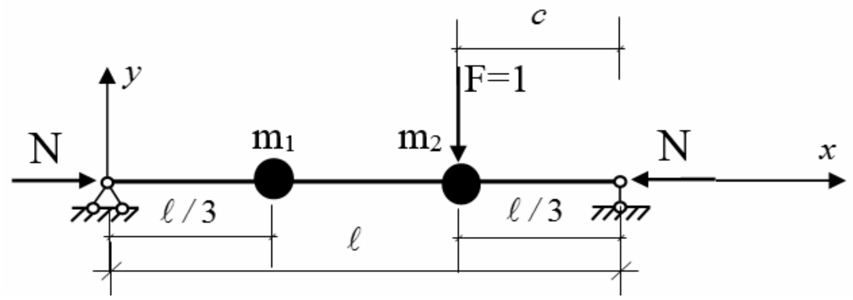

Figure 5 – Design scheme of the dynamic system ($m_1 = m_2 = m$)

Differential equations of curved beam axis (figure 5) have the form:

– for the area to the left of the force F when N<0 ($0 \le x \le \ell - c$)

$$EIy'' = -Ny - Fcx/\ell \ ; \tag{8}$$

– for the area to the right of the force F when N<0 ($\ell - c \le x \le \ell$)

$$EIy'' = -Ny - F(\ell - c)(\ell - x)/\ell \ ; \tag{9}$$

After solving the system of equations (8) and (9) it is obtained the equation of the bent axis which to the left of forces *F* (figure 5) has form [3]

$$y = \frac{F \cdot \sin(kc)}{Nk \cdot \sin(k\ell)} \sin(kx) - \frac{F \cdot c}{N \cdot \ell} x \ , \tag{10}$$

on condition that $k^2 = N/EI$.

Equation (10) allows to calculate displacement in the points of the dynamic system mass location from the *F=1*.

Considering that the dynamic system is symmetric, it is got:

when $x = 2l/3$, $c = l/3$, $\delta_{22} = \delta_{11}$,

when $x = l/3$, $c = l/3$, $\delta_{12} = \delta_{21}$.

Thus, the expressions for these movements will be:

$$\delta_{11} = \delta_{22} = \frac{F \cdot sin(k\ell/3)}{Nk \cdot sin(k\ell)} sin\left(\frac{2k\ell}{3}\right) - \frac{2F\ell}{9N} = \frac{F \cdot \ell}{N}\left(\frac{9 \cdot sin(k\ell/3) \cdot sin(2k\ell/3) - 2k\ell \cdot sin(k\ell)}{9k\ell \cdot sin(k\ell)}\right), \tag{11}$$

$$\delta_{12} = \delta_{21} = \frac{F \cdot sin(k\ell/3)}{Nk \cdot sin(k\ell)} sin(k\ell/3) - \frac{F \cdot \ell}{9N} = \frac{F \cdot \ell}{N}\left(\frac{9 \cdot sin(k\ell/3) \cdot sin(k\ell/3) - k\ell \cdot sin(k\ell)}{9k\ell \cdot sin(k\ell)}\right) . \tag{12}$$

$$\frac{F\ell}{N} = \frac{F\ell}{k^2 EI} = \frac{1 \cdot \ell^3}{(k\ell)^2 EI} \ .$$

When $k\ell = 0$, $\delta_{11} = \delta_{22} = \frac{8}{486} \cdot \frac{\ell^3}{EI}$, and $\delta_{12} = \delta_{21} = \frac{7}{486} \cdot \frac{\ell^3}{EI}$.

The results of calculating frequency spectrum ω_1 i ω_2 depending on $k\ell$ and their ratio ω_1/ω_2 are shown on figures 6, 7 and 8 respectively.

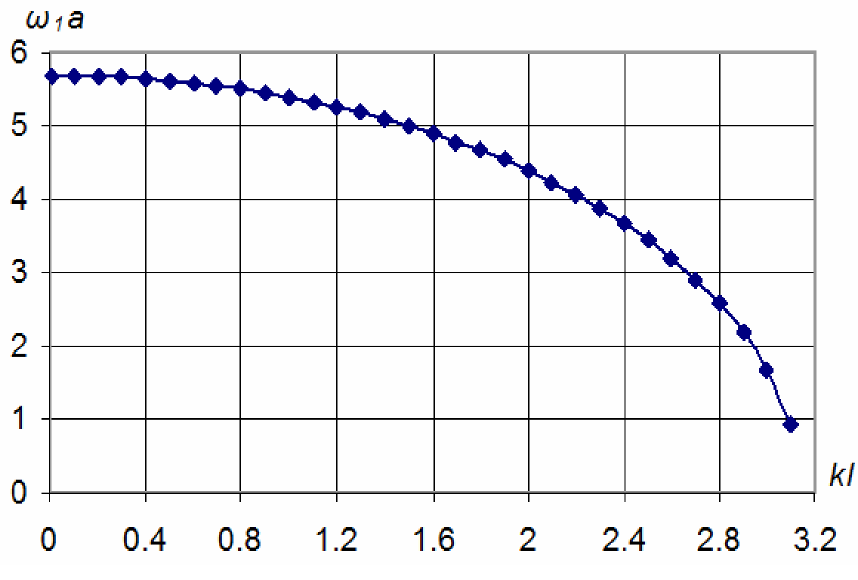

Figure 6 – The dependence of frequency ω_1 from $k\ell$ when $a = \sqrt{m\ell^3/(EI)}$

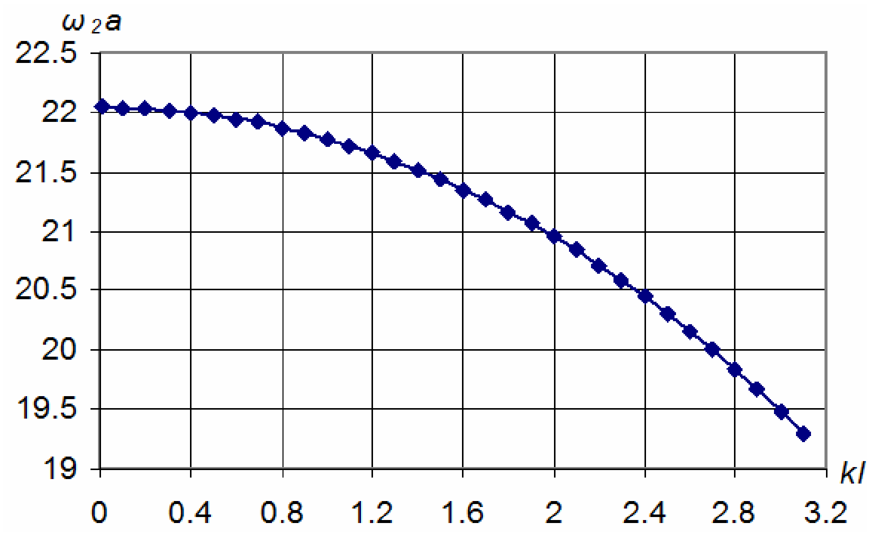

Figure 7 – The dependence of frequency ω_2 from $k\ell$ when $a = \sqrt{m\ell^3 /(EI)}$

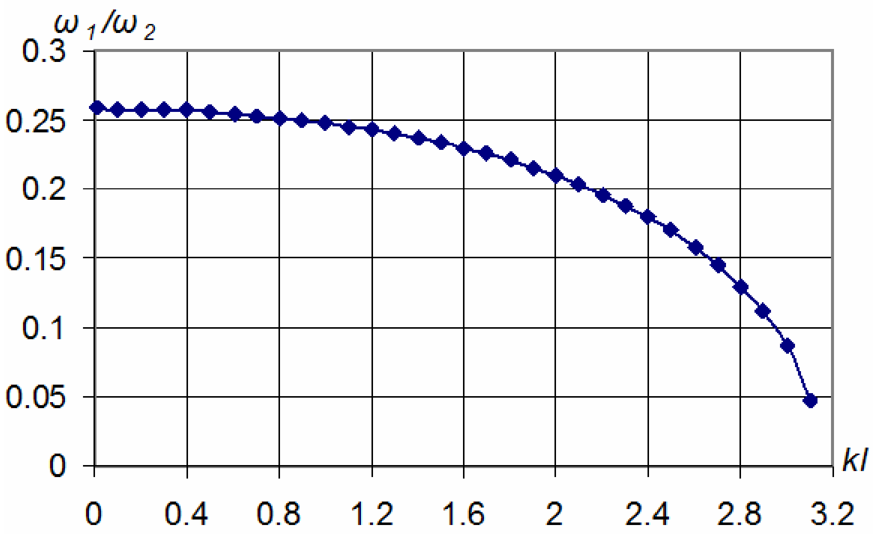

Figure 8 – The dependence of frequency ratio ω_1 / ω_2 from $k\ell$

The dependence of frequency ω_1 on the ratio N/N_{cr} is illustrated in figure 9.

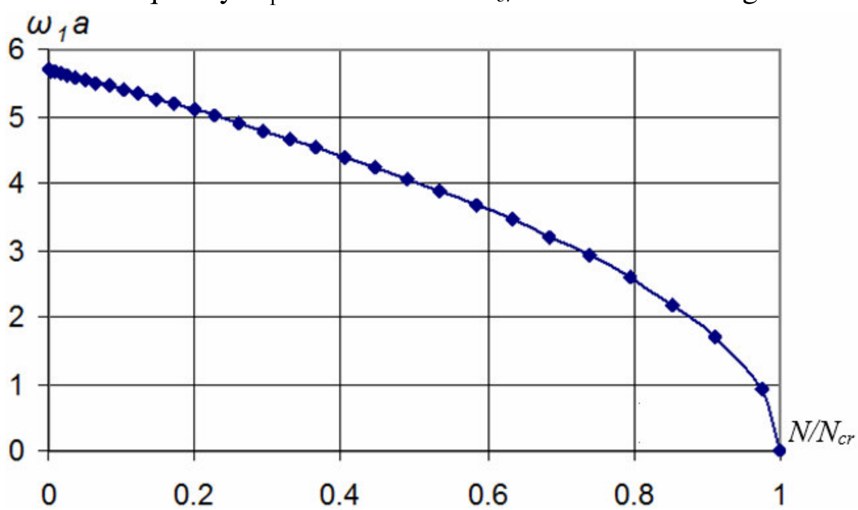

Figure 9 - The dependence of frequency $\omega_1 a$ on the ratio N/N_{cr} when $a = \sqrt{m\ell^3 /(EI)}$

As can be seen from the graphics, the influence of longitudinal compressing forces on fundamental tone frequency is less than 5% to the ratio N/N_{cr}=0.1. With increasing longitudinal forces, frequency significantly reduced.

For example, when $N/N_{cr}=1\cdot10^{-5}$ frequency $\omega_1 = 5{,}692/a$, and when N/N_{cr}=0,58 frequency decreased to $\omega_1 = 3{,}675/a$, which is 35,4%.

Conclusions. Analysis of longitudinal forces quantitative estimation influence on frequency spectrum of dynamic rod systems free oscillations with finite number of freedom degrees at bending are showed. Longitudinal forces considerably change the frequency of free oscillations (see figures 3, 4, 6, 7 and 9). Compressive forces reduce the frequency of free oscillations but stretching forces increase them. Longitudinal forces also are changing frequencies position on the frequency spectrum of free oscillations (see figure 8).

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Received 22.02.2017

UDC 624.011

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THE CONSTITUENT ELEMENTS STRUCTURES COVERING OF HYPERBOLIC PARABOLOID

Hypar is a hyperbolic paraboloid representing translational ruled developable anti classical surface, i.e., the surface of negative Gaussian curvature. Shaping of the parabolic elements corresponds to buckling of the shell and the main tensile forces are arranged in the ascending direction of parabolas, and the main compression force - in the direction of the descending parabola. Composite materials are formed from the combination of two or more layered materials, each having very different properties. ANSYS Composite PrepPost software provides all the necessary functionality for the analysis of layered composite structures. The paper discloses a possibility of using for shell covering negative curvature. Design solutions into constituent elements structures and computations such structures are presented.

Keywords: *Hypar, negative curvature, negative Gaussian curvature, ANSYS Composite PrepPost , shell, software.*

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ПОКРИТТЯ ТИПУ ГІПЕРБОЛІЧНИЙ ПАРАБОЛОЇД ЗІ ЗБІРНИХ ЕЛЕМЕНТІВ

Гіпар – гіперболічний параболоїд, що має трансляційну лінійчату нерозгортаючу антикласичну поверхню, тобто поверхню негативної гаусової кривизни. Формоутворення з параболічних елементів відповідає випинанню оболонки і головні зусилля розтягування розташовуються у напрямку висхідних парабол. Композиційні матеріали мають анізотропні характеристики, у зв'язку з чим їх часто використовують при вирішенні специфічних конструкторських завдань. ANSYS Workbench спеціалізований програмний продукт – ANSYS Composite PrepPost (ASP), у якого всі слойоні елементи дозволяють оцінювати міцність за допомогою різних критеріїв руйнування (критерій максимальних деформацій, напруги, цая-ву, хашин, гипотеза цая-хилл). Надається конструктивне рішення різних типів складових елементів, а також розрахунки таких конструкцій.

Ключові слова: від'ємна гаусова кривизна, оболонкові покриття, композиційні матеріали, ASP, критерії руйнування.

Introduction. The shell has low weight and at the same time very robust constructive form. The bending deformation property of shells due to the curvature of the surface has better performance comparatively to the plates. The middle surface of the shell – a hyperbolic paraboloid is ruled translational anti-classical surface, i.e. surface negative Gaussian curvature.

In building two types of shells are used:

– Shell with contour, composed of straight lines (with form of twisted rectangle or parallelogram);

– Shell with contour, composed of curved lines;

The middle surface of both types shells is identical –hyperbolic paraboloid. Shell with the contour of straight lines is widely used [1]. Shell of the constituent elements of certain sizes fill one petal (Figure 1) or petals system at shell size 36×24 м (Figure 2) [2-3].

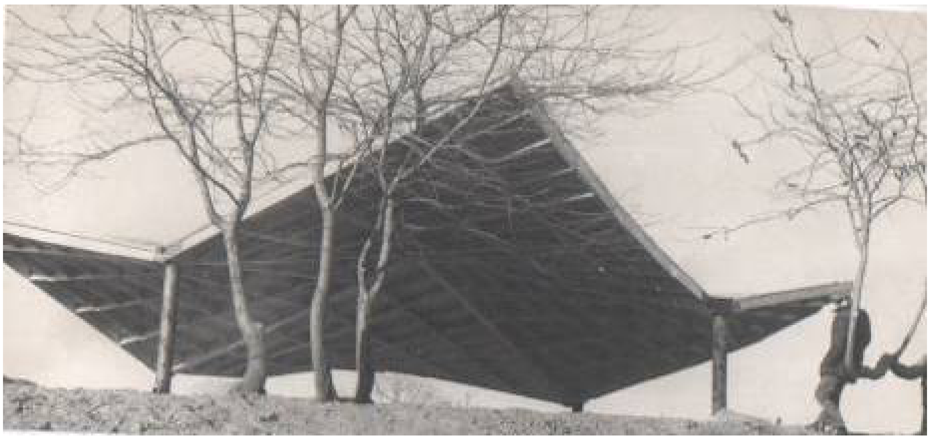


Figure 1 – General view of the shell 12×12 m

Figure 2 – A fragment of shell 36×24 m

Hyperbolic paraboloid surface causes certain difficulties for the approximation of the shell surface in the prefabricated components. Large resistance to this form shell buckling is explained by the main tensile forces located along the ascending direction of parabolas, and the main efforts of compression - in the direction of the descending parabola (Figure 3) and (Figure 4). The longitudinal joints are perpendicular compressive forces and focus on diagonals on the supporting part. Active load results in enhanced pressing element longitudinal of sandwich panels longitudinal edges are increasing. Parts shaping parabolic panels are detailed in the monograph [1].

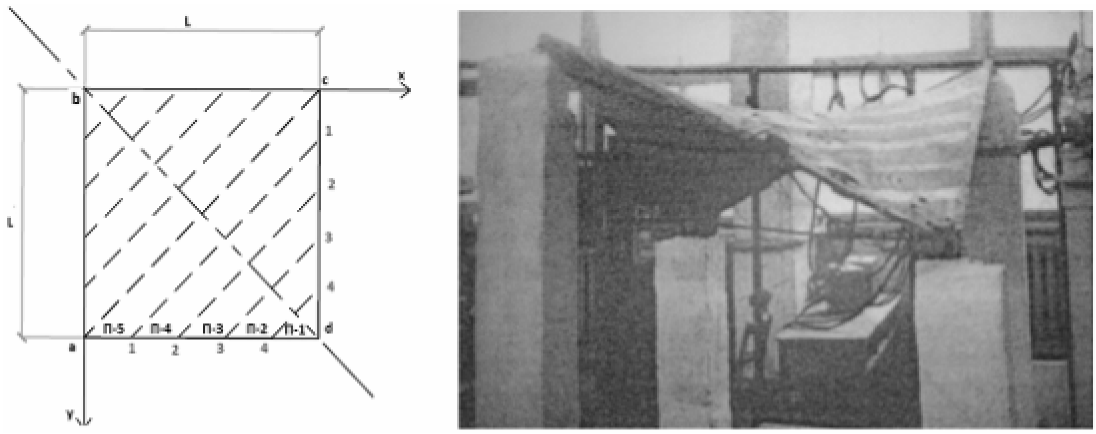


Figure 3 – The cutting surface of the shell to the petals parabolic prefabricated elements and shell parabolic profile size 1,5×1,5 m:
a – the ascending parabola (ac); b – the descending parabola (bd);
c – parabolic elements (П-1) – (П-5); d – the joints parabolic elements (1-1) – (4-4).

The problems of the construction industry are inextricably related with the reduction of material, labor and energy construction resources and buildings and structures operations. They are the cause of permanent searching new methods and improvement of existing design solutions. The most promising ways to address these problems, as well to achieve a significant reduction of the building and structures weigh while maintaining the highest strength and stiffness properties it is considering appropriate practical application of the construction lightweight design in the form of parabolic panels.

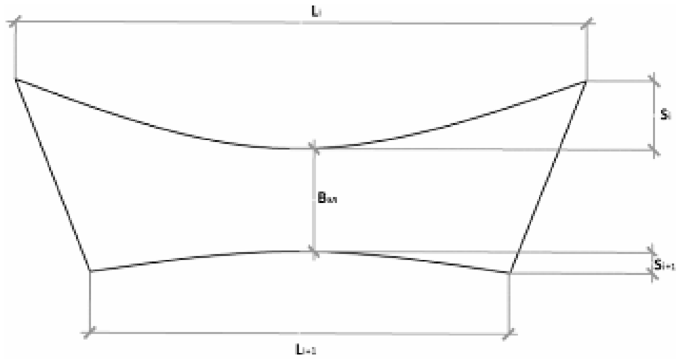


Figure 4 – General view of parabolic element

Review of the latest research sources and publications. Among the unique designs executed by individual projects and structures different from mass production in the first place by its shape, wooden hyperbolic shell are included. Such shell structures are received in Germany, England and several other European countries. Experimental study of shell hyperbolic from composite materials such as wood and plywood were investigated in a number of papers [1 – 7]. For example, in [2] presented testing data shallow shell model on quadratic plan, collected from 10 ribbed panels with twisted configuration were shown. Considering the fact that in Ukraine the construction of coverings experience with the use of combined structures is limited to only a few examples of shell hyperbolic, but it is possible to elaborate on the actual results of the their cost-effectiveness evaluation, performed in [5] and [6]. Particularly these works indicated that the two-piece shell construction plan dimensions of 2×10×10 m were fabricated by five qualified workers for only one month. By value, in practice shell structure can be compared with the design of modular panels of hyperbolic shell. The comparison shows that the shell is 30% more economical.

Problem statement. The aim of this work is to study the effect of forming on the carrying capacity and deformation property the shell in the shape of hyperbolic paraboloid. In this context, the authors assumed the construction of light metal-wood prefabricated hyperbolic coverings (Figure 5), composed from sandwich constructions of standard panels (Figure 3) and (Figure 4).

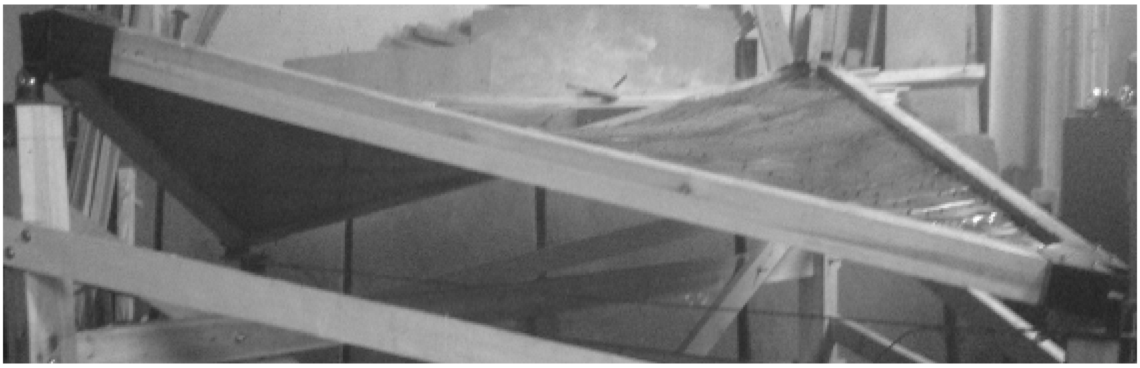
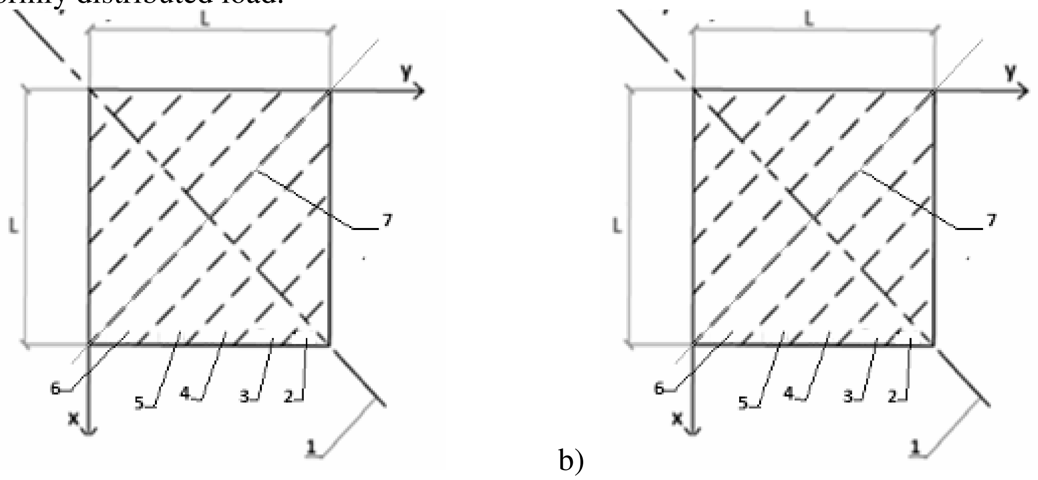


Figure 5 – Side view of the shell from a parabolic panel size 3×3×0,35 m in the laboratory of the department MW&PC, OSABA.

The effectiveness of structures created on the basis of sheet materials and nonflammable lightweight materials, is determined by their low weight, simplicity and speed of assembly, high operational reliability. Buildings erected by using light sandwich panels, have widely used in the world practice of building, which to some extent determines the relevance of their use. However, experience of design and operation discussed structures is insufficient relative to parabolic prefabricated elements (as the sandwich panels) (Figure 4). Sandwich panels consist of profiles skins exterior and interior, which are 0.4 mm thick, among that is an insulating layer of polyurethane foam, and specified step are cross wood elements. All panels are joined together through longitudinal frame timber elements, consisting of longitudinal and transverse members directed downward and upward parabolas.

The basic material and results. On the basis of calculations performed in the ANSYS software system [8 – 10], it was examined the influence shaping the shell (Figure 6) of the component elements considering pliability joints on the state of shell stress under the action of uniformly distributed load.



a) b)

Figure 6 – Comparison calculation models of two types of compound shell
a – shell from the parabolic elements;
b – shell from asymptotic elements as the a twisted rectangle;
1 – Diagonal axis of symmetry; (2-6) – Panels 1; 7 – Symmetry of typical panel axis

Examination of presented diagrams and graphs shows:

– Figure 7. The curves represented the maximum deflection under load for shell from the parabolic elements and asymptotic (in the form of a twisted rectangle) with the same suppleness in the joints of the elements. Comparison of deflection shells graphs shows that, vertical displacement increase considerably almost twice in comparison 2nd and 3th shells, thus shell from the parabolic elements perceives the load better than relatively asymptotic elements. Also in Figure 8. contour plots of displacements in z were depicted, which shows that the biggest deflections in three shell available at points is close to a quarter of the span. From the deflections distribution in (Figure 9) it is noted a great similarity between the first and 3th shells that substantially demonstrates the commitment of the 1st shell work as rigid shell, due to increase in the active load and clamping parabolic elements.

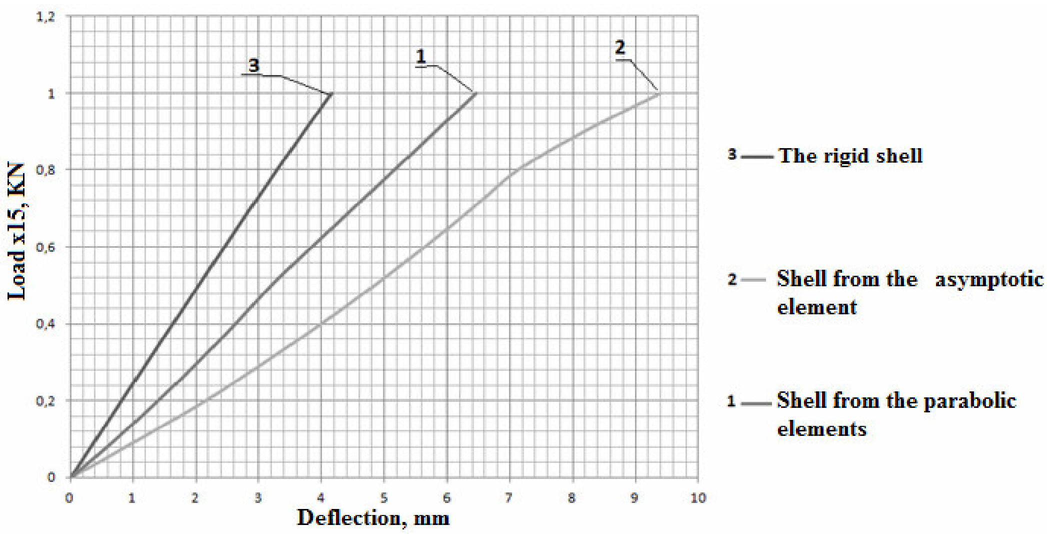


Figure 7 – The dependence of the load-deflection

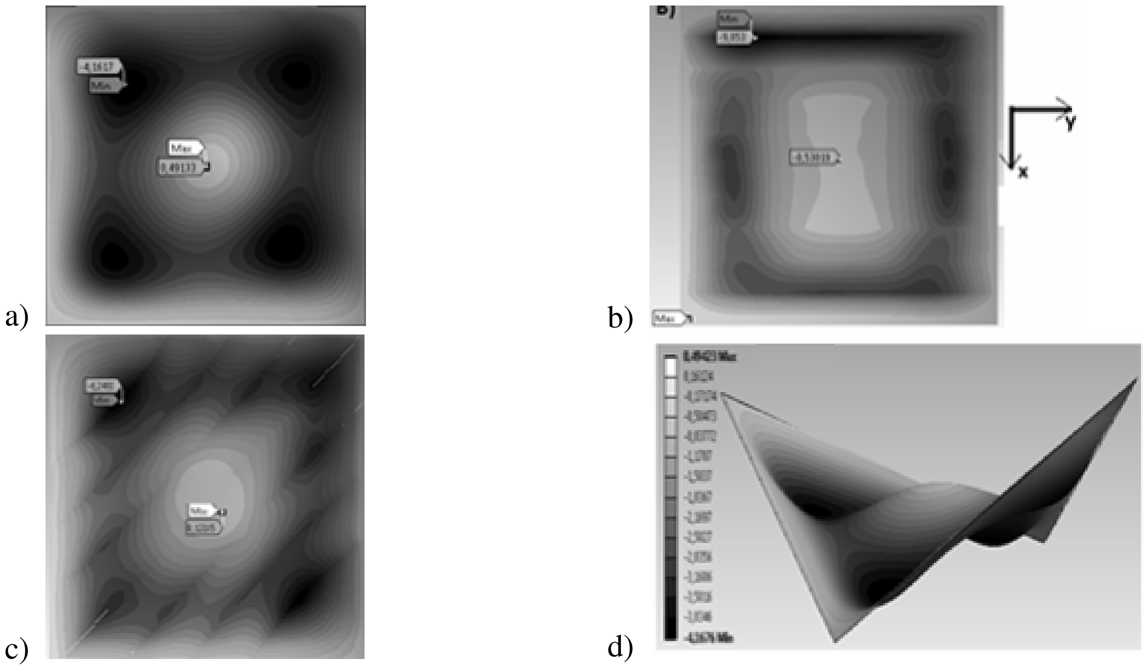


Figure 8 – Displacement contour plots in the direction Z

a – Rigid shell; b – Shell from the asymptotic elements;

c – Shell from the parabolic elements

– Figure 9 is characterized by elastic-plastic structure work area, and it is observed in the elastic deformation stage, large proximity works of first and 3th shells, while the 2nd shell experiences the most effort. Similarly, in (Figure 10) the main compressive stress, easily visible individuality stress distribution between the first and 3th shells is represented.

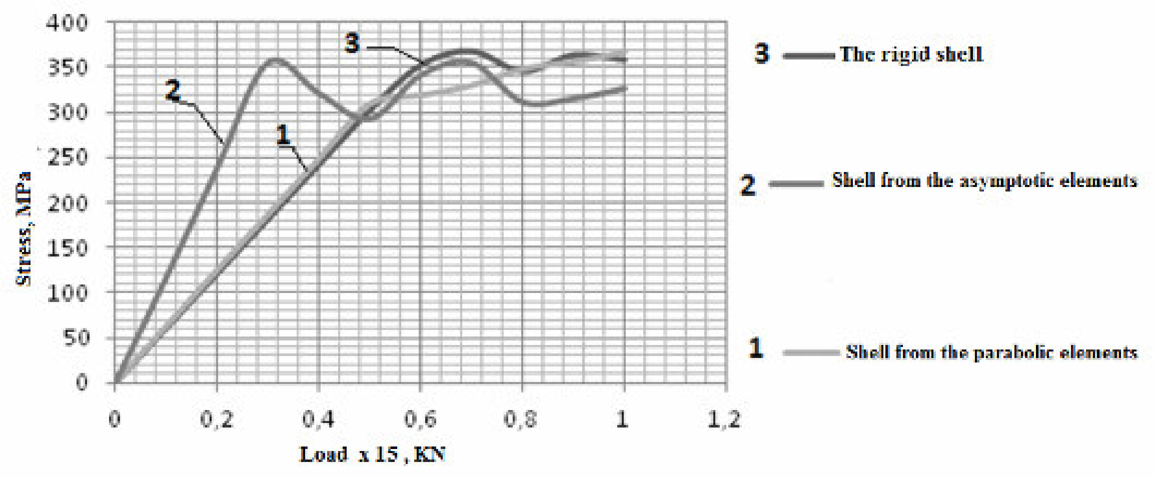


Figure 9 – The dependence of the maximum equivalent stresses the Von Mises on the load

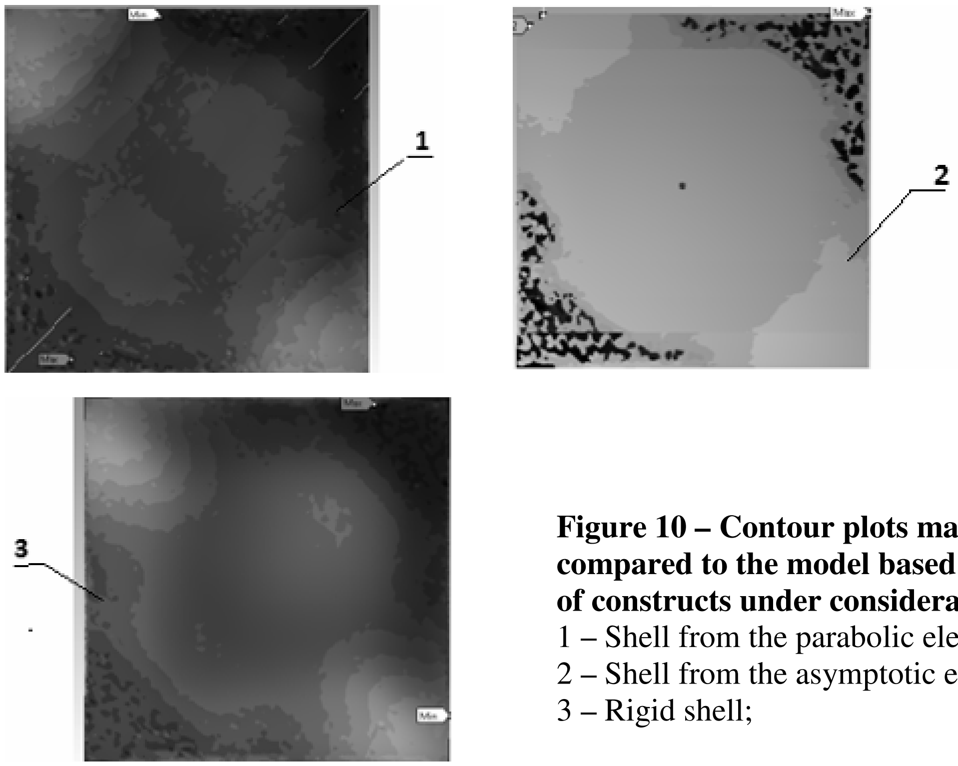


Figure 10 – Contour plots main stress, compared to the model based on three types of constructs under consideration shell:
1 – Shell from the parabolic elements;
2 – Shell from the asymptotic elements;
3 – Rigid shell;

– It is shown in (Figure 11) that the main compressive stress is much increased to bottom support, and reaches the highest value at points close to them, and they decrease in shells field, and they has decreasing character.

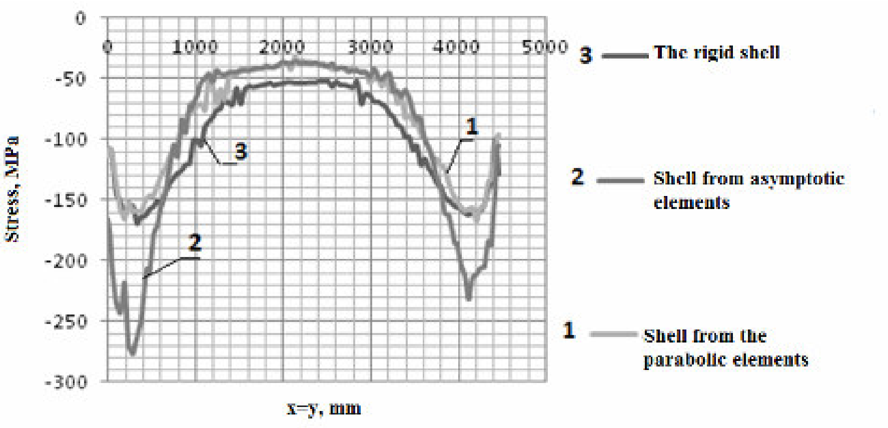


Figure 11 – The diagram of the main compressive stress in the shell along the line x = y

– From (Figure 12) in the middle zone of shells, the normal value of the stress is low, but in marginal zone they grow to the clamped edges, and may become of the same order of magnitude as the shear stress (Fig. 13). As a result, there is high probability formed biaxial compression zone.

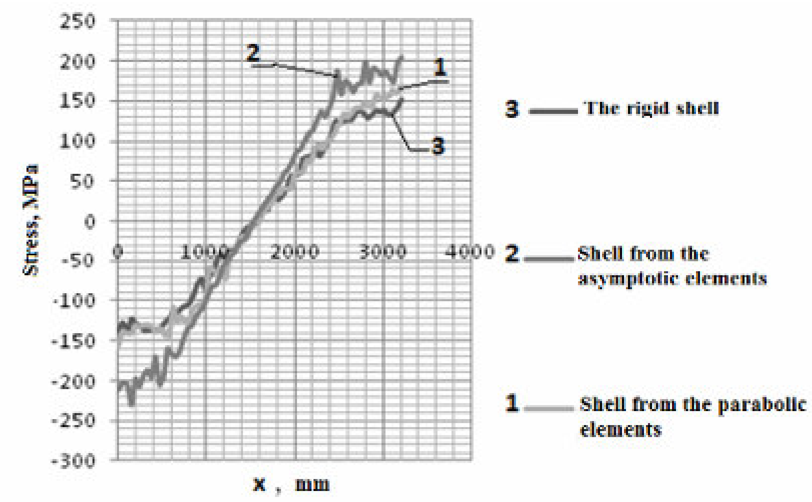


Figure 12 – The diagram of normal stresses in the shell along the line y = 0,03L

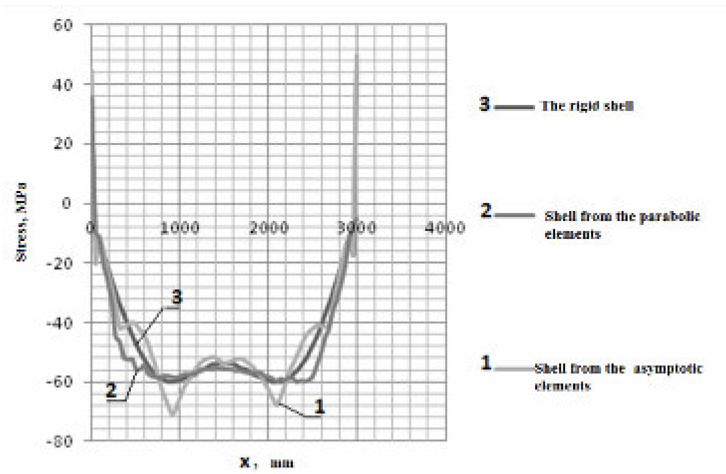


Figure 13 – The diagram of shear stress in the shell along the line y = 0,5L

Conclusions. Under uniformly distributed load, work shell using parabolic and asymptotic elements is substantially different. The use of parabolic panels unlike asymptotic, enhances compression in joints under conditions uniform load distribution.

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Received 04.03.2017

UDC 624.012.35:620.173/174

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STRENGTH OF REINFORCED CONCRETE IN BENDING ELEMENTS CALCULATIONS

The concept of the design strength of reinforced concrete in bending elements is proposed. This concept can be considered as generalized description of concrete. Such approach makes it possible to consider not only the separate strength of concrete and reinforcement, but also their interaction. The design strength of reinforced concrete is determined by ratio of force, which causes destruction of the standard reinforced concrete specimen, to the corresponding geometric characteristic. It was found that using the introduced concept reinforced concrete elements calculations can be reduced to well-known formulas of the strength of materials. It is based on generally accepted hypothesis and stress-strain diagrams of materials. The engineering method is developed, which allows to calculate the strength of the bending reinforced concrete elements of rectangular and circular cross-sections equally simple.

***Keywords:** design strength of reinforced concrete, bending, beam.*

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МІЦНІСТЬ ЗАЛІЗОБЕТОНУ В РОЗРАХУНКАХ ЗГИНАЛЬНИХ ЕЛЕМЕНТІВ

Запропоновано поняття розрахункового опору залізобетону в елементах, що зазнають згинання. Це поняття дозволяє розглядати узагальнену характеристику залізобетону. Такий підхід дає змогу враховувати не тільки окремо міцність бетону та арматури, а і їх взаємодію. Розрахунковий опір залізобетону визначається відношенням зусилля, при якому руйнується еталонний залізобетонний зразок, до відповідної геометричної характеристики. З'ясовано, що, використовуючи введене поняття, розрахунок залізобетонних елементів можна звести до загальновідомих формул опору матеріалів. У його основу покладено загальноприйняті гіпотези та діаграми деформування матеріалів. Розроблено інженерний метод, котрий дозволяє виконувати розрахунки міцності однаково просто згинальних залізобетонних елементів прямокутного та круглого поперечних перерізів.

***Ключові слова:** розрахунковий опір залізобетону, згинання, балка.*

Introduction. Modern methods of calculating of bending concrete elements involve the use of non-linear stress-strain diagrams. This allows to assess the strength of these elements more accurately not only with different reinforcement (single, double, distributed), but different in shape (rectangular, circular, tee, etc.) with common methodological aspects. But implementation of stress-strain diagrams [1, 2] into the practice of calculating of the building structures is needed in computer technology use, which often requires the results of computer testing verification by simple engineering techniques. One of these methods is proposed in this article.

Recent sources of research and publications analysis. Almost all existing approaches to create engineering design procedures of reinforced concrete elements are based on more accurate definition of design values of parameters used in the methods specified in the rules, particularly in the normative document [3]. Many scientists [4 – 8] devoted their works to the development of advanced engineering methods for calculating of the strength of bending reinforced concrete elements sections. Thus in their studies, they show that the use of non-linear stress-strain diagrams of materials in the calculation of the bearing capacity of reinforced concrete elements allows using unified methodology for calculation of almost all kinds of cross-sections and reinforcement. At the same time, these works suggest that for reinforced concrete elements with cross sections other than rectangular, solving many problems even with the use of computer technology is quite complex. Therefore there is still need for specific simplifications, including taking more simple stress-strain diagrams «$\sigma_c - \varepsilon_c$» and «$\sigma_s - \varepsilon_s$», or using specified ratios. This eliminates the majority of engineering techniques universality and the required accuracy of the calculation, which is inherent in a general method based on nonlinear deformation model, implemented with the help of computer technology.

Specifing still unsolved aspects of the problem. Designed for today engineering methods of calculation of the bearing capacity of bending reinforced concrete elements mainly is devoted to elements with rectangular and T cross-section with the single and in some cases double reinforcement. Practical recommendations for the strength calculations of elements of a circular cross-section are largely absent. But these elements can be successfully used in construction practices, and lack of proper engineering calculation methods inhibits their widespread adoption. There is a need for a practical engineering method of calculation of bending reinforced concrete elements with a round cross section because of this.

Assignment formulation. The main objective is to develop practical method of strength calculation of normal sections of reinforced concrete elements with rectangular and round cross-section based on the entered for them design characteristic – strength of reinforced concrete.

Basic material and results. In general, the design strength of the material is determined by the ratio of force, under which the sample of the material is destroyed, to the corresponding geometrical characteristic. Area of the section of specimen is used to find this characteristic under compression. Resistance moment of the section is used in bending. This approach to determination materials characteristics regulates the testing of samples from a single material. As a result, the strength and deformation characteristics of concrete are set without considering the strength of materials, working with it, such as steel reinforcement. To eliminate this gap, this paper proposes to use generalized characteristic – the strength of reinforced concrete – instead independent design characteristics of concrete and steel. Thus, this characteristic is determined by testing samples of reinforced concret. This approach defines reinforced concrete given in standarts as a material formed of concrete and principal reinforcement [1; Б6]. Thus in the process of identifying synthetic material strength like reinforced concrete, the fact that it consists of two materials: concrete and steel is considered. Using this approach, the concept of reinforced concrete design strength as a composite material, in general, is presented in the following formula:

$$f_1(a_1,\ldots a_n) = F_{Ed} / f(b_1,\ldots b_n), \tag{1}$$

where $f_i(a_1,...a_n)$ – design strength of composite material (reinforced concrete) in the section of the element provided the destruction in i-composite (material), MPa;

F_{Ed} – external force factor, the value of which corresponds to the destruction of concrete elements in the considered section;

$f(b_1,...b_n)$ – corresponding geometrical characteristic;

$a_1,...a_n$ – physical and mechanical parameters of materials, cross section of element consists of;

$b_1,...b_n$ – geometric parameters of cross section of reinforced concrete elements.

It is known that the strength of sections of reinforced concrete elements depends not only on the strength of materials, but the location of reinforcement in which (single, double, distributed). Therefore, the design strength of reinforced concrete elements with different reinforcing schemes will have a different value.

Design strength of reinforced concrete is determined by the minimum value of strength provided the section collapse of the element in all materials it consists of,

$$f = \min\left(f_1(a_1,...a_n),...f_i(a_1,...a_n),...f_n(a_1,...a_n)\right), \tag{2}$$

where f – total material design strength (reinforced concrete) of the section.

The following conditions are accepted in order to determine the dependencies for f values:

1. Flat cross-section hypothesis are considered, ie,

$$\frac{1}{r} = \frac{\varepsilon_c}{x} \quad or \quad \varepsilon_c = \frac{1}{r}x, \tag{3}$$

where x – neutral axis depth;

$1/r$ – curvature.

Violation of the flat cross-section hypothesis occurs during the formation of cracks due to sharp reduction of force in the tension zone. That is the so-called «scrapping» of the section in compressed and tensioned zones. The lower the reinforcement ratio is, the greater «scrapped» one is. The section straightens again with further deformation of element. In general, the linear strain distribution in the section of element identifies the principle of minimum potential energy. Thus, the smaller the difference between the time point of cracking and the time point of destruction is, the more strains linearity is disturbed. But elements of this reinforcement ratio is rather limited in building practice, besides there are structural limitations of minimum reinforcement ratio.

2. The stresses in the concrete compressed zone are distributed according to the «stress – strain» diagram that meets the ultimate deformation criteria [4, 5]. This diagram can be obtained by testing standard concrete prisms in hard mode or by transformation diagrams «$\sigma_c - \varepsilon_c$», obtained by testing prisms according to norms.

3. Stresses in reinforcing steel are described by idealized bilinear diagram.

4. Work of tensioned zone of concrete is not included.

The equilibrium equation is worked out using the accepted prerequisites for any section of element shown in Figure 1:

$$N_c = \sum_{i=1}^{n_s} N_{si} \ , \tag{4}$$

$$M_c = \sum_{i=1}^{n_s} M_{si} = M_{Ed} \ , \tag{5}$$

where M_c, M_{si} – respectively internal bending moments in concrete and in the i-th reinforcing bar about the neutral axis;

n_s – number of reinforcing bars in the section of the element.

After simple transformations using assumptions, it is got:

$$\int_0^{A_c} \sigma_c \left(x, \varepsilon_c \right) dA_c = E_s A_{si} \sum_{i=1}^{n_s} \left(\frac{k_i d}{x} - 1 \right), \tag{6}$$

$$\int_0^{A_c} \sigma_c \left(x, \varepsilon_c \right) x dA_c + E_s A_{si} \sum_{i=1}^{n_s} \varepsilon_c x \left(\frac{k_i d}{x} - 1 \right)^2 = M_{Ed} \ , \tag{7}$$

where $\sigma_c(x, \varepsilon_c)$ – the function of stress distribution in the concrete compressed zone obtained under the premise 2;

A_{si} – area of the i-th bar in the section of reinforced concrete element.

It is necessary to equate (6) and (7) apply the criteria of destruction to determine the bearing capacity of bending element in the section under consideration. The next dependencies are taken as criteria of destruction:

$$\begin{cases} \dfrac{dM_{Ed}}{d\varepsilon_c} = 0 \quad when \quad \varepsilon_c \le \varepsilon_{cu}, \\ \sigma_{si} = \varepsilon_{si} E_s \le f_{yd}, \quad i = 1...n, \\ \quad \varepsilon_{si} \le \varepsilon_{su}, \quad i = 1...n. \end{cases} \tag{8}$$

In the system of equations (8) ε_{su} – relative ultimate deformations of reinforcing steel meet the yield strength, ε_{cu} – maximum possible relative deformations of compressed concrete.

Expressions (4), (5) and (6) make it possible to calculate the bearing capacity of bending concrete elements, after taking the stress-strain diagram for concrete. This calculation is usually performed by iterations using a computer.

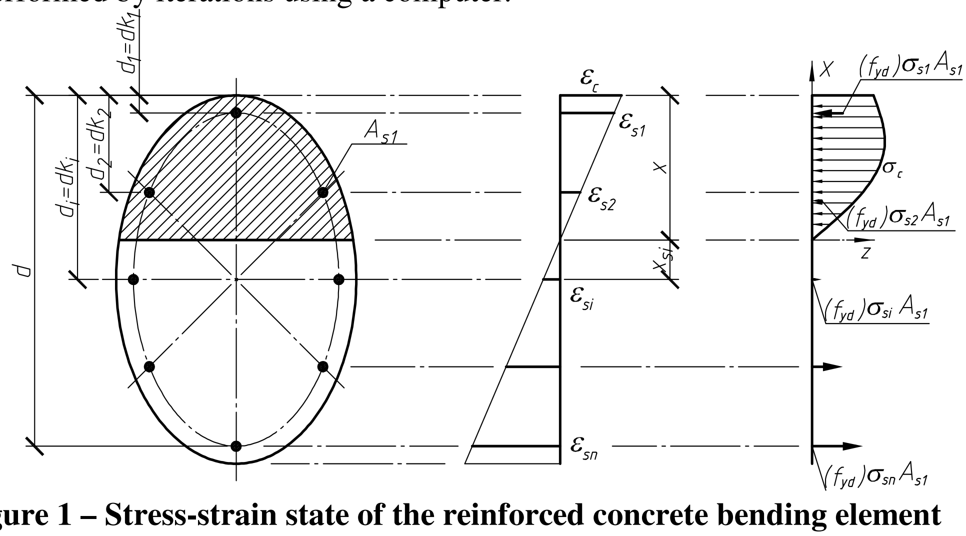

Figure 1 – Stress-strain state of the reinforced concrete bending element

To get the basic equation of reinforced concrete strength considering that in case of rectangular cross sections

$$A_c = b\,d\,, \quad W_c = bd^2/6\,, \tag{9}$$

and in case of circular cross sections

$$A_c = \pi d^2 / 4\,, \quad W_c = \pi d^2 / 32\,, \tag{10}$$

equations (6) and (7) for element with a circular cross section (including addictions (10) and symbols in Figure 2) are brought into view:

$$\frac{\int_0^{A_c} \sigma_c dA_c}{\pi d^2 / 4} = \frac{E_s \rho_f}{n_s} \sum_{i=1}^{n_s} \left(k_i k - 1\right), \tag{11}$$

$$\frac{\int_0^{A_c} \sigma_c x dA_c}{\pi d^3 / 32} + \frac{E_s \rho_f \varepsilon_c}{k n_s / 8} \sum_{i=1}^{n_s} \left(k_i k - 1\right)^2 = \frac{M_{Ed}}{W_c}\,. \tag{12}$$

The following designations are introduced in deriving dependencies (9) and (10):

$$\rho_f = \frac{4 A_{si} n_s}{\pi d^2}, \quad k = \frac{d}{x}, \quad n_a = \frac{a_s}{d} \tag{13}$$

All designations are conformed to ones given by [4, 5].

The type of function-approximation «$\sigma_c - \varepsilon_c$» of stress-strain state of concrete is substantially irrelevant for further change dependencies (11) and (12). The main thing is that it contains conventional parametric points.

Integrals in expressions (11) and (12) can be established by the direct integration of functional relationships or by replacing the curved chart into barchart. The integral sign can be replaced by corresponding summation symbol in this case. Inacurracy of this method is equally predetermined when a sufficient number of areas. The transcendental equations will be received with direct integration of analytical expression «$\sigma_c - \varepsilon_c$», solution of which can be performed by iterative methods.

The second method is considered in more details. The compressed zone of circular cross-section is presented with a certain number of parts (Fig. 2). The strain values is assumed constant within each of parts and those that are corresponded to the value of strain at the gravity centre of each rectangle (Fig. 2).

In this case, the internal forces in the concrete compressed zone are determined by the formulas:

$$N_c = \int_0^{A_c} \sigma_c\left(x, \varepsilon_c\right) dA_c = \sum_{i=1}^{n_c} \sigma_{ci}\left(\varepsilon_{ci}\right) b_i h_n\,, \tag{14}$$

$$M_c = \int_0^{A_c} \sigma_c\left(x, \varepsilon_c\right) dA_c = \sum_{i=1}^{n_c} \sigma_{ci}\left(\varepsilon_{ci}\right) b_i h_n x_{li}\,. \tag{15}$$

The strains of the concrete compressed zone of the i-th part are defined by the expression

$$\varepsilon_{c,i} = \varepsilon_c \left(1 + \frac{1 - 2i}{2 n_c}\right), \tag{16}$$

where n_c – number of areas on which the concrete compressed zone is broken; $i = 1 ... n_c$.

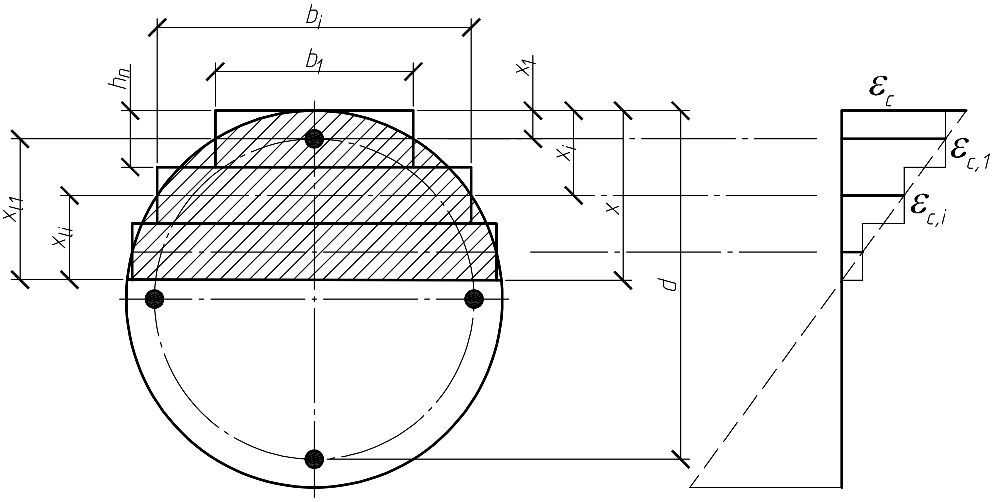

Figure 2 – To the definition of internal forces
in the concrete compressed zone of a circular profile element

Finally, equations (11) and (12) are taken the following form after simple transformations:

$$\sum_{i=1}^{n_c}\left(\sigma_c(x...)\frac{8\varepsilon_c\left(1+m\right)}{\pi n_c k}\sqrt{\frac{\left(1+n_a\right)^2}{4}-\left(\frac{1+n_a}{2}+\frac{m}{k}\right)^2}\right)=\frac{E_s\rho_f}{n_s}\sum_{i=1}^{n_s}\left(k_i k-1\right), \tag{17}$$

$$\sum_{i=1}^{n_c}\left(\sigma_c(x...)\frac{\varepsilon_c\left(1+m\right)^2}{\pi n_c k^3/64}\sqrt{-m\left(k+n_a k+m\right)}\right)+\frac{E_s\rho_f\varepsilon_c}{kn_s/8}\sum_{i=1}^{n_s}\left(k_i k-1\right)^2=\frac{M_{Ed}}{W_c}, \tag{18}$$

where

$$m=(1-2i)/n_c\,. \tag{19}$$

The left side of expression (18) in conjunction with criteria (8) is the formula for determining the design strength of reinforced concrete of the circular section of bending elements, ie,

$$f_{zM}=\sum_{i=1}^{n_c}\left(\sigma_c(x...)\frac{\varepsilon_c\left(1+m\right)^2}{\pi n_c k^3/64}\sqrt{-m\left(k+n_a k+m\right)}\right)+\frac{E_s\rho_f\varepsilon_c}{kn_s/8}\sum_{i=1}^{n_s}\left(k_i k-1\right)^2. \tag{20}$$

Similar expressions are obtained for reinforced concrete bending elements with rectangular cross-section with a single, double and distributed reinforcement [8]. The concept of the strength of reinforced concrete can be used in calculations of bearing capacity of reinforced concrete elements. The tables of the strength of reinforced concrete values are stacked depending on the reinforcement ratio and the strength class of concrete and reinforcement.

Tables 1 – 3 shows the values of the design strength of reinforced concrete for various classes of materials and reinforcement ratios for cross-section of the bending elements. Function (3.4) in the norms [1] is accepted as the stress-strain curve of concrete "$\sigma_c-\varepsilon_c$" in calculating these values.

The general condition of normal sections strength of the reinforced concrete elements in bending is given by

$$f_{zM} \geq \frac{M_{Ed}}{W_c}, \tag{21}$$

where $f_{zM} = f(C, \rho_f, f_{yd})$– design strength of reinforced concrete in the bending elements taken from Tables 1 – 3;

W_c – elastic moment of resistance of the effective section of concrete: $W_c = bd^2/6$ – for the rectangular cross-sections;

$W_c = \pi d^3/32$ – for the circular sections.

Condition (21) allows to solve all problems with the design of reinforced concrete elements: to check for their carrying capacity in the considered section, to define reinforcement. This method allows to reduce calculation method of reinforced concrete elements to classical strength of materials method, but considering its nonlinear deformation. It should be noted that the proposed method of calculation is not simplified. After all, the strength of the cross-section calculated using the formula (21) for the corresponding values of reinforcement ratios coincides with the strength determined by computer according to the norms [1] in the relevant software systems.

The method is universal. It can be used for various loads: permanent, cyclic, impact, dynamic and others. It is necessary to obtain stress-strain diagram for concrete under corresponding loads at first. The proposed method of calculation significantly reduces the process of designing the reinforced concrete elements and in many cases allows to avoid errors when using the computer.

The application of the proposed method is illustrated by the example of definition of the required amount of reinforcement in reinforced concrete elements.

Task 1 (see Fig. 3). Calculate the area of reinforcement required for reinforced concrete bending elements with concrete class *C20/25* and steel class *A400C* if the external bending moment $M_{Ed} = 150\ kHm$. The sections of the elements are:

1) rectangular section with single reinforcement, $b \times d = 40 \times 55\ cm$;

2) rectangular section with double reinforcement, $b \times d = 40 \times 55\ cm$;

3) round section diameter $D = 60\ cm$ with the effective depth of the section $d = 55\ cm$.

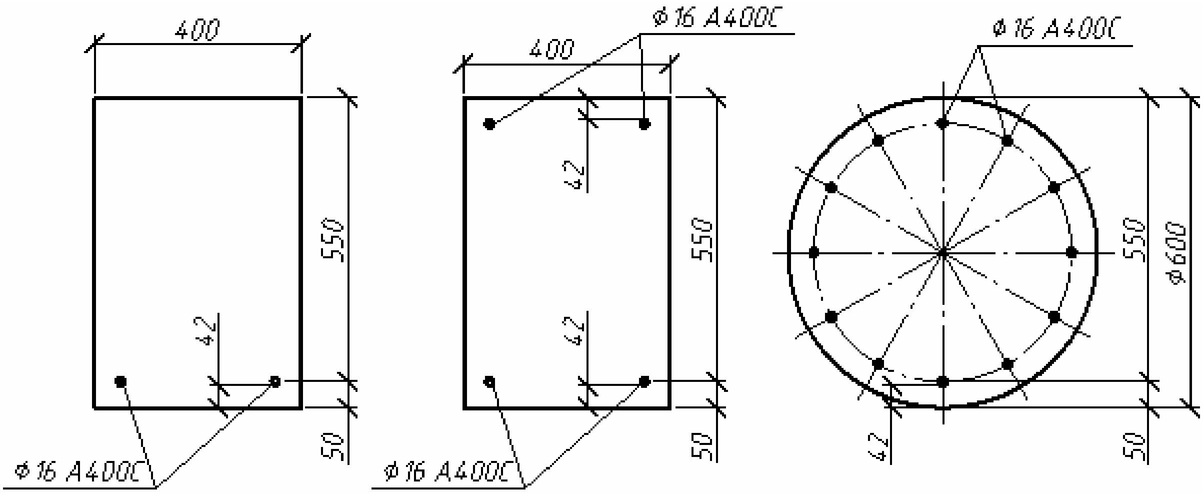

Figure 3 – To determination the cross-sectional area of reinforcement for bending elements

Solution. The required design strength of reinforced concrete:
– for rectangular sections

$$f_{zM} = \frac{M_{Ed}}{W_c} = \frac{6M_{Ed}}{bd^2} = \frac{6\times150\times10^3}{40\times55^2} = 7,44\ MPa;$$

– for circular sections

$$f_{zM,r} = \frac{M_{Ed}}{W_c} = \frac{M_{Ed}}{0,1d^3} = \frac{150\times10^3}{0,1\times55^3} = 9,02\ MPa.$$

The necessary reinforcement ratios are finded in corresponding tables:
– rectangular section with a single reinforcement $\rho_{f1}=0,363\%$;
– rectangular section with double reinforcement $\rho_{f2}=0,355\%$;
– circular section $\rho_{f3}=0,722\%$.
Determine the cross-sectional area of reinforcement:
– rectangular section with a single reinforcement
$A_s=\rho_{f1}\times bd=0,363/100\times40\times55=7,99$ *см*^{2},
take *4∅16 A400C*, $A_s=8,04$ *см*^{2} (see Fig. 3);
– rectangular section with double reinforcement
$A_s = A_{sc}=\rho_{f2}\times bd=0,355/100\times40\times55=7,81$ *см*^{2},
take *4∅16 A400C*, $A_s= A_s =8,04$ *см*^{2},
$A_s+ A_s=16,08$ *см*^{2} (see Fig. 3);
– circular section
$A_s=\rho_{f3}\times\pi d^2/4=0,722/100\times3,14\times55^2/4=17,15$ *см*^{2},
take *12∅16 A400C*, $A_s=24,13$ *см*^{2} (see Fig. 3).

Table 1 – The design values of reinforced concrete strength f_{M1} for bending elements with a single reinforcement (f_{yd} = 375 MPa; A400C), MPa

Concrete class	Reinforcement ratio ρ_f							
	0.05	0.5	1	1.25	1.75	2	2.5	3
C8/10	1.078	9.205	13.545	13.992	14.590	14.802	15.123	15.354
C12/15	1.083	9.714	16.956	18.643	19.626	19.982	20.531	20.936
C16/20	1.086	10.033	18.233	21.646	25.196	25.736	26.582	27.217
C20/25	1.088	10.220	18.981	22.814	29.359	30.978	32.138	33.021
C25/30	1.089	10.326	19.403	23.474	30.679	33.807	36.553	37.645
C30/35	1.089	10.405	19.718	23.966	31.643	35.072	40.687	42.062
C32/40	1.090	10.465	19.961	24.345	32.386	36.043	42.630	46.307
C35/45	1.091	10.522	20.189	24.702	33.086	36.957	44.058	50.238
C40/50	1.091	10.561	20.342	24.941	33.554	37.568	45.013	51.679
C45/55	1.091	10.592	20.468	25.137	33.939	38.071	45.799	52.810
C50/60	1.092	10.623	20.590	25.328	34.313	38.560	46.563	53.911

Table 2 – The design values of reinforced concrete strength f_{M1} for bending elements with double symmetrical reinforcement (f_{yd} = 375 MPa, A400C, n = 0,06-0,1), MPa

Concrete class	Reinforcement ratio ρ_f						
	0.05	0.5	1	1.25	1.5	2	2.5
C8/10	1.089	10.341	20.621	25.764	30.907	41.196	51.486
C12/15	1.108	10.370	20.647	25.788	30.930	41.216	51.504
C16/20	1.128	10.405	20.679	25.818	30.959	41.243	51.529
C20/25	1.146	10.438	20.712	25.851	30.990	41.272	51.556
C25/30	1.159	10.466	20.740	25.877	31.016	41.297	51.580
C30/35	1.171	10.494	20.766	25.904	31.042	41.322	51.604
C32/40	1.182	10.520	20.794	25.930	31.068	41.347	51.628
C35/45	1.194	10.550	20.827	25.962	31.098	41.376	51.656
C40/50	1.203	10.574	20.855	25.989	31.124	41.399	51.678
C45/55	1.212	10.597	20.882	26.017	31.151	41.423	51.699
C50/60	1.221	10.623	20.915	26.050	31.184	41.453	51.726

Table 3 – The design values of reinforced concrete strength f_{M1} for bending elements with a circular cross-section (f_{yd} = 375 MPa, A400C), MPa

Concrete class	Reinforcement ratio ρ_f						
	0.05	0.5	1	2	3	4	5
C8/10	0.868	6.852	12.716	23.338	33.651	43.931	54.200
C12/15	0.887	7.161	13.138	24.278	34.619	44.888	55.128
C16/20	0.901	7.473	13.583	25.232	35.634	45.873	56.055
C20/25	0.909	7.643	13.963	25.738	36.480	46.678	56.787
C25/30	0.915	7.769	14.251	26.141	37.089	47.248	57.285
C30/35	0.919	7.877	14.503	26.508	37.592	47.709	57.672
C32/40	0.922	7.969	14.720	26.842	38.033	48.126	58.034
C35/45	0.925	8.061	14.932	27.184	38.420	48.486	58.332
C40/50	0.927	8.122	15.073	27.426	38.685	48.745	58.559
C45/55	0.928	8.171	15.181	27.621	38.856	48.909	58.696
C50/60	0.930	8.218	15.281	27.814	39.015	49.071	58.839

Conclusions. The application of idea of the design strength of reinforced concrete allows to obtain simple calculations of bearing capacity of reinforced concrete elements. The proposed method of calculating the strength of reinforced concrete bending elements with rectangular and circular profiles by applying the design characteristics of strength of reinforced concrete can be used in engineering practice.

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   ISBN 978-617-601-125-5.

Received 22.11.2016

UDC 624.012.4.071:539.4

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ULTIMATE ELONGATION OF CONCRETE

It is shown the results of experimental studies of the deformation and concrete fracture process under the axial tension and under the conditions of non-homogeneous state of stress when changing the deformation gradient over the cross section. The test samples were loaded by the short duration load to the point of structural and with load alleviation under the stresses close to destruction. The basic influencing factors on the ultimate elongation of studied types of concrete are found out and the analytical dependences for their description are proposed. The experiments have shown that under the non-uniform stress state the ultimate elongation is not constant; it varies within a wide range and depends on the critical strain gradient. For solving applied problems, the suitable, for practical use, universal form of connection load – deformation was offered, which truly reflects the unity of the relation between stress and deformation of concrete, which is in the condition of homogeneous and non-homogeneous stress states.

Keywords*: stress-strain diagram of cracked concrete, ultimate elongation of concrete, axial tension, the deformation gradient.*

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ГРАНИЧНІ ДЕФОРМАЦІЇ РОЗТЯГНУТОГО БЕТОНУ

Викладено результати експериментальних досліджень закономірностей деформування і процесу руйнування бетону при осьовому розтягу і при наявності градієнта деформацій по перерізу. Дослідні зразки завантажувалися короткочасним навантаженням до руйнування і з розвантаженням при напруженнях, близьких до руйнування. Визначені фактори,що впливають на граничні деформації розтягнутого бетону різних видів та запропоновано аналітичні рівняння для їх опису. Дослідами встановлено, що при неоднорідному напруженому стані граничні деформації не постійні, а змінюються в широкому діапазоні та залежать від градієнта деформацій. Для вирішення прикладних задач запропоновано універсальну форма зв'язку «навантаження – деформації», яка достовірно відображає єдність зв'язку між напруженнями та деформаціями бетону що знаходиться в умовах однорідного та неоднорідного напружених станів.

Ключові слова: *діаграма напруження – деформації розтягнутого бетону, гранична деформація розтягнутого бетону, осьовий розтяг, градієнт деформацій.*

Introduction. Planning of concrete and reinforced-concrete constructions is carried out on the basis of data of materials' durability and udeformations, which were got by the test of samples, as a rule, under the single-axis loading in laboratory conditions that are only the models of the real elements of constructions. Thus, the law of geometrical similarity was reckoned in, supposing the complete identity of samples' properties and the process of concrete deformation and destruction. However, the mechanism of concrete deformation in the conditions of the inhomogeneous high-pressure state is not identical to the axial tension and does not allow credibly to estimate deformations and durability. Therefore the experimental researches of the influence of different significant factors on durability and ultimate deformations of the cracked concrete have practical value.

Analysis of the last researches and publications. Operating rules [1], during calculation of durability of concrete constructions' standard cross-sections and crack strength of reinforced-concrete constructions, set the limit state, coming from the presentations, and dopted from the strength theory of elastic and inelastic materials. For the estimation of the stress and stress state of cross-sections, including the uniaxiality fabric hypothesis (the diagram $\sigma_{ct} - \varepsilon_{ct}$ under the axial tension is identical to the connection for the conditions of the inhomogeneous high-pressure state) and the supposition that the limit state is reached upon condition the extreme fibers achieve the ultimate strain of tension $\varepsilon_{ct2} = -2f_{ct} / E_{cto}$ Thus, it is considered that ultimate tensility is invariable.

Experimental data, given in literature [2 – 4], show that under the non-homogeneous tension the ratios $\varepsilon_{ct2} / \varepsilon_{ct1}$ and $6M_u / bh^2 f_{ct}$ (for the beams of rectangular cross-section) are non-constant, change in a wide range and depend, mainly, on the gradient of ultimate strain.

There isn't the very opinion at explaining of physical nature of variety of these sizes, the researchers offer different hypotheses.

It is supposed that the effective mechanism of stress grading acts in tension sides of the concrete beams, allowing the plastic flow to develop deep into the cross-sections. Such stress redistribution, maybe, takes place due to the nonlinear creep of concrete. However, direct experiments do not confirm this suspicion: nonlinearity of concrete creep under tension is rather weak, and elastic strains of the non-homogeneous cracked concrete are higher, than under axial tension. There is also the hypothesis about the possibility of concrete behavior in the «cracked» state and the statistical theory of concrete brittle strength. They describe the stress-strain state of beam's normal cross-sections in different ways, and, accordingly, offer the different computational schemes of ultimate states [2, 4, 6]. Analogical principle is characteristic and for the conditions of non-homogeneous pressure [5, 8 – 10].

All this information about the specific of deformation of heterogeneously cracked concrete and heterogeneously compressed concrete, point to iniquity of some hypotheses adopted from the theory of elastic and inelastic materials for the estimation of its stress and strain state.

Selection of problematical parts of general issue and setting the task of researches. The requirement of reliability at designing of concrete and reinforced concrete constructions actualizes caring out the special experimental researches according to this issue with the purpose of studying such questions:

– to investigate experimentally numerical influence of significant factors on common denominator of concrete resistance under non-homogeneous tension;

– to estimate ultimate tensility ε_{ct2} under bending in comparison with maximum axial tensility ε_{ct1} and to develop recommendations on the determination ε_{ct2} taking into account influencing factors;

– to investigate the features of destructions' development (strength and contractible) in the cracked concrete;

– to estimate the actual deformed state of normal cross-sections of concrete beam and construct a calculation model.

Basic material and results. For the quantitative estimation of influence of different factors (macrostructure and viscoelastic properties of concrete, type of the tense state, gradient of ultimate deformations, loading conditions) on ultimate tensility of concrete, the special experimental researches are carried out.

For the design of macrostructure and viscoelastic properties the model fine-grained concrete (MC), normal heavy concrete (HC) and light ceramsite concrete (LC) of wide range of compression breaking strengths are used from 10 to 80 MPa. Every class of investigational types of concretes contained from 3 to 24 samples.

The preproduction test was carried out under axial tension and in the conditions of the non-homogeneous state of stress. The wide range of gradients of ultimate deformations was designed by the deflection test of similar beams of cross-sections h = (1 – 60) cm, siding b = (1/3 and 3)h and bearing distance l = 6 h. Nonrigid behaviors of concrete under axial tension are determined at the test of samples-cylinders with the diameter 12 cm, length 40 cm. The samples (cylinders and beams) were tested in the short run by the step load till the destruction with the aggregate exposure on each state. For development of deformation of simple bending the beams were loaded by two concentrated forces in three spans, the cylinders – with the use of the device allowing to except centering error in the process of load application. For the selection of completely recoverable deformations of concrete expansion, the part of samples was off-loaded after every stage of loading. Deformations were being measured on the long bases and locally.

The tests verified the idea about the mechanism of concrete deformation in the conditions of axial uniform extension as about the process of starting and development of dispersed cracks that unite in the cross-section of destruction in a critical crack with the load growth. In the cracked concrete the tiny cracks appear long before the destruction and do not cause its bearing-capacity failure. There are the micro-discontinuities in concrete under axial tension with the loads $N_{crc} = (0.3 - 0.8)N_u$ in the tests. Body shrinkage-related stresses are able to break down soundness of coating surface or in isolated point reach the value similar to the limiting values before load application. In this case the behavior of limit equilibrium of ties is arrived even under the light external load, and there are micro-fractures of local character. Thus, cracked, under axis stress, concrete works with cracks and deforms on the length of sample very homogeneously ensuing incipient micro-fractures. The character of concrete's local deformations along the length of the centrally cracked sample with the growth of the load is shown on fig. 1.

The deformations, measured by devices with a different base, are different and incomparable. Devices, measuring local deformations, where cracks are formed in the base, show deformations that outnumber the values by several timeds according to the devices on a large base. In the next cross-sections with a progressive crack the local deformations can have even an opposite sign (reduction deformations), that is the result of initial surface-tension (for example, devices 11 and 15, fig. 1).

Inhomogeneity of the structure and micro-crack forrmation (shrinkable and power destructions) in the cracked concrete is responsible for the large nonuniformity of deformations both on length and on cross-section of samples, and, as a result, stochastic changeability of measurable deformations of concrete. In this case such concept, as «maximum tensility» usually used in the theory of concrete strength, is conditional. In the article the category «maximum tensility» of concrete under the axial tension ε_{ct1} is interpreted as peak deformations (the end of rising branch), corresponding to the concrete tension f_{ct} and the deformations, measured by devices on a large base (smoothed deformations).

For all tested types of concrete the diagrams $\sigma_{ct} - \varepsilon_{ct}$ are plotted under axial tension and the association between the parameters of concrete ε_{ct} and f_{ct} is set (fig. 2).

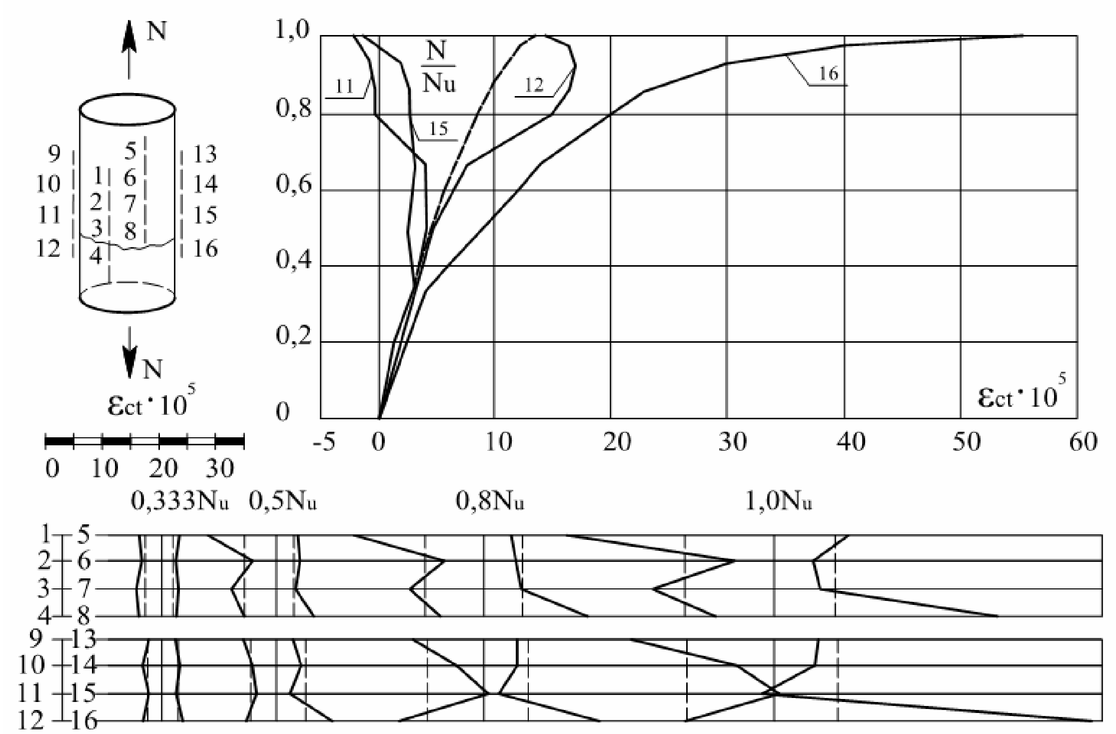

Figure 1 – Strain distribution along the length of the centrally cracked sample
—— local deformations; ------ normal deformations.

Deformation of concretes under axial tension with the load growth has nonlinear character. Nonlinearity is characteristic not only for total deformations but also for immediate elastic ones. It decreases with the growth of concrete class and less expressed for MC and LC, than for HC. The size of pseudo-plastic deformations, measured by devices on a large base, is insignificant, and the destruction of samples, regardless of type and class of concrete, takes place brittle.

For the solving the applied tasks, the analytical dependences, comfortable for practical use, are offered, describing the diagrams $\sigma_{ct} - \varepsilon_{ct}$ under axial tension definitely:

$$\sigma_{ct} = f_{ct}\left[1-\left(1-\varepsilon_{ct}/\varepsilon_{ct1}\right)^{1/n}\right]; \tag{1}$$

$$\varepsilon_{ct} = \varepsilon_{ct1}\left[1-\left(1-\sigma_{ct}/f_{ct}\right)^{n}\right]; \tag{2}$$

$$\frac{N}{N_u} = 1-(1-\frac{\varepsilon_{ct}}{\varepsilon_{ct1}})^{1/n}. \tag{3}$$

The physical meaning of n expands: $n = f_{ct}/\varepsilon_{ct1}E_{cto} = \nu_{ctu}$, where ν_{ctu} – is limiting factor of concrete elasticity under axial tension.

Equations (1) and (2) were got at the solving of differential equation (4):

$$E_{ct} = \frac{d\sigma_{ct}}{d\varepsilon_{ct}} = E_{cto}(1-\frac{\sigma_{ct}}{f_{ct}})^{1-n}. \tag{4}$$

Equation (4) of the module of deformations under short standard tension satisfies to the basic phenomenological requirements to the arc

$$\sigma_{ct} - \varepsilon_{ct} \ (\sigma_{ct} = 0 \ - \ \frac{d\sigma_{ct}}{d\varepsilon_{ct}} = E_{cto} \text{ and } \sigma_{ct} = f_{ct} \ - \ \frac{d\sigma_{ct}}{d\varepsilon_{ct}} = 0 \ ).$$

Formula (4) corresponds to concrete downloading with $V_{\sigma_{ct}} = const$ so it does not describe the descending arm of diagram $\sigma_{ct} - \varepsilon_{ct}$ ($V_{\varepsilon_{ct}} = const$).

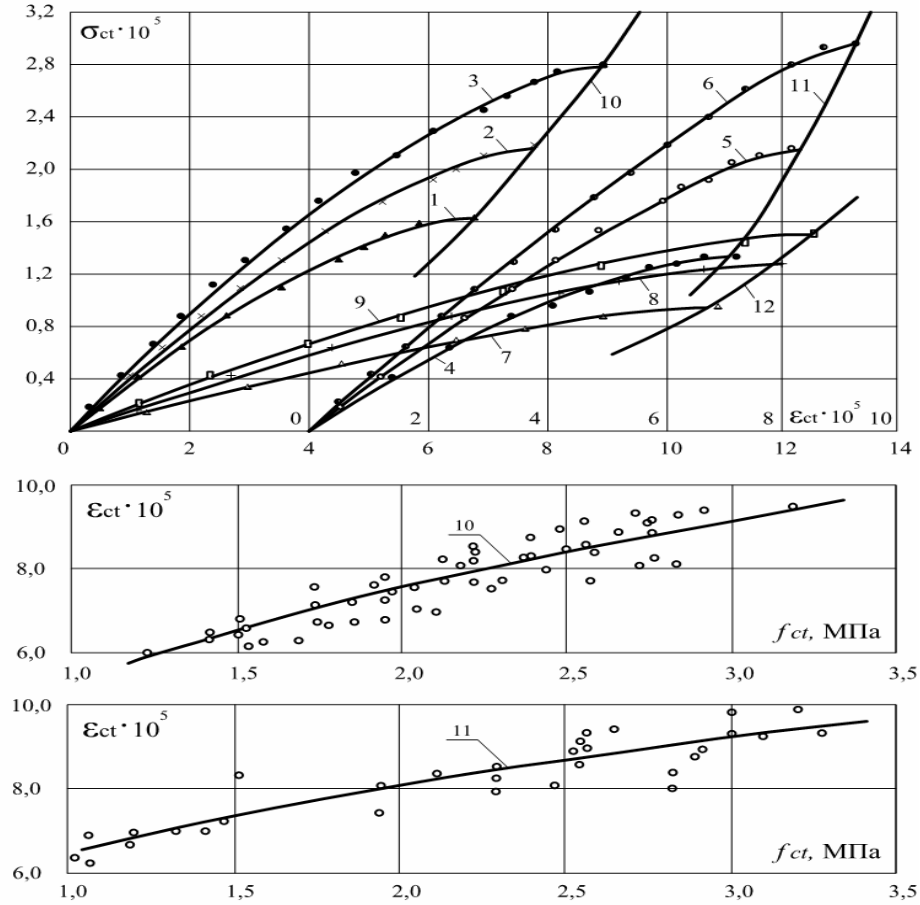

Figure 2 – Diagrams $\sigma_{ct} - \varepsilon_{ct}$ under axial tension (*a*) and relation of maximum tensility ε_{ct1} from concrete strength under axial tension f_{ct} (*b*)

1, 2, 3 are MC, accordingly, strength $f_{cm.cube10*10}$ = 23,7; 41,8 and 59,5 MPa;

4, 5, 6 – HC, $f_{cm.cube10*10}$ = 28,0; 47,5, 76,5 MPa;

7, 8, 9 – LC, $f_{cm.cube10*10}$ = 11,1; 19,4, 29,9 MPa;

10, 11, 12 – relation $\varepsilon_{ct1}(f_{ct})$ for MC, HC, LC.

According to test results (fig. 2b) ultimate deformation under short axial tension for investigational concretes changes in a range $\varepsilon_{ct} = (6{,}8 \div 12{,}5)10^{-5}$. Variability of elements ε_{ct1} is high; however, there is certain conformity in its arithmetic mean values, depending on a type and size of aggregate, cement content and class of concrete. Lightweight concretes have more deformability, than high weight concrete. MC and HC have, practically, identical valuesε_{ct1}. However, there is tendency to increase ε_{ct1} with decreasing of aggregate size. They are more in MC than in HC. The researches show that total ultimate deformations of tension for all tested concretes increase with increasing the strength. Between these characteristics there is correlation relationship that is approximated by equation:

$$\varepsilon_{ct1} = A f_{ct}^{\alpha}\ , \tag{5}$$

where f_{ct} in MPa; A and α – parameters of this concrete type (for HC – A = $5{,}35 \cdot 10^{-5}$, α = 1/2; MC – A = $6{,}41 \cdot 10^{-5}$, α = 1/3; LC – A = $10{,}91 \cdot 10^{-5}$, α = 1/3).

It should be noted that restrictions $\varepsilon_{ct1}(f_{ct1})$ have private character, as the properties and component type of concrete mixture, storage instructions and age of samples, technological factors, conditions of deformation of standards etc. influence on a valueε_{ct1}.

Inhomogeneous tension brings substantial change in behavior of concrete deformation.

The results of beams' tests show that their deformation, as well as under axial tension, is accompanied by progressive development of destructions in the cracked concrete long before its destruction. From the first stages of load application new micro-cracks are formed in a sample with force action with the simultaneous opening of solidification shrinkage cracks. With light load the speed of destruction development is low, it dies out, and does not result momentary drop of bearing strength of concrete beams. With load increasing the micro-cracks gradually develop and, closing up, they form macro-cracks, their accelerated development results to crushing of sample. It is set experimentally, that the width of micro-crack opening in concrete beams before crushing is a considerable size, i.e. the beams in the tension side work with cracks. Local deformations along the length of beam in the area of simple bending, both on the most tensile face and on the depth of section, are very different. The devices, the micro-cracks are formed in their base, show sharp increasing of local deformations of tension («pseudoplastic» deformations by O. Berg or «separated» deformations by M. Holmyanskiy). In nearby cross-sections increasing of deformations discontinued or decreased. The deformations of shortening in the tension side to the moment of destruction sometimes were $\varepsilon_{ct} = (4 \div 6)10^{-5}$.

According to the size and speed of local deformation it is possible to judge what will happen with destruction of beam cross section. The cross-sections, where were a few critical cracks forming along the length of zone of simple bending, but, as a rule, only in one of them the crack was, resulting in destruction in future, progressing under the load (0,7–0,9)M / M_u. Critical cracks divided the zone of simple bending into separate blocks.

The character of local deformations changing is different in the compressive side of the beam. The non-uniformity of local deformations changing along the length of the beam in the compressive side is inconspicuous. It is conditioned, mainly, by of concrete structure heterogeneity. However, under the high load levels there is some growth acceleration of local deformations of shortening, located in those cross-sections, where the critical cracks (moving of zero axis) develop in the tension side.

The development of micro-cracks in concrete with the deformation gradient takes place otherwise, than under the uniaxial state of stress. The cracked concrete can not be examined as a solid uniform body, following only the laws of elastic-plastic deforming [2, 4, 5, 7]. This material is heterogeneous with broken soundness of coating surface and works in the «cracked» state. The theory of equilibrium cracks (Hriffits, Ervin), elaborated for the materials such as concrete, confirms this idea.

The analysis of concrete centerline deformations shows that normal sections are plane with the growth of load, i.e. the tests confirm acceptability Bernoulli hypothesis for normal sections only the case if the deformations in the tension side are measured (averaged) on a sufficiently large base. This behavior is observed under all levels of the fraction loading.

Micro-crack formation in the cracked concrete causes some displacement of neutral layer with the load growth toward the compressive face. Its position is determined according to the normal total centerline deformations of the compressed and tension fibers of beams and specified according to the completely recoverable deformations of fibers, came up under unloading:

$$\xi_u = \frac{x}{h} = \frac{\varepsilon_c}{\varepsilon_{ct2} + \varepsilon_c} \quad . \tag{6}$$

For comparison the relative height of the compressive side ξ_u is also calculated through beam deflections and compressive deformations ε_c, their variableness is small:

$$\xi_u = \frac{\varepsilon_c l^2}{8hf^{\max}} . \qquad (7)$$

The differences in averaging values ξ_u, determined by different methods (6) and (7), under other equal terms, are scant; and the value ξ_u is 0,446 for a fine-grained concrete; 0,407 – ceramsite concrete; 0,425 – heavy-weight concrete.

The test results of geometrically similar beams of experimental concrete types of the wide strength range show that total ultimate deformations of fibre tension under the bend ε_{ct2} are higher, than under the axial tension ε_{ct1} of analogical equally efficient concrete, and substantially depend on the gradient of ultimate deformations *grad(*ε_{ct1}*/h)* (depth of beams). With increasing of beam depth the deformation ε_{ct2} decreases and under high values (h > 40 cm) tends to ε_{ct1}. For the beams of low depth (h < 2 cm) the ultimate deformation ε_{ct2}, more than 4 times ε_{ct1} (fig. 3a, б).

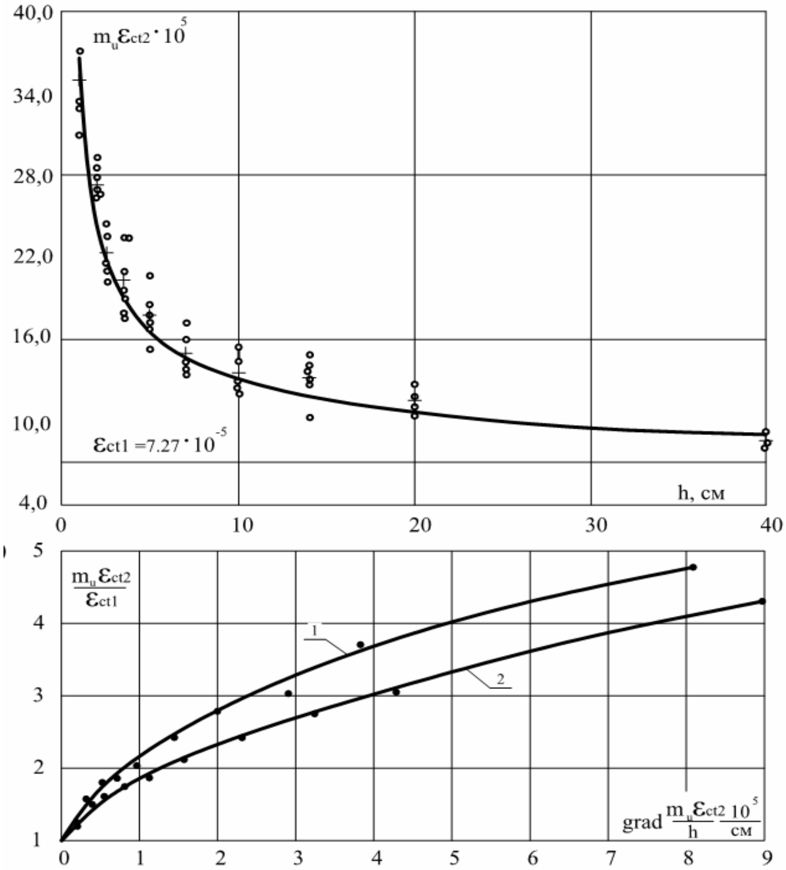

Figure 3 –The relation of ultimate deformations of beams fibre tension under bending ε_{ct2} from the depth of beams (a) and the gradient of ultimate deformations along the cross-section (b).
1, 2 – MC, accordingly, the strength f_{cm} = 23,7; 41,8 MPa.

The increment of ultimate deformations of border fibers of the tension side under bending (considering deformations ε_{yc}) above ultimate deformation under axial tension is related to the depth of beam by hyperbolic dependence:

$$(m_u \varepsilon_{ct2} - \varepsilon_{ct1} + \Delta\varepsilon_{yc}) h^{\alpha} = S = const . \qquad (8)$$

The value of parameter S is determined by the test of standard beam, where ε_{yc} = 0. The depth of standard beam corresponds to the diameter of cylinder (12 cm), according to it ε_{ct1} is determined. In the relation (8) the coefficient $m_u = \xi_u$ / $(1-\xi_u)$ takes into account the limit displacement of neutral axis of beams, and the value $\Delta\varepsilon_a$ – the difference of shrinking deformations of upperbound tension of cylinder and beam of this depth, caused by the inequality of concrete shrinkage. This type of function (8) also allows to estimate the influence of shrinkage-related prestress in the samples of different sizes on ultimate tensility of concretes. For massive elements (h > 40 cm) and concretes with the enlarge cement content the shrinking deformations ε_{yc} tend toε_{ct1}.

It is important to notice that this relation of ultimate deformations under heterogeneous tension from the gradient of deformations also belongs to elastic deformations, got during beam unloading that directly connect with the tensions. This circumstance testifies that the actual tensely-deformed state of the tension side of concrete beam does not rise to calculation, got according to the complete diagram of axial tension taking into account a descending branch. Connection between tensions and deformations under heterogeneous tension is different comparing to that under axial tension. At the same time, the diagrams «load – deformations» of the bending elements are well described by equation (9), analogical to the connection, «load – deformations» (3) of axial tension:

$$\frac{M}{M_u} = 1-(1-\frac{\varepsilon_{ct}}{\varepsilon_{ct2}})^{1/n} \ , \tag{9}$$

where $n = f_{ct} / \varepsilon_{ct1} E_{cto} = \nu_{ctu}$.

The elastic ratio under bending is approximately equal to the elastic ratio under axial tension. Equation (9) estimates the deformations of beams for all levels of loading. Taking into account the succession of hypothesis of plane cross-sections, it allows to define the connection between an external moment and deformation of any fibre of the tension side (fig. 4).

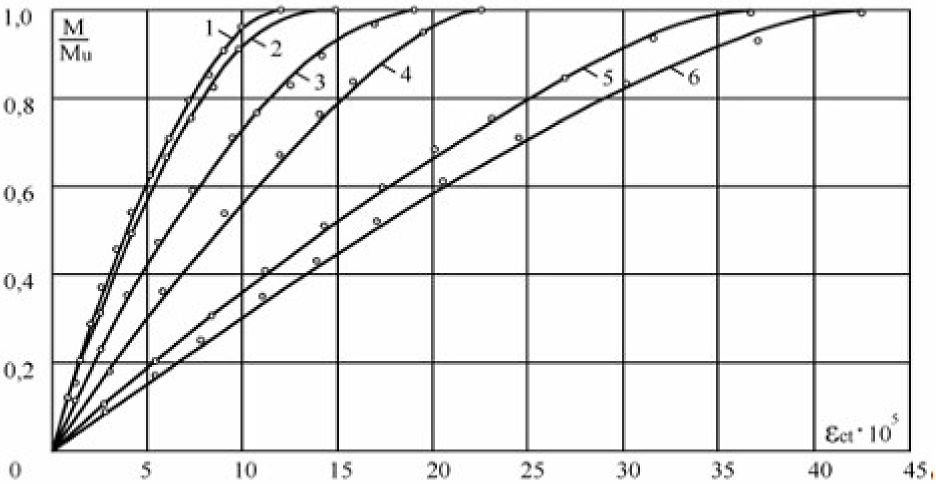

Figure 4 – Diagrams «load – deformations» of the tension fibers of the beams of different depth

1, 2, 3, 4, 5, 6 – accordingly, the beams from MC with the strength f_{cm} = 23,7 MPa

Depth of sections 40; 20; 7; 5; 1,9; 1,0 sm

Conclusions. Experimental data show that deformation of concrete beams is accompanied by development of micro- and macro-cracks in the cracked concrete (beams work in the «cracked state»). Shrinkable and power destructions, creating in the concrete, cause the very difficult field of deformations in the tension side of beam and allow to suppose that the mechanism of beam's fiber deformation is not identical to the axial ladening (the hypothesis of uniaxiality fiber works is not acceptable). Total ultimate deformations of the concrete (including elastic ones) under non-homogeneous tension are not constant. They depend on the gradient of ultimate deformations and are changed in a wide range ε_{ct2} / ε_{ct1} = 1÷5. Such conformities give reason to take equivalent cross-section with the depth of crack development $t = h$ (1–2ξ_u) in the tension side, the three-cornered form of deformation epure and border ε_{ct2}, depending on the depth of cross-section of beams as calculated deformed cross-sectional beams in the ultimate state. Bernoulli hypothesis is acceptable to this cross-section and the position of neutral axis is known.

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Received 15.02.2017

UDC 624.01

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AN EFFECTIVE STRUCTURE FOR STRENGTHENING REINFORCED CONCRETE BEAMS

The study presents a new system of strengthening reinforced concrete beams. The system includes strengthening bars acting on the beam through other strained flexible elements. It operates in a conditionally paradoxical way. It does not compress but strains directly the upper beam zone compressed under the load. Additional flexible elements are made in the form of rods placed obliquely. The study provides the results of determining the most efficient angle of inclination of transverse bars for the external system of strengthening. The paper calculates an efficient angle of inclination of transverse bars that are to unload the beam by the moment reverse in sign of the moment from external load. It also analyzes the results obtained after examining the strengthening by means of a longitudinal-transverse external system and building the diagrams of moments.

***Keywords:** bars, cross-section, stress, strain, strength, rigidity, span.*

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ЕФЕКТИВНА СХЕМА ПІДСИЛЕННЯ ЗАЛІЗОБЕТОННИХ БАЛОК

Запропоновано нову конструктивну систему підсилення залізобетонних балок, яка включає затяжку, що діє на балку через інші гнучкі розтягнуті елементи; вона працює умовно парадоксально – не стискає, а розтягує безпосередньо верхню стиснуту під навантаженням зону балки. Додаткові гнучкі елементи виконано у вигляді похило розташованих срижнів. Наведено результати пошуку найбільш ефективного кута нахилу поперечних стрижнів для зовнішньої системи підсилення балки. Визначено раціональний кут нахилу поперечних стрижнів, які мають розвантажувати балку моментом, зворотним по знаку до моменту від зовнішнього навантаження. Проаналізовано результати, отримані після дослідження підсилення поздовжньо-поперечною зовнішньою системою та побудовано епюри моментів.

***Ключові слова:** затяжка, переріз, напруження, деформації, міцність, жорсткість, проліт.*

Recent sources of rresearch and publications analysis. Strengthening of reinforced concrete elements such as beams and slabs is widely used in construction practice. In recent years, reinforcement application has increased due to a significant increase in the share of reconstruction in construction. This is the result of changes in stress, physical deterioration of existing buildings because of their poor maintenance and other factors, which leads to bearing capacity loss; thus structures need to be strengthened to ensure sufficient performance under normal maintenance and reliability. [1, 2].

Different methods of strengthening bending concrete elements were developed [3-14]. The findings of national and foreign authors, such as E.M. Babich, A.M. Bambura, G.I. Berdichevsky, V.A. Roach, A.A. Gvozdev, A.B. Golyshev, E.A. Hrynevych, F.S. Zamaliyev, M.Y. Izbash, V.G. Kvasha, F.E. Klimenko, F. Leonhardt, E.A. Luchkovskyy V.V. Mikhailov, N.M. Onufriev, S.F. Pichugin, E.G. Ratts, L.I. Storozhenko, L.N. Fomytsya, E. Freyssinet, Jiang De, A.L. Shahin, Richard W. Plavidal, Thomas Keller and others make the scientific, design and technological basis for using reinforced structures in construction. The variety of mentioned methods can be divided into 3 groups: strengthening with a change in the constructive and design scheme of a reinforced concrete member; strengthening without changing the work pattern of structures by building up their cross-sections with additional external reinforcement; strengthening by prestressing beams with horizontal ties.

There can be three types of tie bars: horizontal, sprengel and combined ones. When applying prestressed tie bars, the strengthened elements change their original constructive scheme because they turn into a combined system. Due to this fact, conventional bending elements are compressed noncentrally, and with their supports additional bending moments are created, which in turn influence the initial span moments. In this case, in all the variants of reinforcing beams with tie bars there are still significant reserves of increasing carrying capacity.

Specifying aspects of the problem unsolved before. After considering many schemes of reinforcement it can be concluded that their disadvantage is the inability to effectively unload the compressed zone of concrete beam effectively, which greatly affects rigidity and bearing capacity.

Setting objectives. The aim is to study the most efficient inclination angle for placing cross bars of the reinforcement system. The angle is to meet the following requirement: maximum unloading of the beam by the moment reverse in sign to the moment of external load.

Research objectives:

– to conduct research setting different angles for the inclined transverse reinforcing bars;
– to identify relative and absolute elongation, tensile value in the bars;
– to analyze the results obtained after examining the reinforcement with a longitudinal-transverse external system, and build diagrams of moments, to choose the most efficient inclination angle;
– to evaluate the results of experimental studies;
– to determine the effectiveness of strengthening reinforced concrete beams with a longitudinal-transverse external system of reinforcing bars.

Basic material and results. For experimental studies three series of reinforced samples of reinforced concrete beams were conducted. Reinforced concrete beams were 2.1 m long, had a cross-section of 100×200 mm and were made of concrete of class C45/55. They were reinforced with frames made of rebar of nominal diameter Ø 6 A240S. The БПП beams series were strengthened according to the patent. [15] Three options for the location of inclined bars at different angles such as 30°, 45° and 60° were considered. The external reinforcement system is presented in Fig. 1. It is proposed a design solution of a stressed-regulated beam containing a reinforced concrete body and tie, mounted at the ends onto the beam interacting in the middle with the stressing element contacting the lower part of the beam, and lateral external reinforcement interacting at the ends of the beam with its upper and lower parts, and interacting with the tie in its middle part. Transverse reinforcement bars are strained, flexible and placed symmetrically in the area near the supports of the beam with an inclination to the longitudinal axis of the beam.

Figure 1 – Scheme of strengthening БПП-III-1 beam with external longitudinal and transverse rebar

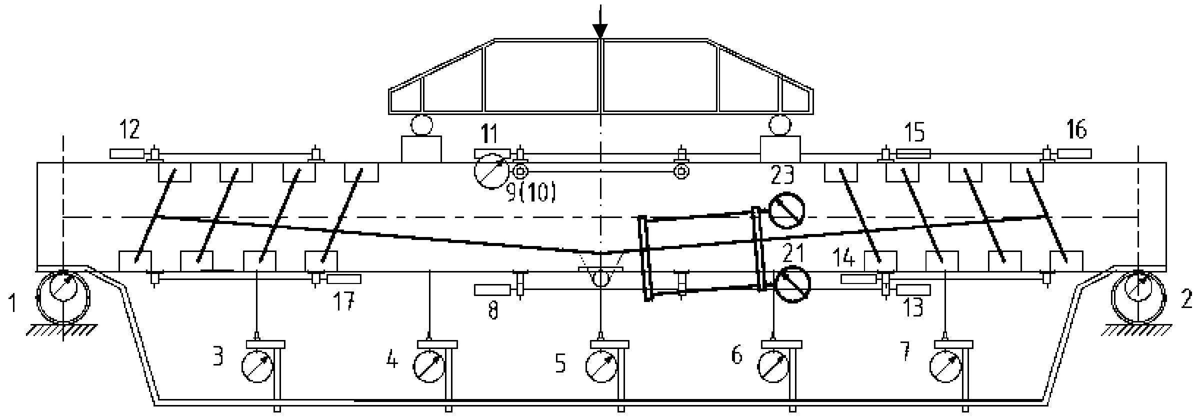
Inclined strands are composed of identical reinforcing bars that are connected with the tie by welding. When external force is applied, inclined bars deform and bend. Fig. 2 shows the initial position of the inclined bar at an angle of 30 °.

The work of the strengthening system was examined at the level of load η =0,9. The force in the tie was fixed in all experiments.

Figure 2 – Starting position of the inclined bar at an angle of 30°, before load application.

When placing the bars at an angle of 30 ° the design scheme the following form is obtained, shown in Fig. 4.

To build a diagram of deformations, it is used the solution proposed by Timoshenko S.P., James M. Gere. [16]. The diagram shows the deformations schematically in Fig. 5.



Figгку 3 – Scheme of testing a strengthened beam of БПП-III series:
1, 2 – force meter; 3-7 – flexometer with a value of a division of 0.01 mm
to measure beam deflection;
8-17 – indicators with a value of a division of 0.001 mm
to measure the deformation of concrete

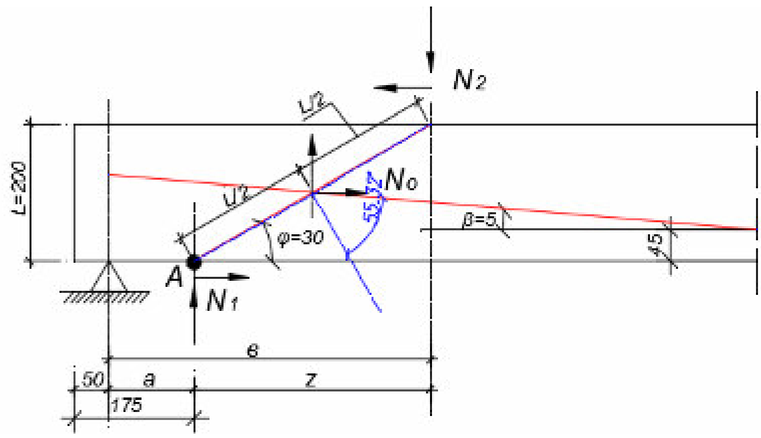


Figure 4 – Scheme of bar placement at an angle of 30 °

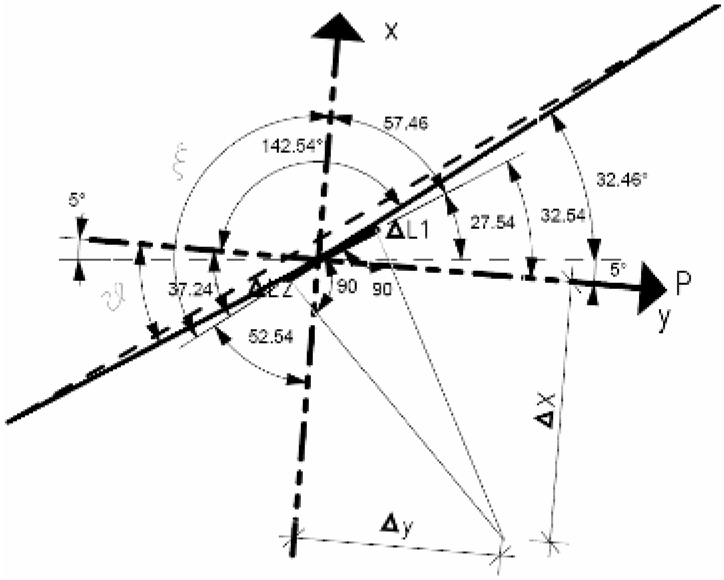


Figure 5 – Estimated diagram to determine the elongation of bars

After applying the load onto the deviation angle of the inclined bar was 1.54 °, and the vertical deformation was 4.88 mm; the data are presented at an angle of inclination of 30 ° for transverse bars.

The forces acting in the upper and lower parts of the cross rod can be expressed by the theorem of sines:

$$N_1 = \frac{P\sin\ \xi}{\sin\left(180 - (\vartheta + \xi)\right)}\ , \tag{1}$$

$$N_2 = \frac{P\sin\vartheta}{\sin\left(180 - (\vartheta + \xi)\right)}\ , \tag{2}$$

where: $\gamma = 180 - (\vartheta + \xi)$,

Absolute elongation of bars is:

$$\Delta L_1 = \frac{N_1 \cdot L_1}{E \cdot A}\ , \tag{3}$$

$$\Delta L_2 = \frac{N_2 \cdot L_2}{E \cdot A}\ . \tag{4}$$

To determine the displacement of the point of maximum deformation of the inclined transverse cross bar, it is obtained the crossing of radii $L_1 + \Delta L_1$ and $L_2 + \Delta L_2$. Due to the small value of deformations, it can be added the value of every elongation and drawn a perpendicular from the point obtained to the direction of the bar. The diagram of the elongation of bars is shown in Figure 5. The diagram shows that ΔL_1 is the algebraic sum of the projections of horizontal Δ_x and vertical sΔ_y displacement of the specified point in the direction of the 1st bar, but ΔL_2 is equal to the sum of the projections Δ_x and Δ_y in the direction of the 2nd bar. The position of the maximum displacement is determined by the system of equations:

$$\begin{cases} \Delta L_1 = -\Delta_x cos\theta + \Delta_y sin\theta \\ \Delta L_2 = \Delta_x cos\varphi - \Delta_y sin\varphi \end{cases} . \tag{5}$$

After calculating the forces in each bar at different angles of inclination, it is obtained diagrams of moments of external loads and from the strengthening system for getting maximum efficiency and the most efficient angle of inclination. For beams with a 30 ° angle of inclination of transverse bars, the angles were θ = 57.46 °; φ = 52.54 ° under the load η = 0.9.

Research results. Fig. 5, 6, 7 show the diagrams of moments at different angles of inclination under the level of load η = 0.9 on the strengthened beam.

Stress-strain state of each cross-section of reinforced beams can be determined under the equilibrium of the bending moment and longitudinal forces in the cross-section:

$$\sum M = 0 \qquad M_{зов} = \int_A \sigma_c h\, d\, A_c + \sum_{i=1}^{n} \sigma_s A_s h_i + M_{nX} + M_{nY}\ , \tag{6}$$

$$\sum N = 0 \qquad N_{nX} = \int_A \sigma_c d\, A_c + \sum_{i=1}^{n} \sigma_s A_s\ , \tag{7}$$

where σ_c – compressive stress in concrete;

σ_s – stress strain in reinforcement;

A_c – cross-section area;

A_s – cross-section area of steel;

M_{nX}, M_{nY} – moments under strengthening.

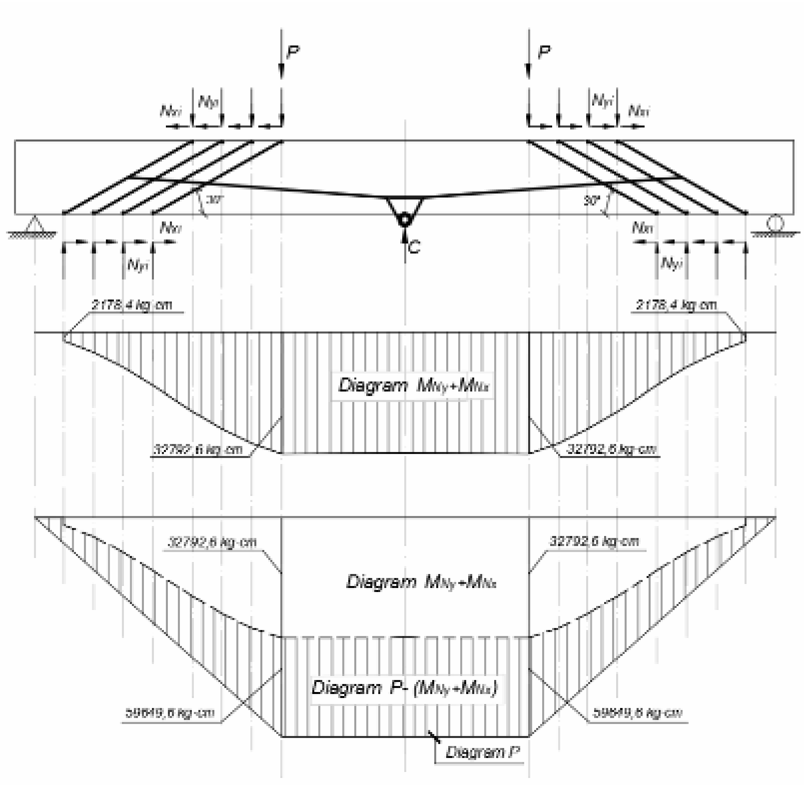


Figure 6 – Diagrams of moments considering strengthening at a 30° angle of inclination of transverse bars

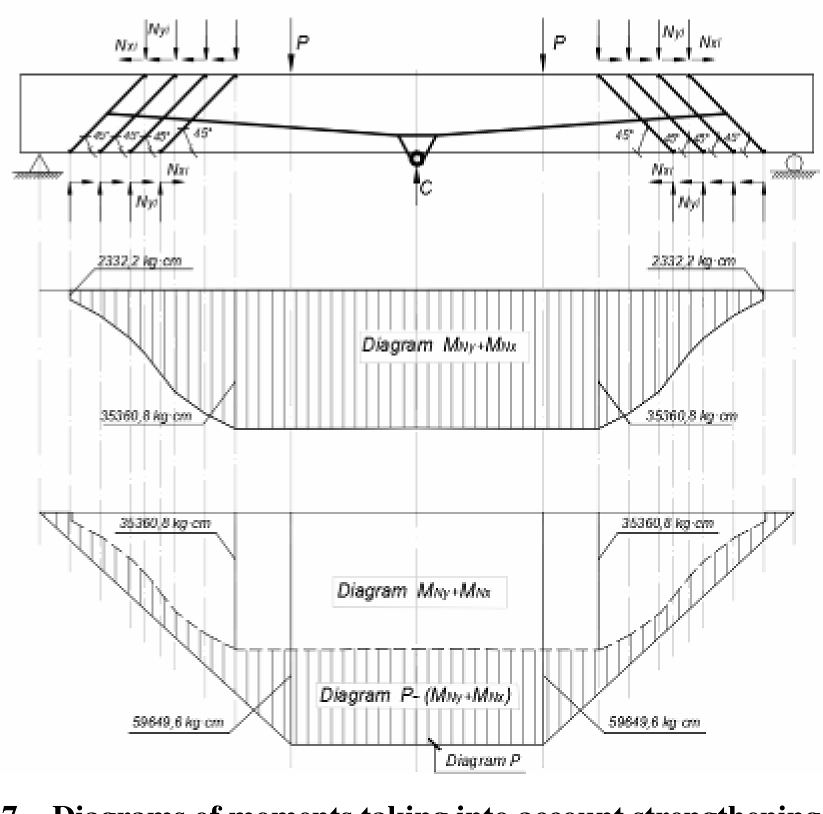


Figure 7 – Diagrams of moments taking into account strengthening at a 45° angle of inclination of transverse bars

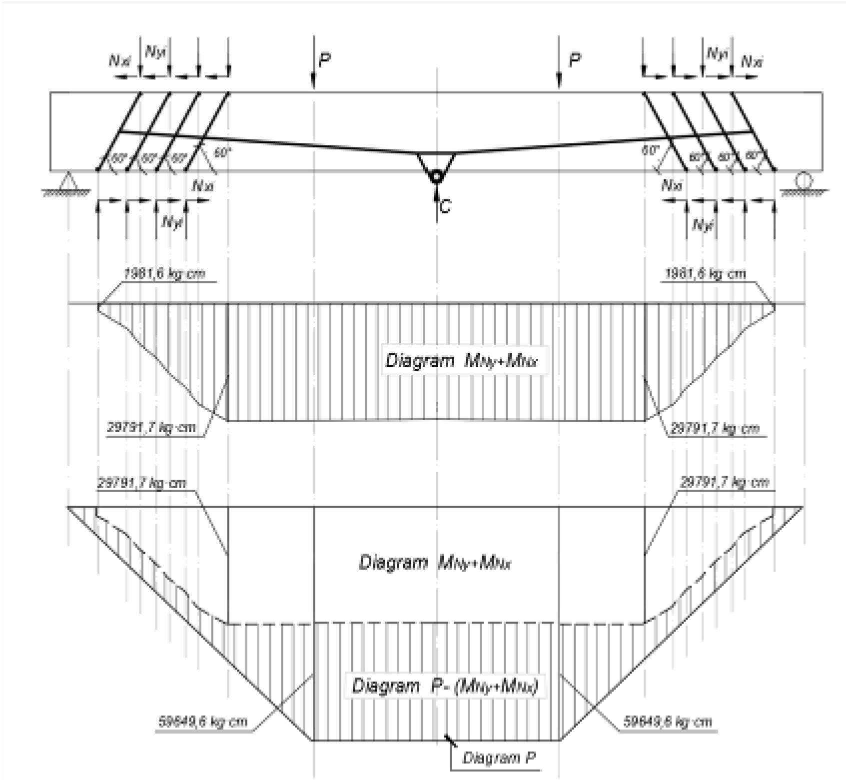


Figure 8 – Diagrams of moments considering strengthening at a 60° angle of inclination of transverse bars

Depending on the angle of inclination of transverse bars near the supports of the beam, the values of the moment were obtained. The most efficient angle for placing the inclined bars was 45° under the level of load η =0.9. The efficiency of strengthening is 59,28%, which is by 4,31% more than for the angle of 30 ° and by 9,34% higher than for the angle of 60°. The main research results are shown in the table.

Table of results

№	Inclination angle of the bars of the strengthening system, α	$M_{Nx}+ M_{Ny}$, kg·cm	M_{Σ}, kg·cm	$\frac{M_p}{M_{Nx}+M_{Ny}}$	Effect, %
1	30°	32792,8	26856,8	1,81	54,97
2	45°	35360,72	24288,88	1,68	59,28
3	60°	29791,6	29858	2,00	49,94

Conclusions. The findings proves that the effect of the application of the developed system of strengthening reinforced concrete beams is within 49-60%. The most efficient angle of inclination of transverse bars of the external reinforcement is 45°.

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Received 06.10.2016

UDC 621.432.3

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STRESS-STRAIN STATE OF REINFORCED CONCRETE BEAMS STRENGTHENED WITH A NEW EXTERNAL STEEL STRUCTURE

The study presents a new structure for strengthening of one-span reinforced concrete beams in rectangular cross-section using external steel bars. The specific feature of the proposed strengthening is the unloading of the compressed upper zone of a beam with simultaneous compression of its lower stretched zone. The article considers some variants of making the strengthening structure with rigid and flexible reinforcement elements for faster tension of external bars, and the variant including only flexible elements. It provides a design scheme and method for such reinforced beams. The study provides experimental research data on the series of beams with different parameters of the strengthening structure in the form of «bending moment – deflection» and «bending moment - deformation of concrete» dependencies.

Keywords: *strengthening, reinforced concrete beam, external steel bars, stress-strain state, deflection, deformation, bending moment.*

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НАПРУЖЕНО-ДЕФОРМОВАНИЙ СТАН ЗАЛІЗОБЕТОННИХ БАЛОК, ПІДСИЛЕНИХ НОВОЮ ЗОВНІШНЬОЮ СТАЛЕВОЮ КОНСТРУКЦІЄЮ

Представлено нову конструкцію підсилення однопрольотних залізобетонних балок прямокутного перерізу за допомогою зовнішньої сталевої арматури. З'ясовано, що особливістю запропонованого підсилення є розвантаження верхньої стиснутої зони балки при одночасному стисканні її нижньої розтягнутої зони. Розглянуто варіанти виконання конструкції підсилення балок з жорсткими й гнучкими елементами підсилення для прискореного натягу зовнішньої арматури і варіант лише з гнучкими елементами. Запропоновано розрахункову схему для такої підсиленої балки та методику розрахунку. Наведено експериментальні дані дослідження серій балок з різними параметрами конструкції підсилення, що наведені у вигляді залежностей «згинальний момент – прогин» і «згинальний момент – деформації бетону».

Ключові слова: *підсилення, залізобетонна балка, зовнішня сталева арматура, напружено-деформований стан, прогин, деформації, згинальний момент.*

Introduction. In recent years, the majority of long-maintained construction objects of the Ukrainian building complex require upgrading, reconstruction and strengthening of existing structures. Concrete beam elements are among the most used in such buildings, and their carrying capacity should be restored or increased.

Analysis of the latest research findings and publications. In order to increase the carrying capacity and reduce deformability of bent elements, the cross-section of a structure is increased, using ties or sprengels in the strained zone, arranging duplicating elements. There are many ways of strengthening bent elements, reflected in the works of outstanding Ukrainian and foreign scientists [3-9, 12-15]. The most effective and conventionally used ways of strengthening include ties and especially sprengels.

Specifying the unsolved aspects of the general problem. Thus, the application of a sprengel-type structure is accompanied by additional compressive forces that load additionally the beam's compressed zone, thereby accelerating the time of its failure. Therefore, the question of simultaneous unloading of both the stressed and compressed zones is currently unresolved. In addition, the evaluation of the work of the external strengthening system is quite a challenge because beam-strengthening system is statically indeterminated and requires additional experimental and theoretical researches.

Problem statement. The aim is to propose, make and research an effective structure of strengthening reinforced concrete beams that regulate forces in the beam element and compensate the negative impact of external load, while using the strength properties of concrete and steel.

Basic material and results. A new design solution for the system of strengthening one-span reinforced concrete beams performed by external steel bars and protected by patent [10] is shown in Fig.1.

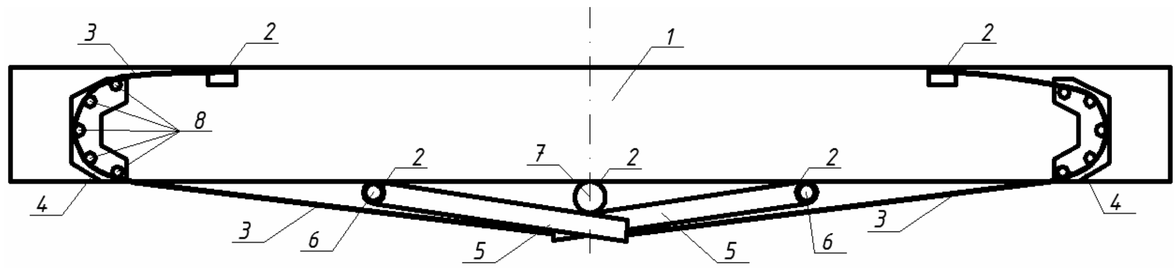

Figure 1 – Scheme of the beam with the proposed regulated structure of strengthening with external bars with levers, БП-I - БП-VI series:
1 – reinforced concrete beam; 2 – steel plates fixed on the surface of the beam;
3 – external bars; 4 – guiding element; 5 – stressing structure in the form of two levers;
6 – hinge; 7 – roller; 8 – guiding rollers for elements 4

The specific feature of this structure is the ability to unload the compressed zone of the beam, in contrast to conventional sprengel ties that have an additional load on it. In addition, the system can work effectively under symmetrical load.

In order to test the proposed structure of strengthening reinforced concrete beams, there has been developed an experimental research program that included testing of samples of concrete cubes, prisms, steel bars. The measurements of beam samples were the following: length – 2100 mm, cross-section – 100×200 mm. Concrete class was the same for all experimental beams and was accepted C35/45.

The study accepted the reinforcement of beams with spatial frames of bars with a diameter of 6.5 mm. The working bars were of class A-240C. Reinforcement bars of the external strengthening system were of steel wire Ø5 mm for all samples of experimental beams. Seven series of reinforced concrete beams strengthened with external bars with different parameters of the strengthening system, and a series of conventional reinforced concrete beams were experimentally investigated.

The following beam marking was accepted: the first letters show whether the beam is strengthened (БП) or not (БО), the second number indicates the number of series, the third - the number of beams in each series. Geometric characteristics and number of external wires of experimental beams are given in Table 1.

Table 1 – Geometric characteristics of strengthened beams

№	Beam series	Roller diameter ***d***, mm	The number of external wires, ***N***	Characteristics of the guiding element **c**, mm	Type of strengthenin g	***k***, mm
1	БО	-	-	-	-	-
2	БП-I	35	1 Ø5 mm	100	Rigid levers	620
3	БП-II	55	1 Ø5 mm	70		620
4	БП-III	55	1 Ø5 mm	40		620
5	БП-IV	55	2 Ø5 mm	70		620
6	БП-V	55	3 Ø5 mm	70		620
7	БП-VI	55	2 Ø5 mm	70	Without levers	620
8	БП-VII	55	2 Ø5 mm	70		185

In Table 1 it is marked: ***n*** - number of external steel wires in each branch of the strengthening system; ***c*** - distance from the bottom ends of the beam to the point of maximum curvature of the external bar near the ends of the beam; ***k*** - distance from the beam support to the point where the flexible external bars are fixed to the bottom of the beam.

a) б)

Figure 2 – Testing of reinforced concrete beams strengthened with external steel wires of БП-V series including rigid levers – *a*, and of БП-VI series without levers - *b*

The testing of beams was conducted according to the given deformations in the certified building materials and structures laboratory (LBMK). Special mechanical jack was used. Control of the deformations of tested beams was performed using clock type indicators with a point value of 0.001 mm on the base that was 200 mm and the row of tensoresisters.

According to the experimental data the dependences «load - relative fiber deformation of concrete» and «load – deflection» were obtained. The results of the experimental data are presented in Table 2 and are shown in the dependency «carrying capacity M - deflection w in the middle and in the thirds of beam span» for conventional and strengthened beams in Fig. 3. In diagrams load is presented as a bending moment.

Table 2 shows the maximum values of deflections and bending moments achieved in the experiment and their values under the fixed parameter. In the first case the moment is shown under fixed deflections and in the second case the value of deflection is shown under fixed bending moment for beams.

Table 2 – The results of testing of beams

The name of the beam series	Bending moment, M, kNm		Deflection in the middle of the beam span W, mm	
	while w_{max}	while $W = \frac{1}{200} L_0$	while M_{max}	while M=*4.79 кНм*
БО	4.79	4.772	14.29	14.29
БП-I	11.39	10.423	14.41	1.429
БП-II	11.96	11.878	9.22	0.718
БП-III	12.90	11.033	16.33	0.975
БП-IV	16.57	16.570	10.02	0.714
БП-V	18.41	18.377	10.28	0.896
БП-VI	15.69	14.407	15.04	0.897
БП-VII	21.193	16.086	19.85	0.621

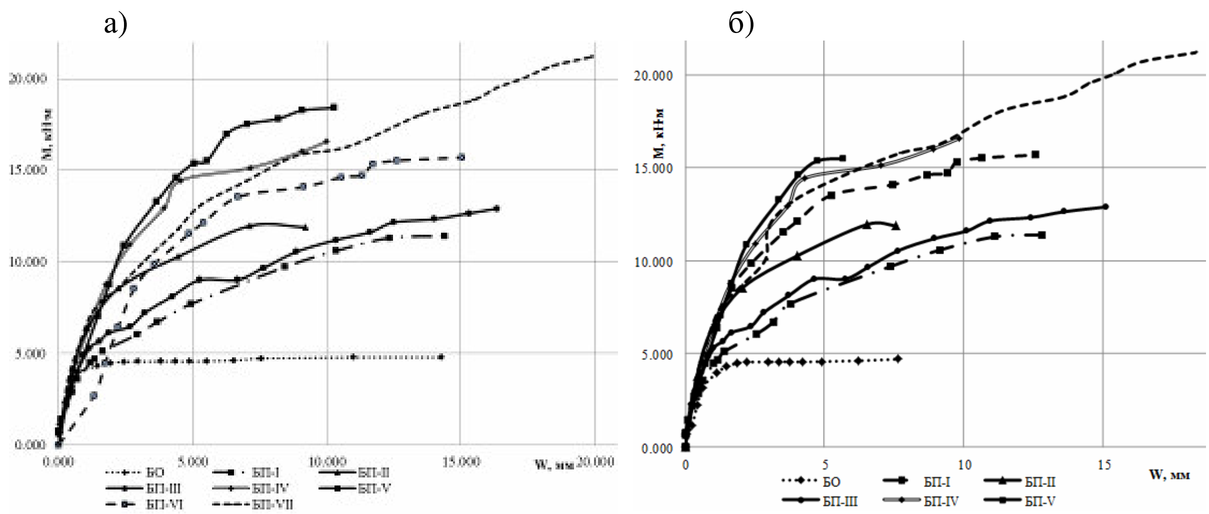

Figure 3 – Dependence «M-w» in the middle of the span - *a* and in the thirds of the span - *b* of strengthened and conventional beams

Comparative analysis of the carrying capacity and deformability of strengthened beams compared with the conventional beams is presented in Table 3.

It is proved, the carrying capacity of strengthened beams was higher than of the conventional ones. According to the results of experimental studies the carrying capacity of reinforced beams was significantly higher than the control one. Thus, while strengthening beams with the system including rigid levers and one external wire element with a diameter of

5 mm in each branch the carrying capacity of beams increased up to 2.7 times; when two external wire elements were used in each branch, the carrying capacity increased up to 3.5 times; when three wire elements were used, carrying capacity increased up to 3.85 times. But this variant required additional costs on making rigid steel levers. Therefore, beams without levers with two wire elements in one branch were developed, manufactured and tested. An increase in carrying capacity amounted to 3.3 times (БП-VI) when the branches were fixed in the places where rigid levers were fixed. After determining an efficient place for fastening the strengthening branch, carrying capacity increases up to 4.42 times (БП-VII). The strengthened beams have much greater rigidity than the regular ones; a beam with rigid levers and three wire elements in one branch showed the greatest rigidity among the strengthened beams.

Dependence «load M - relative fiber deformations of concrete ε» on the central cross-section is presented in Table 4 and diagrams in Fig. 5.

Table 3 – The strengthening effect

The name of the beam series	$\frac{M_{БП}}{M_{БО}}$		$\frac{W_{БП}}{W_{БО}}$	
	while w_{max}	while $W = \frac{1}{200} L_0$	while M_{max}	while M=4.79 *кНм*
БО	1	1	1	1
БП-I	2.378	2.184	1.008	0.100
БП-II	2.497	2.489	0.645	0.050
БП-III	2.693	2.312	1.143	0.068
БП-IV	3.472	3.472	0.701	0.050
БП-V	3.843	3.851	0.719	0.063
БП-VI	3.276	3.019	1.052	0.0628
БП-VII	4.424	3.371	1.389	0.043

Table 4 - Relative fiber concrete deformations of the beam central cross-sections

The name of beam series	Relative concrete deformations according to the bending moment that is equal to			
	carrying capacity, M_{max}, kNm		carrying capacity of the conventional beam M=4.79 *kNm*	
	ε_{c1} x10^{-5}	ε_{c2} x10^{-5}	ε_{c1} x10^{-5}	ε_{c2} x10^{-5}
БО	320.7	-489.3	320.7	-489.3
БП-I	68.7	-135.1	28.7	-49.3
БП-II	46.3	-55.5	13.79	-21.1
БП-III	77.3	-870.1	17.4	-16.3
БП-IV	46.7	-49.6	15.5	-16.1
БП-V	125.0	-90.0	46.1	-25.1
БП-VI	106.3	-133.7	21.6	-16.1
БП-VII	125.0	-119.1	16.4	-16.5

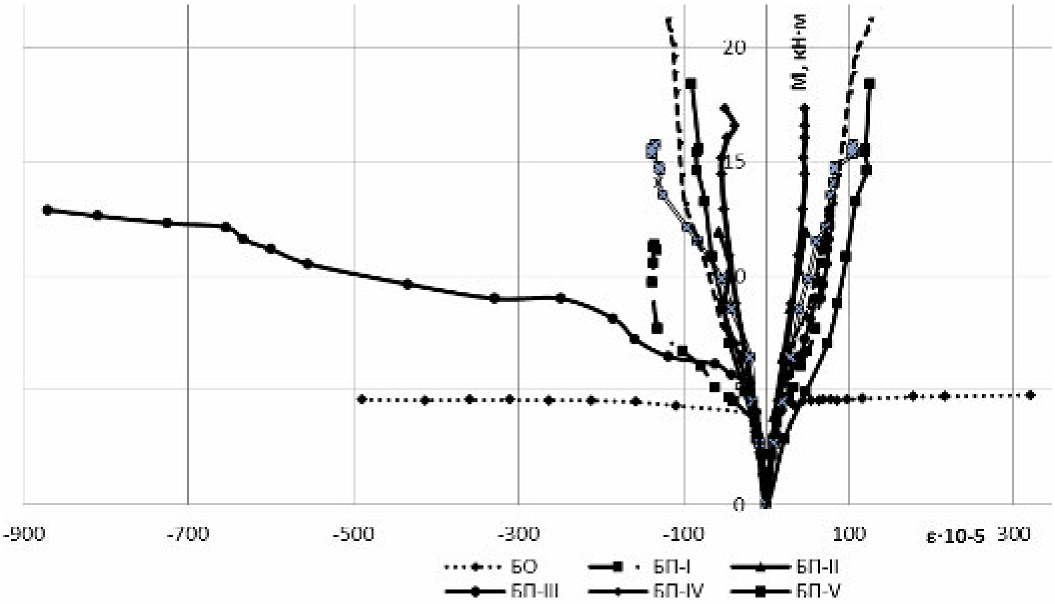

Figure 5 – Dependence «*M - ε*» on the beam central cross-sections

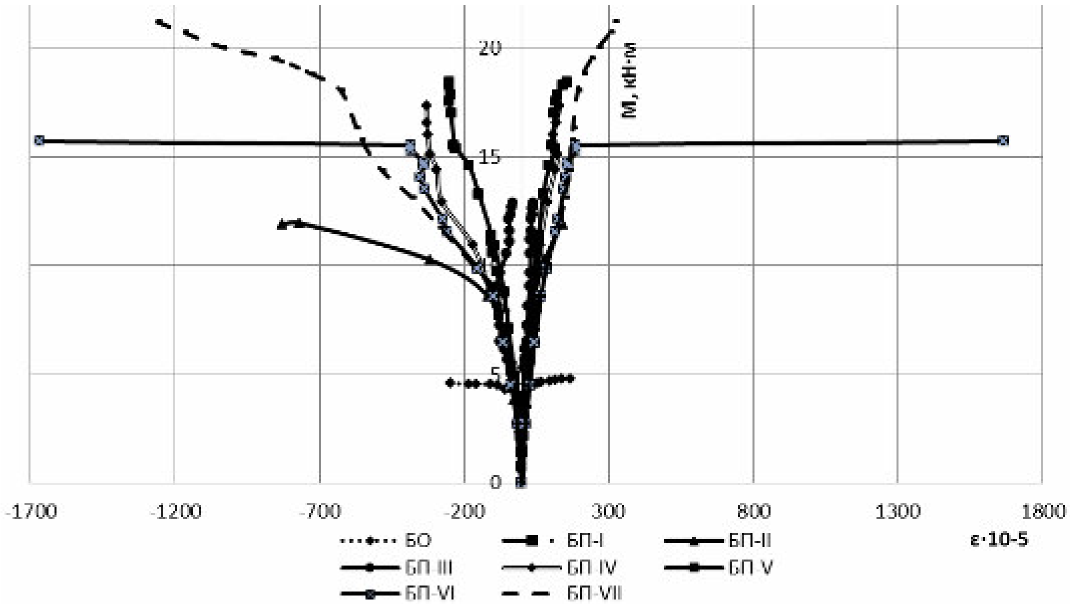

Figure 6 – Dependence «M - ε» on the beam cross-sections located in the thirds of their spans

It has been established that significant reduction in deformations of concrete is observed in strengthened beams in comparison with conventional beam series under the same load levels.

The character of crack formation and failure of the series of strengthened БП-I - VI and conventional control beams БО is shown in Fig. 7.

The study proposes a calculation method of such strengthened beams. It is considered the algorithm of calculation by the example of strengthened beam БП-VI without rigid levers. The method consideres the loss of tension because of friction under contact of external wire with rollers. The scheme of determining the forces in the branch of external wire is shown in Fig. 8.

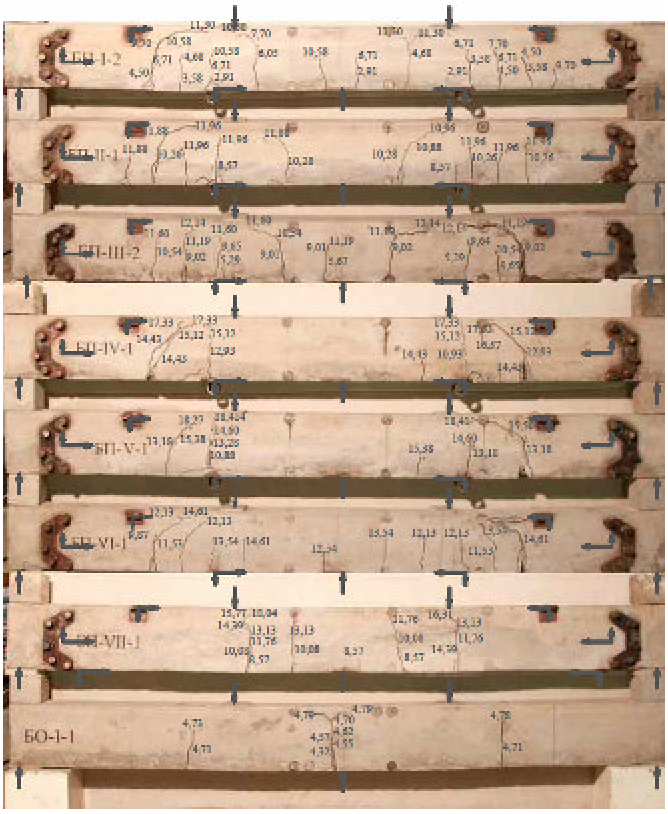

Figure 7 – Characteristic crack formation and failure of experimental beams

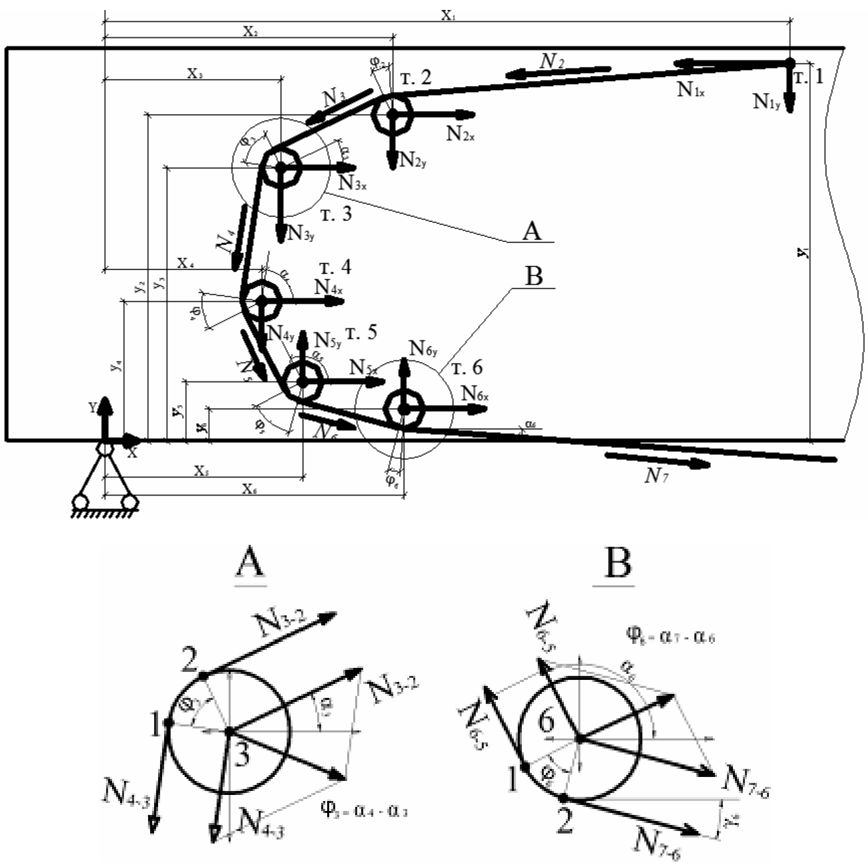

Figure 8 – Scheme of distribution of forces due to the action of the strengthening structure

According to the above scheme (Fig. 8) the resulting force on the lateral guiding element from the external reinforcement was obtained. The loss of the stress force in strengthening wire because of friction on rollers was considered. The calculation was conducted for the strengthening force of 19.796 kN in each branch of the reinforcement system for БП-VI beams. The value of the strengthening force was found in the experiment. The forces in the external wire in each part, from point 7 to point 1, is determined by the formula [5]:

$$N_{i-1} = \frac{N_i}{e^{f \cdot \varphi_{i-1}}} \tag{1}$$

where N_i – force in the leading branch;
N_{i-1} – force in the passive branch;
f – friction coefficient "steel on steel";
φ – angle of the contact between the branch and the roller.

After determining the resulting horizontal and vertical force the main calculation scheme of the strengthened beam has the form shown in Fig. 9.

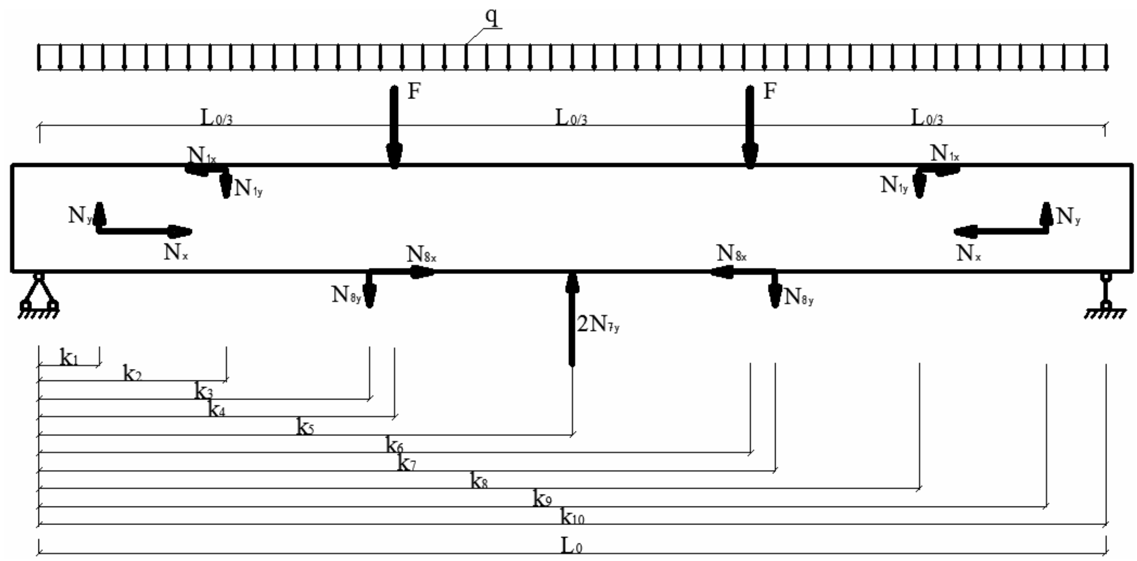

Figure 9 – The calculation scheme of the strengthened beam under the load of two concentrated forces F and live weight q

The diagram of total bending moment of the beam series БП- VI, strengthened with external bars, is shown in Figure 10.

The solid line shows total bending moment of the beam of БП- VI series (from unloading forces of the strengthening system and external load and live weight); the dotted line schematically shows the value of the bending moment without strengthening. In the calculations, the bending moment of the horizontal forces of the strengthening system was determined against the neutral line in each cross-section along the length of the beam, with the consideration of the variable eccentricity of beam deflections [1, 2, 11].

For the БП-VI series of reinforced beams the calculated bending moment was 15.508 kNm, and the corresponding experimental bending moment was 15.523 kNm. For this series of beam the discrepancy was insignificant, which confirms the adequacy of the calculation model БП-VI.

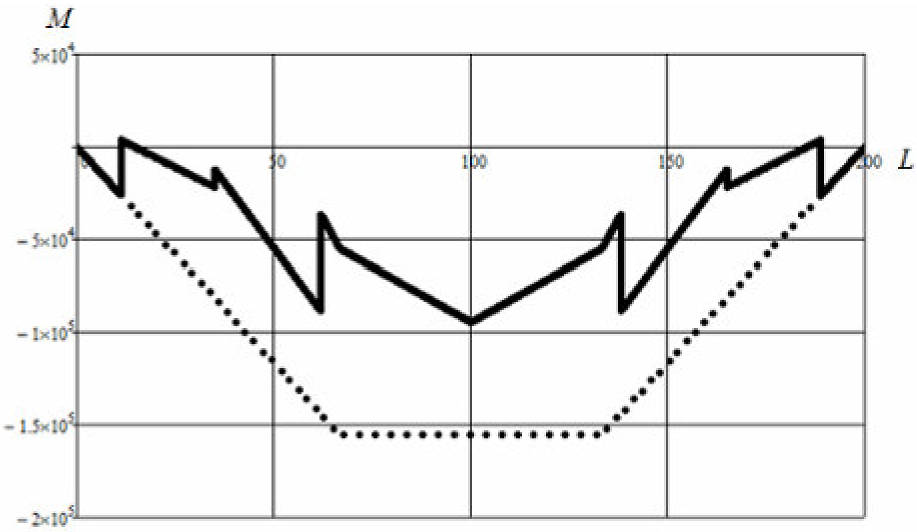

Figure 10 – Diagrams of the total bending moment of strengthened beams of БП- VI series

Conclusions. New strengthening system with external reinforcement for one-span reinforced concrete beams of a rectangular cross-section is presented. According to the results of experimental studies, carrying capacity increased up to 2.7 times in the beams strengthened with the system including rigid levers and one external wire element with a diameter of 5 mm in each branch; when two external wire elements were used in each branch, carrying capacity increased up to 3.5 times; when three wire elements were used. carrying capacity increased up to 3.85 times. Increase in the carrying capacity of the beams strengthened without levers amounted to 3.3 times (БП-VI) when the branches included two wire elements. After determining an efficient place for fastening the strengthening branch carrying capacity increases up to 4.42 times (БП-VII). Here, the strengthened beams had much greater rigidity than the regular ones. The beam with rigid levers and three wire elements in one branch showed the greatest rigidity. Convergence between the theoretical and experimental data by the example of БП-VI beams meets the requirements of calculation accuracy.

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Received 07.10.2016

UDC 624.073

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MULTISTORY FRAMED BUILDINGS WITH SLAB CAST OVER PRECAST JOISTS: RECOMMENDATIONS FOR DESIGNING CONCRETE ELEMENTS KEY JOINTS

The role of joints which are used to provide interoperability of separate elements in prefabricated monolithic frame design systems with flat slabs was researched. Special attention is paid to key joints that are exposed to significant shear strength and provide the structures' solidity during the operation. On the basis of the experimental studies, recommendations for the joints design are suggested. In Poltava National Technical University a key joints calculation method has been developed, which considers the damage nature, a set of determining impact factors and is recommended for extensive use.
***Keywords:** precast-and cast-in situ structural system, joint, key, strength, plasticity, cut.*

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БАГАТОПОВЕРХОВА КАРКАСНА БУДІВЛЯ ЗІ ЗБІРНО-МОНОЛІТНИМ ПЕРЕКРИТТЯМ: РЕКОМЕНДАЦІЇ ДО ПРОЕКТУВАННЯ ШПОНКОВИХ З'ЄДНАНЬ ЗАЛІЗОБЕТОННИХ ЕЛЕМЕНТІВ

Висвітлено роль стиків, котрі використовуються для забезпечення спільної роботи окремих елементів збірно-монолітних каркасних конструктивних систем із плоскими перекриттями. Особливу увагу приділено шпонковим з'єднанням, які сприймають значні зусилля зсуву та забезпечують монолітність конструкції в період експлуатації. На основі виконаних експериментальних досліджень запропоновано рекомендації для проектування стиків. У Полтавському національному технічному університеті імені Юрія Кондратюка розроблено методику розрахунку шпонкових стиків, за допомогою якої досліджено характер руйнування, сукупність визначальних факторів впливу; її рекомендовано до широкого застосування.
***Ключові слова:** збірно-монолітна конструктивна система, стик, шпонка, міцність, пластичність, зріз.*

Introduction. The experience of national and foreign multi-story residential and public buildings construction shows that the most promising in this respect are framed systems with flat discs of slabs.

Frames are suggested to be made of precast and cast-in-place reinforced concrete, that permits their manufacture without additional costs as manifold statically indeterminate system with great potential for redistribution efforts under load between its individual components, reducing the size of sections and the amount reinforcement required, thus reducing material consumption of structures. The use of precast components improves the products' quality and promotes the development of construction industry plants.

Analysis of recent research sources and publications. One of the first attempts to use slab cast over precast joists with smooth ceilings is considered the «SOCHI» system [1]. It includes columns and precast-monolithic discs of the slab (Fig. 1) formed by hollow core slabs and monolithic girders with the height equal to the plates' thickness. For longitudinal slab-to-slab joints reinforcement is designed. Expansion reinforced joints between the slabs and solid-cast bearing girders form a rigid cross-system of primary and secondary beams. Leaning of plates on the girders is provided by means of concrete keys at the ends and on the side faces of the plates, formed when the girders are concrete casted due to the concrete mixture embedding of holes and cavities in plates. The system was not widely used, but it has inspired other present-day popular systems.

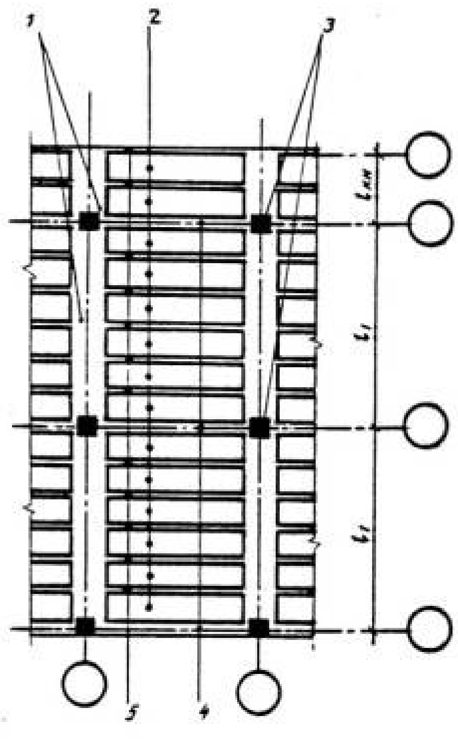


Figure 1 – The «SOCHI» type slab cast over precast joists:
1 – solid-cast bearing girders; 2 – precast slab panels; 3 – precast columns; 4, 5 – solid-cast beams between the columns and precast slabs

«ARCOS» structural system [2], in which: precast-monolithic discs of slabs using hollow-core panels and monolithic bearing and braced girders with the similar cross sections are made flat; reduced to the solid floor thickness is 12 – 14 cm and provides usage of spans with the length of 7.2 m or more (Fig. 2). Leaning of slabs on the bearing girders is performed due to concrete keys formed in the plates' hollows at their ends at the girders' concrete casting.

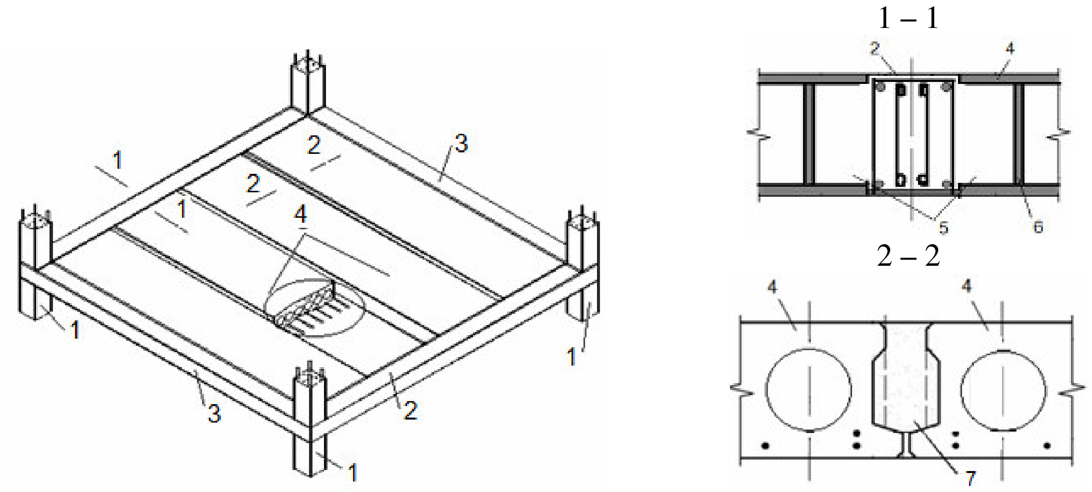


Figure 2 – Fragment of the «ARCOS» structural system building:
1 – columns; 2 – solid-cast bearing girders; 3 – solid-cast braced girders;
4 – precast hollow-core slabs; 5 – concrete keys of bearing girders;
6 – key size stopper; 7 – concrete keys between the slabs

The existing foreign analogues of slab cast over precast joists using hollow-core slabs (even at a higher organization and manufacturing culture) for security reasons it is not recommended to place the supporting concrete sections in the form of concrete keys. Ukraine has a number of patents that improve the joint of plates with a solid-cast girder by reinforcing concrete keys with flat or spatial frames in the form of a pyramid [3, 4], which increases their reliability.

At Prydniprovsk State Academy of Civil-Engineering and Architecture a slab cast over precast joists is suggested [5], which is different from that of Arcos in terms of the hollow-core plates size which is made shorter in each site, and the width of solid-cast bearing girders is due to it wider [6]. The hollow-core plate is connected to the solid-cast girder by means of keys, reinforced by flat frames with longitudinal reinforcement 2∅6 A240C.

Precast and cast-in-place building frame suggested by Ye.P. Gurov [7] includes columns and precast-and solid-cast girders, which prefabricated parts are made with open or hidden supporting consoles. Girders are installed on the columns floor-by-floor mostly forming corbels. Hollow-core slab panels with anchor keys in the cavities at their ends are having embedded anchoring wall ties. The keys arrangement is performed through a hole cut in the top of the concrete slab by pour concrete casting through it. The key is reinforced by a frame in which the longitudinal reinforcement is placed in the top and in the middle of its height.

To increase the height of the pier cap at high loads, a technical solution is suggested (Fig. 3), which provides key joint of the girder with the precast column without holes in the floor level [8].

The current standards [9] when calculating key joints, are taking into account the limited number of impacts, and therefore the strength standards are significantly loose compared to the strength indicators demonstrated in the experiments [10 – 12].

At PoltNTU, a variance calculation method has been developed based on the plasticity concrete theory [13], which is commonly used, accurate and simple enough, permitting to calculate the strength of key joints at cutting, with account of all the determinants totality. The method has passed a reasonable testing.

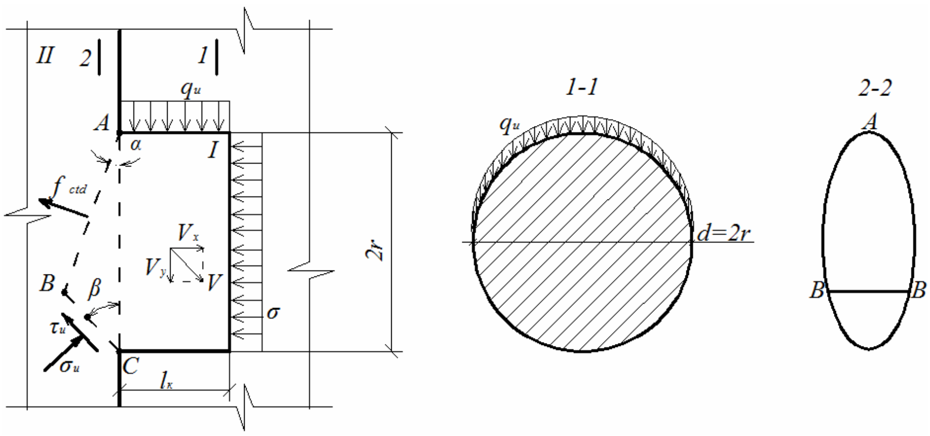


Figure 3 – Cinematically possible diagram of round evenly reduced concrete keys at cutting

Highlighting of still unsolved aspects of the general problem. In all the above mentioned slab cast over precast joists, key joints are available providing strength and reliability of the entire system of the building as a whole.

Setting the objective. The aim of the study is to submit proposals for the design of concrete elements' key joints in the precast and solid-cast systems.

Basic material and results. The suggested method of strength calculation is based on considering the damage nature and taking into account the main impact factors. At calculating a single-key joint, two possible cases of destruction are considered: «the key» and «the joint» destructions that define a cinematically possible scheme of destruction and the ultimate load value. The destruction type depends on the ratio of the joint's geometric parameters [14].

The hollow-core slab panels' joint with solid-cast bearing girders in the Arcos system have a round cross-section of keys. At the slab deformation a spread occurs, leading to the keys' reduction. According to the current standards [9], the round section is replaced by the equivalent square one.

The authors suggest scheme of cinematically possible destruction of evenly reduced round concrete key, which is presented in Fig. 3 [15].

According to the suggested method, the known parameters in solving the problem of durability are the dimensions of the key l_k, $2r$, b_k, the embedding concrete strength characteristics f_{cd}, f_{ctd}, reduction.

The unknown values in the present problem are: the ultimate load q_u, slope angle α of the AB platform and β platforms BC to the vertical, the ratio of speeds $k = V_x / V_y$.

The formula for determining the ultimate load is as follows:

$$q_u = m\left[2B\left(1+\frac{1}{4}\left(\frac{ktg\beta+1}{k-tg\beta}\right)^2\right)^{0,5}-1\right]\frac{2}{3}\gamma(k-tg\beta)\frac{tg\alpha}{tg\beta+tg\alpha}+ \\ + f_{ctd}(k+tg\alpha)\frac{2}{3}\times\gamma\frac{tg\beta}{tg\beta+tg\alpha}+\frac{\pi\sigma k}{4}\gamma \quad , \qquad (1)$$

where $\gamma = 2r/h_k$,

$m = f_{cd} - f_{ctd}$,

$B = \sqrt{(1+\chi/(1-\chi)^2)/3},$

$\chi = f_{ctd}/f_{cd}$.

The round key's strength turned to be by 10% lower than that of the equivalent square one due to the load features and the form of the surface damaged. There is also the possibility of considering the uneven reduction force application heightwise the key.

The joint strength calculation results for a round-hollow panel with the width of 1,5 m (7 hollows) and the key diameter (the panel's cavity) d = 159 mm, the length of l_k = 100 mm, the embedding concrete class C25/30: f_{cd} = 15,3 MPa, f_{ctd} = 1,08 MPa, provided γ_{c2} = 0,9 with account of the cross-section shape, reduction and its application place and reinforcement are presented in Table 1.

Table 1 – Comparison of the hollow-core panel and a bearing girder joint's strength performed by different methods

Source / Joint type	[17] round section	[9] equivalent square section	[13] round/ equivalent square section
Load (kN)			
Concrete keys	286,7 / –	– / 77,14	– / –
Reinforced concrete keys	– / –	– / 115,1	202,65 / 230,16
Reduced concrete keys	– / –	– / 265,2	341,32 (207,69) / 381,15(231,42)
Reduced reinforced concrete keys	– / –	– / 302,17	364,63 (239,2) / 412,09 (268,7)
* in brackets the value of the joint strength is given for the reduction applied in the lower part of the key			

At the calculations of reinforced concrete keys, their smallest possible reinforcement by flat frames with longitudinal reinforcement is set 2∅3 Bp-1.

The submitted suggestions as to improving the design of the hollow-core panel with a girder, which provide the use of cylindrical frame instead of a flat frame and its being performed as a hollow triangular pyramid that will provide equal strength of the key joint both in the vertical and in the horizontal planes, will improve its reliability at seismic impacts [16].

In [8], the joint of precast column with solid-cast girder, which has thickening on support, is designed as three-keys joint (Fig. 4).

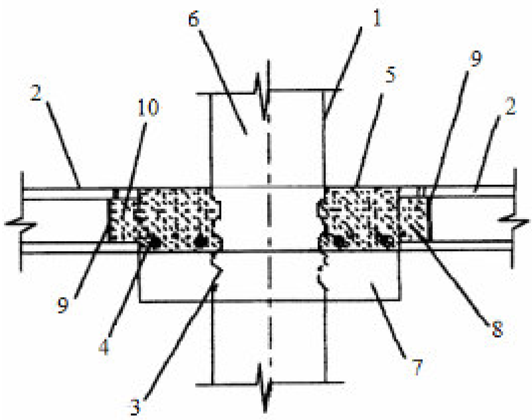


Figure 4 – Key joint of the precast column with solid-cast girders:
1 – columns; 2 – hollow-core panels; 3 – concrete keys; 4 – reinforcing frames; 5 – middle solid-cast girders; 6 – side of the column; 7 – thickening of the girders' bearing; 8 – end hollows of the panel; 9 – stoppers; 10 – keys of girders

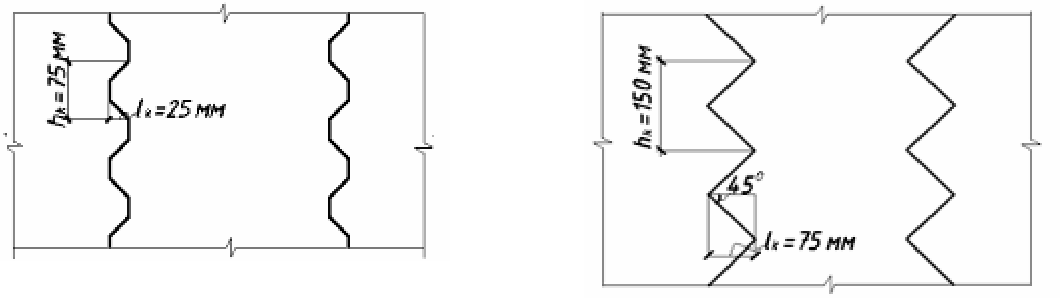
According to the results of series of experimental studies performed at PoltNTU, it is suggested to calculate the uneven behaviour of keys heightwise according to the suggested empirical dependence:

$$V_{sh,n}^{k} = V_{sh,1}^{k}\left(1+0{,}95\ln n_k\right), \tag{2}$$

where $V_{sh,1}^{k}$ – is a single key’s strength,

n_k – is the number of keys in the joint.

It is recommended to design keys of the trapezoidal shape, both in terms of the increased strength and the technology of manufacturing joints, or triangular joints to avoid inter-key space. The calculation method considers the shape of the key profile. The ratio of the key depth to its height should be taken within l_k / h_k =0,25 – 0,5.



$V_{sh,c} = 243{,}9\ kN\ \left(l_k / h_k = 0{,}3\right)$ $\qquad$ $V_{sh,c} = 361{,}32\ kN\ \left(l_k / h_k = 0{,}5\right)$

Figure 5 – Geometric dimensions of the column with a girder joint with the trapezoid and triangular keys

Conclusions. When solving the problems of the key joints’ strength in the reinforced concrete elements of the slab cast over precast joists, the circular shape of the key’s cross-section was considered together with the reinforcement features and the reduction effort’s location. The relation was also suggested for the uneven keys’ behaviour lengthwise, which improves the suggested calculation method, increases its accuracy and opens opportunities for improvement of design solutions. All the joints considered are having the sufficient strength reserve.

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Received 28.11.2016

UDK 69.036.03

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STRESS AND STRAIN BEHAVIOUR OF REINFORCED CONCRETE ANISOTROPIC SHELLS

The character changes deflected mode of «Monofant» system fragments. For experimental studies state snippets new structural system of "Monofant" multi conformity was deformed and method based on hydrostatic load when the load is given by the weight of water, was used, and its value is governed by the height of the water column. To determine the qualitative and quantitative research object deformation measurements displacement cylindrical and spherical shells in 25 points was carried. However, the configuration structure complexity was given, which in turn led to the emergence of both vertical and horizontal movements of the studied surface shell under vertical load, the need to measure movements in two directions in a cylindrical shell of vertical displacement and a spherical shell. The study shows that the structural elements are endowed with all the necessary strength characteristics and attributes rigidity bearing elements of reinforced concrete buildings.

***Key words:** stress-strain state, strain, envelope, reinforced concrete, experimental research.*

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НАПРУЖЕНО-ДЕФОРМОВАНИЙ СТАН ЗАЛІЗОБЕТОННИХ АНІЗОТРОПНИХ ОБОЛОНОК

Досліджено характер зміни напружено-деформованого стану фрагментів системи «Монофант». Для експериментальних досліджень деформованого стану фрагментів нової конструктивної системи багатокритеріального призначення «Монофант» використано метод, заснований на гідростатичному навантаженні, коли воно задається вагою води, і її величина регулюється висотою водяного стовпа. Для визначення якісного та кількісного характеру деформування об'єкта дослідження здійснено виміри переміщень циліндричної і сферичної оболонок у 25-ти точках. При цьому з огляду на складність конфігурації конструкцій, що у свою чергу зумовило появу як вертикальних, так і горизонтальних переміщень досліджуваної поверхні оболонки під дією вертикального навантаження, виникла необхідність вимірювання переміщень у двох напрямках у циліндричній оболонці й вертикальних переміщень в сферичної оболонці. З проведеного дослідження випливає, що конструктивні елементи наділено всіма необхідними характеристиками міцності та жорсткісними атрибутами несучих елементів будівель з монолітного залізобетону.

***Ключові слова:** напружено-деформований стан, переміщення, оболонка, залізобетон, експериментальні дослідження.*

Introduction. Building structures own weight minimizing is one of the most important trends in researching and enhancing construction options with limited resources. A new construction system named «Monofant» [1] was developed by scientific efforts of the building structure department of O.M. Beketov NUUE.

Using the large scale sheet blockouts of expanded polystyrene and mineral rock wool significantly decreases building cost and enhances its thermal and sound insulation parameters. Making the shells by the wet shotcreting method is the special feature of «Monofant» spatial curved structures [2]. The formed structure saves concrete (up to 40-45%) and provides slight saving of reinforcing bars as well.

In this case the shape and dimensions of created internal cavities drastically influences on the character of stress and strains conduct of the system.

Analysis of recent research sources and publications. The research objects are fragments of cylindrical/ spherical shells of the «Monofant» system of plan dimensions 2200×2200 mm. Each fragment is made of 50 mm thick external/ internal concrete coatings and 160 mm permanent blockout of expanded polystyrene inserted between them. Coating reinforcement is done with grid of 200×200 mm mesh, and d = 6 mm (A-I). The 50 mm ribs providing combined operation of shells are arranged in diagonal direction of the shell. The ribs are reinforced with flat frame of d = 10 mm (A-I). The general diagram of structures is given in fig. 1 -- 2.

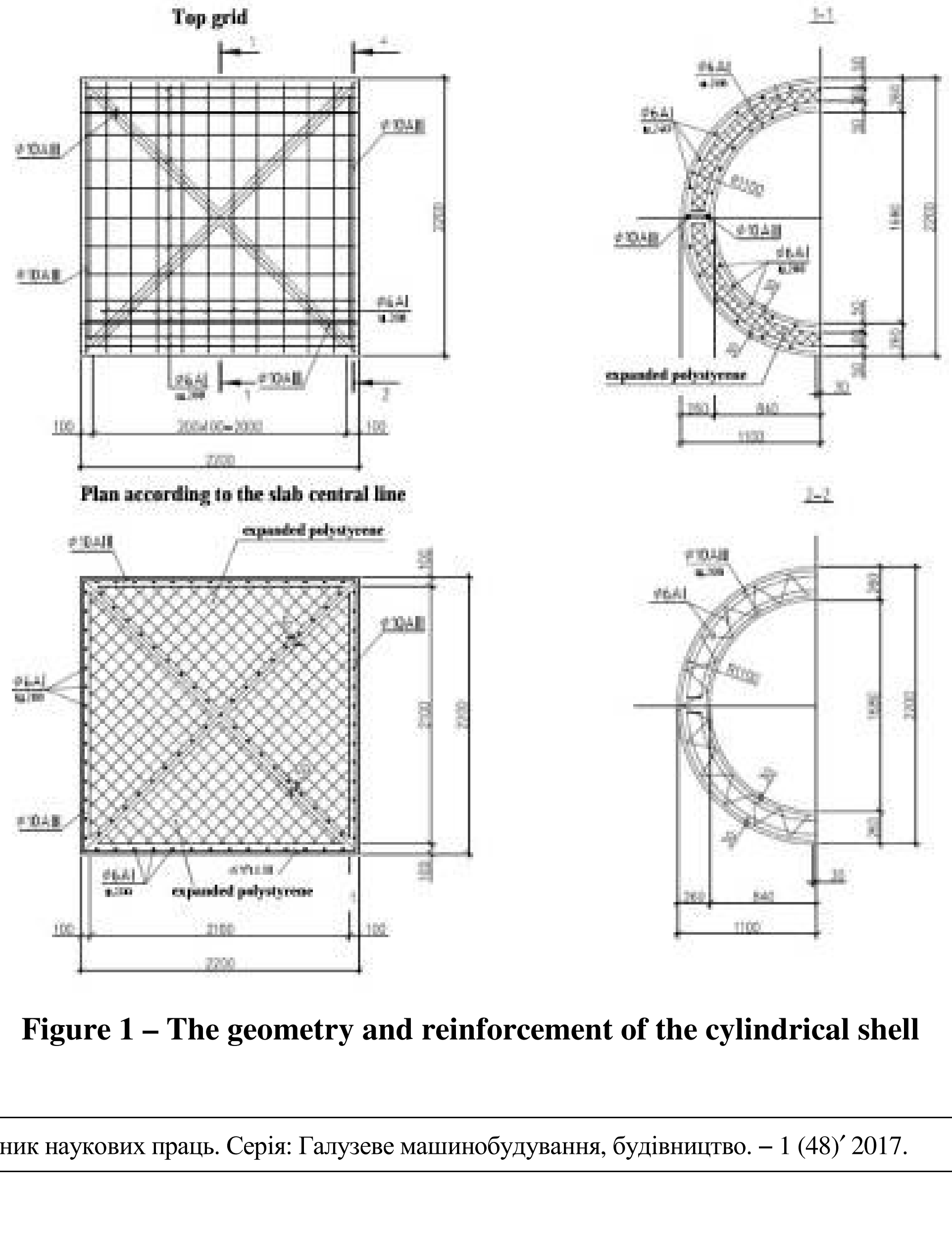


Figure 1 – The geometry and reinforcement of the cylindrical shell

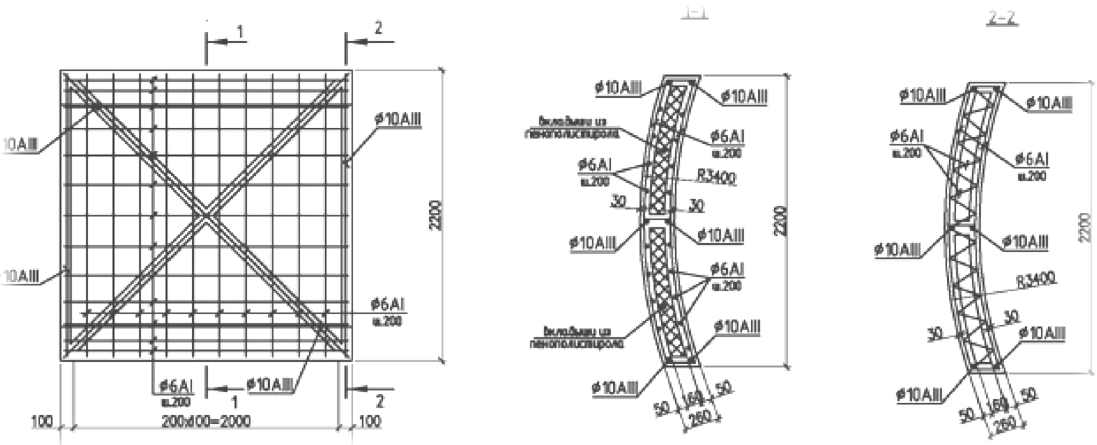


Figure 2 – The geometry and reinforcement of the spherical shell

Main part of the research and obtained results. A finite element model of the developed shell was built in the «ANSYS» software within the research.

The toolset of the mentioned calculation package allows to carry out the automatic triangulation of complex shapes into finite volumetric elements through the automatic import from additional software package of 3D modeling (in this case «Autodesk Inventor» was used). (fig. 3, 4).

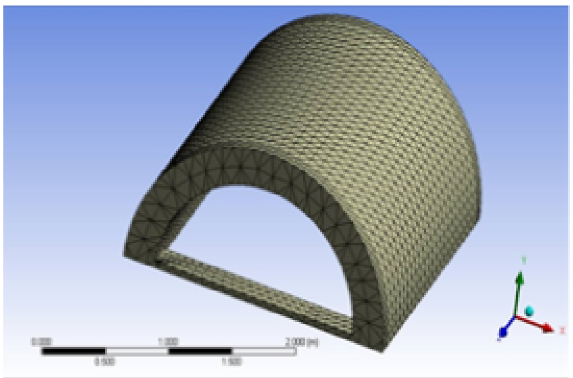


Figure 3 – Building a cylindrical shell finite element grid

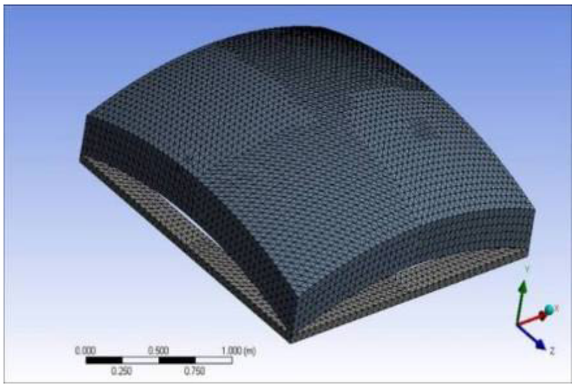


Figure4 –shell finite element model

The built cylindrical shell model contains 209,319 assemblies and 100,271 finite elements, whereas the built spherical shell contains 459,061 elements and 261,791 assemblies.

Resting was chosen as swiveling and fixed on four angular points of the shell in the plan.

Evenly distributed load was applied to the shell surface according to diagrams described in papers [3, 4]. Taking into account the structure rigidness, the evenly distributed load value made 10 kN/m^2. Once a stress-strain behavior theoretical analysis of a spherical shell fragment had been completed, experiment series on fabricated elements followed.

Basic material and results. The completed theoretical analysis [4, 5] is supported by experimental researches of new «Monofant» structural system fragments strained state against many criteria and by the hydrostatic method.

Loading system. In the research the method based on hydrostatic loading was used, when the load was created by water mass and its value was controlled by height of water column.

The loading mode was chosen according to an experiment program. Loading level was defined by height of water in the pool. In this case the mentioned level was provided by water supply/ drainage system to/ from the pool. To register measured movements of the studied object, series of dial indicators was installed; they allowed to measure stress-strain features of the studied object. A device to test slabs and shells in situ under influence of vertical short/long time loads was chosen as a basis for the mentioned tests.

The device contained the studied structure 1, installed on supports 2, boards 3 that created a tank, being installed along the contour of the loaded area, waterproof film 4 inside the tank, supplying/ draining hoses 5. Such structure is simple, cheap, and easy matching with the measurement system.

Considering the difficult configuration of the loaded shell surface, and in order to evenly distribute the water column pressure, the arrangement of additional partitions inside the pool in the longitudinal direction was considered as necessary. Therefore, the pool constructed over the spherical shell was a cell system consisting of 121 cells with dimension of 200×200 mm and height of 1200 mm (fig.5).

Pool walls and internal partitions were made of 20 mm thick multilayer glued plywood (fig. 6). The pool constructed over the cylindrical shell was a cell system consisting of 11 cells with dimension of 200×2200 mm and height of 1200 mm.

Pool walls and internal partitions were made of 20 mm thick multilayer glued plywood (fig. 7, 8).

Figure 5 – A general view of the cell pool to test the spherical shell

Figure 6 – The cell system over the shell

Figure 7 – The cell system over the cylindrical shell to carry out tests

Figure 8 – Boards forming a pool over the cylindrical shell

The pool external walls rested on the frame, whereas the internal partitions were rigidly attached to the external walls by glue and self-tappers. 10 mm space was arranged between the pool internal partitions and the shell. The space was filled with expanding foam to distribute the pool own mass over the frame only. Specifically fabricated plastic pockets were installed in each formed cell. It provided pool impermeability.

Measurement system. To define qualitative and quantitative features of studied object straining, shell movement measurement were done in 25 points. In this case, considering the complexity of the structure, that in its turn had preconditioned the occurrence of both vertical and horizontal movements of the studied shell surface under the vertical load effect, the necessity of measuring 2 direction movements (fig. 9) in the cylindrical shell and vertical movements in the spherical shell emerged (fig. 10).

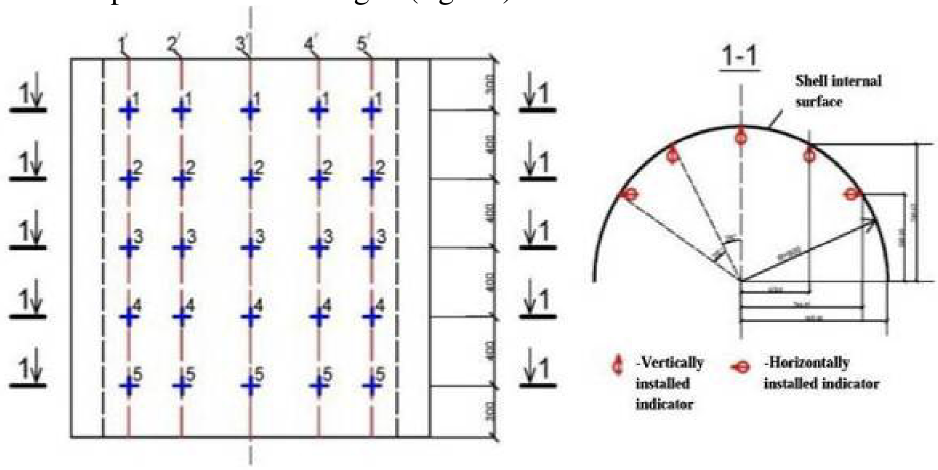


Figure 9 – A diagram of indicator installation in the cylindrical shell

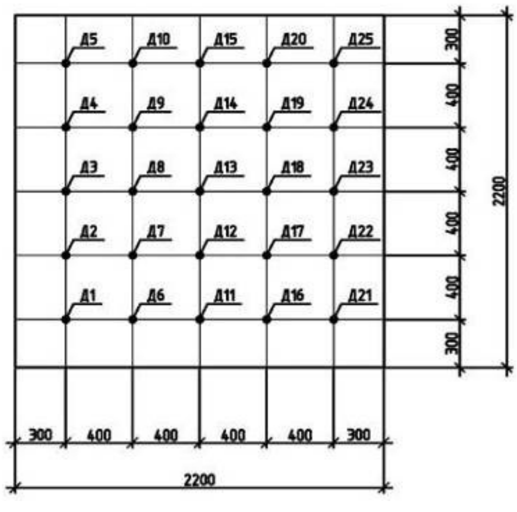


Figure 10 – A diagram of indicator installation in the spherical shell

Besides, considering the shell significant rigidity, which had preconditioned the smallness of the expected movement values, dial indicators with 2 um division value and 2 mm rod travel were used. The indicators were fastened in the indicated points by specifically fabricated frames made of 40×3 mm square pipes and 10×10 mm metal bar (fig. 11) that in their turn were welded to the main frame. Sequence of experiments is illustrated in fig. 12 -- 16.

Six independent cycles of loading/ unloading were done according to each loading diagram. Averaged data of six loading results were analyzed. The data are given in the table 1 – 2.

Figure 11 – Measurement system

Figure 12 – Filling the cells with water

Figure 13 – Adjusting a water level in cells

Figure 14 – A cell filled with water

Figure 15 – Cells filled with water

Figure 16 – The bench to carry out tests

Table 1 – Averaged readings of vertical movement indicators of the cylindrical shell

Indicator	Vertical movements, absolute values (mm)					
	Full loading	Δ	1/2 of the surface	Δ	1/3 of the surface	Δ
1'1	0,041	-0,007	0,01	0,007	0,024	-0,016
1'2	0,138	-0,006	0,062	-0,008	0,053	-0,007
1'3	0,228	-0,01	0,112	-0,015	0,086	-0,006
1'4	0,138	-0,006	0,062	-0,008	0,053	-0,001
1'5	0,041	-0,007	0,01	0,007	0,024	-0,016
2'1	0,012	-0,004	0,055	0,006	0,045	0,005
2'2	0,194	-0,007	0,069	0,0	0,056	0,0
2'3	0,237	-0,004	0,078	-0,011	0,063	-0,004
2'4	0,194	-0,007	0,069	0,0	0,056	0,0
2'5	0,012	-0,004	0,055	0,006	0,045	0,005
3'1	0,197	-0,003	0,098	0,007	0,081	0,007
3'2	0,217	-0,004	0,105	-0,001	0,086	-0,008
3'3	0,242	-0,013	0,117	-0,01	0,092	0,004
3'4	0,217	-0,004	0,105	-0,001	0,086	-0,008
3'5	0,197	-0,003	0,098	0,007	0,081	0,007
4'1	0,012	-0,004	0,166	0,005	0,131	0,005
4'2	0,194	-0,007	0,178	0,0	0,148	-0,006
4'3	0,237	-0,004	0,192	-0,011	0,157	-0,005
4'4	0,194	-0,007	0,178	0,0	0,148	-0,006
4'5	0,012	-0,004	0,166	0,005	0,131	0,005
5'1	0,041	-0,007	0,088	-0,012	0,088	-0,002
5'2	0,138	-0,006	0,102	-0,006	0,093	-0,007
5'3	0,228	-0,01	0,121	-0,005	0,111	-0,004
5'4	0,138	-0,006	0,102	-0,006	0,093	-0,007
5'5	0,041	-0,007	0,088	-0,012	0,088	-0,002

The completed analysis of vertical movements of the cylindrical shell at full loading, 1/2 and 1/3 of the surface portion is illustrated in fig. 17 – 19.

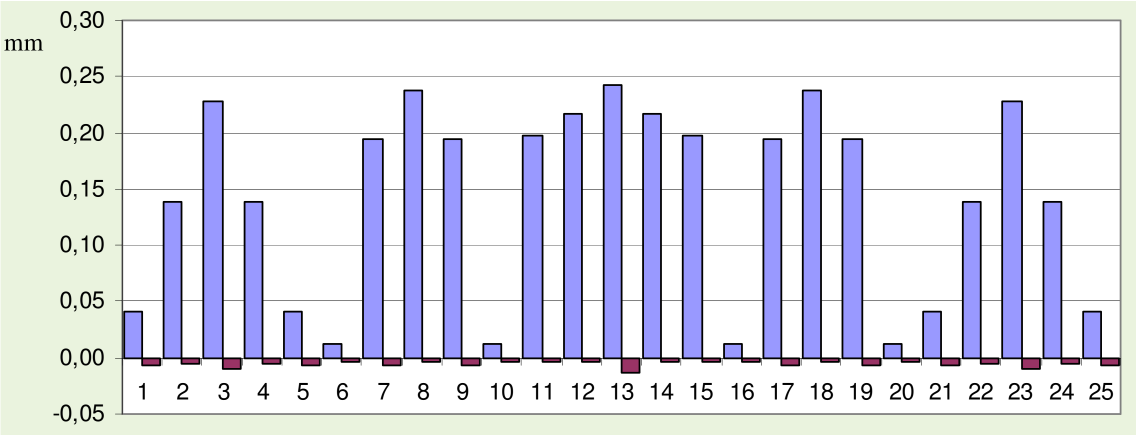


Figure 17 – Graphical representation of cylindrical shell vertical movements under the evenly distributed load of 10kN/m^2 over the entire structure surface

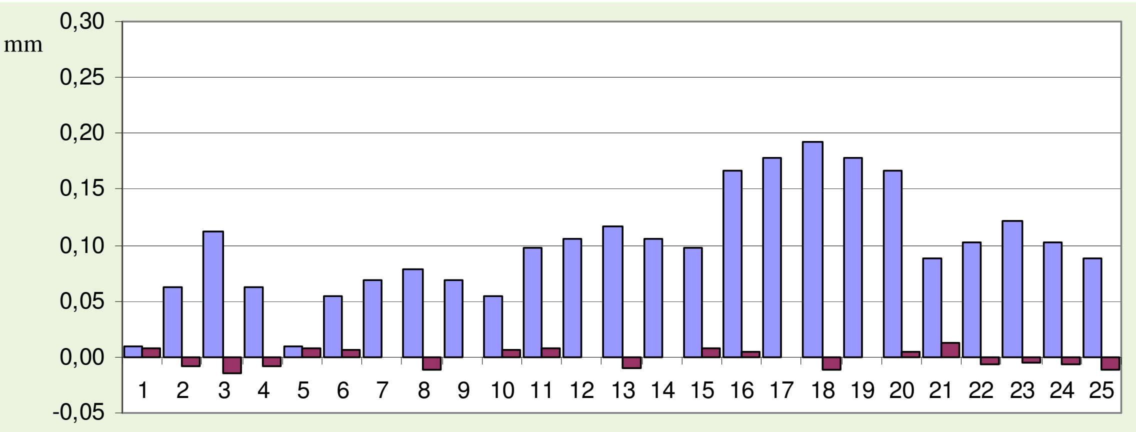


Figure 18 – Graphical representation of cylindrical shell vertical movements under the evenly distributed load of 10kN/m^2 over 1/2 of the structure surface

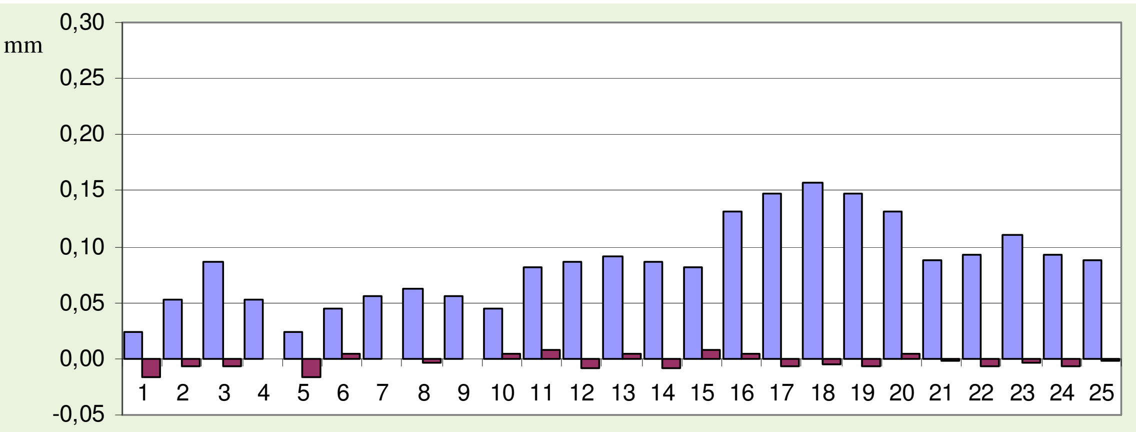


Figure 19 – Graphical representation of cylindrical shell vertical movements under the evenly distributed load of 10kN/m^2 over 1/3 of the structure surface

Table 2 – Averaged readings of spherical shell indicators

Indicator	Vertical movements (mm)							
	Full loading	Δ	1/2 of the surface	Δ	1/4 of the surface	Δ	1/8 of the surface	Δ
1	0,275	0,021	0,117	0,004	0,044	0,026	0,016	0,006
2	0,382	0,009	0,136	0,000	0,055	0,024	0,023	0,006
3	0,419	0,082	0,153	0,021	0,068	0,013	0,027	0,007
4	0,377	0,013	0,133	0,003	0,051	0,003	0,020	0,009
5	0,264	0,030	0,115	0,006	0,039	0,031	0,015	0,007
6	0,383	0,008	0,157	0,023	0,142	-0,021	0,042	0,001
7	0,597	-0,021	0,195	0,004	0,149	-0,018	0,054	0,000
8	0,496	0,026	0,197	0,011	0,159	-0,003	0,063	-0,004
9	0,590	-0,014	0,185	0,014	0,093	0,003	0,049	0,005
10	0,387	0,004	0,153	0,027	0,050	0,004	0,039	0,003
11	0,417	0,084	0,231	0,020	0,208	0,014	0,077	-0,009
12	0,490	0,031	0,257	0,031	0,231	-0,014	0,091	0,000
13	0,686	-0,054	0,281	0,022	0,178	-0,026	0,096	-0,001
14	0,495	0,026	0,265	0,023	0,156	-0,005	0,095	-0,004
15	0,420	0,081	0,241	0,010	0,064	0,019	0,080	-0,012
16	0,384	0,007	0,295	0,005	0,233	-0,016	0,108	-0,016
17	0,595	-0,019	0,331	-0,010	0,237	0,011	0,130	-0,008
18	0,498	0,021	0,349	0,012	0,227	-0,018	0,184	-0,015
19	0,593	-0,017	0,335	-0,006	0,151	-0,020	0,127	-0,005
20	0,381	0,010	0,308	-0,008	0,059	0,020	0,103	-0,012
21	0,268	0,026	0,238	-0,039	0,178	0,021	0,120	-0,017
22	0,386	0,004	0,349	-0,008	0,236	-0,013	0,138	-0,011
23	0,413	0,088	0,405	-0,007	0,211	0,011	0,223	0,003
24	0,380	0,010	0,358	-0,017	0,144	-0,023	0,135	-0,008
25	0,275	0,021	0,246	-0,047	0,047	0,019	0,118	-0,015

Graphical representation of spherical shell vertical movements under the evenly distributed load of 10 kN/m^2 over the entire structure surface, 1/2, 1/4, and 1/8 portions of it is given in fig. 20 – 23.

Comparison of theoretical and experimental displacements spherical shell and cylindrical shell is given in the table 3 – 4.

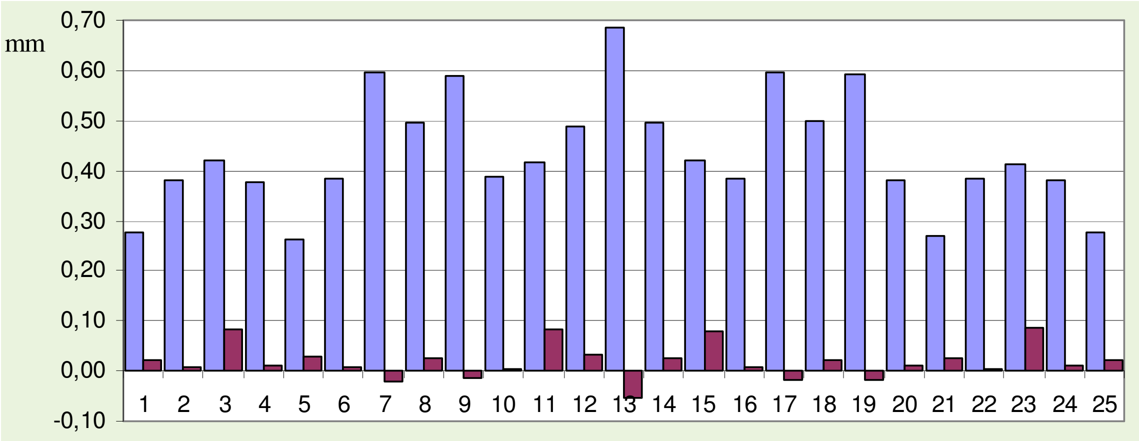


Figure 20 – Graphical representation of spherical shell vertical movements under the evenly distributed load of 10kN/m^2 over the entire structure surface

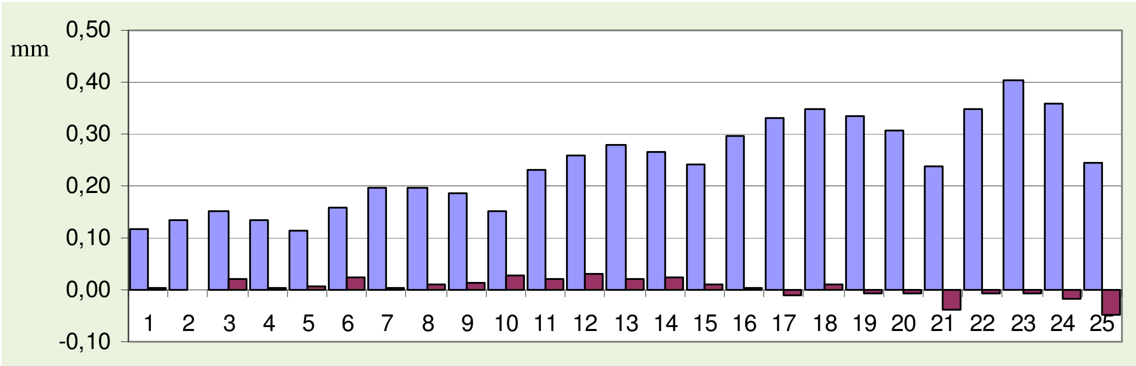


Figure 21 – Graphical representation of spherical shell vertical movements under the evenly distributed load of 10kN/m^2 over 1/2 of the structure surface

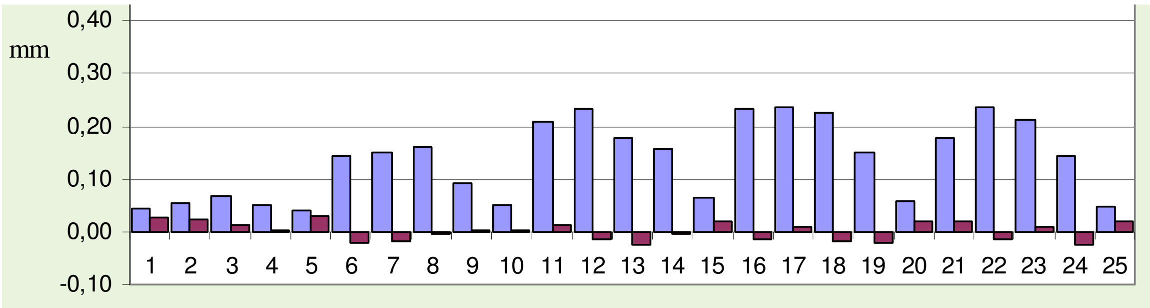


Figure 22 – Graphical representation of spherical shell vertical movements under the evenly distributed load of 10kN/m^2 over 1/4 of the structure surface

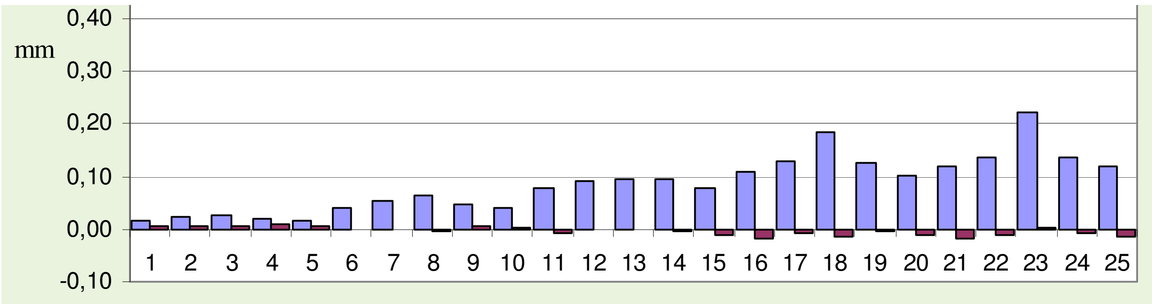
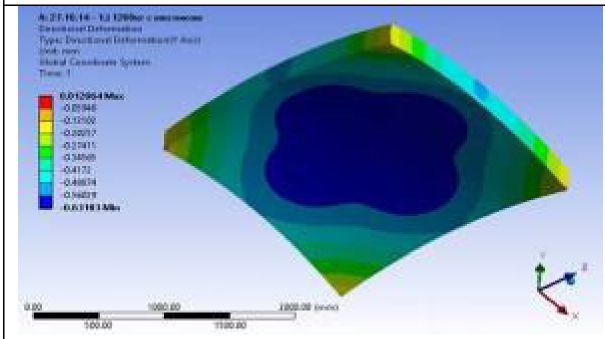
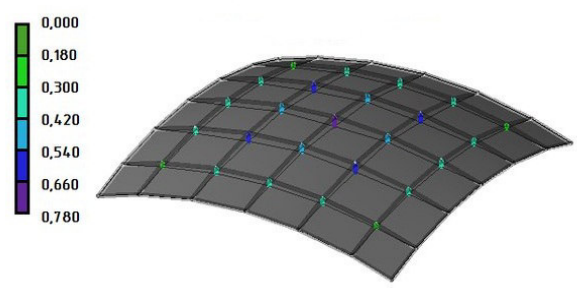


Figure 23 – Graphical representation of spherical shell vertical movements under the evenly distributed load of 10kN/m^2 over 1/8 of the structure surface

Table 3 – Comparison of theoretical and research experimental spherical shell

<table>
<tr><th>Theoretical displacements</th><th>Experimental displacements</th></tr>
<tr><td></td><td>
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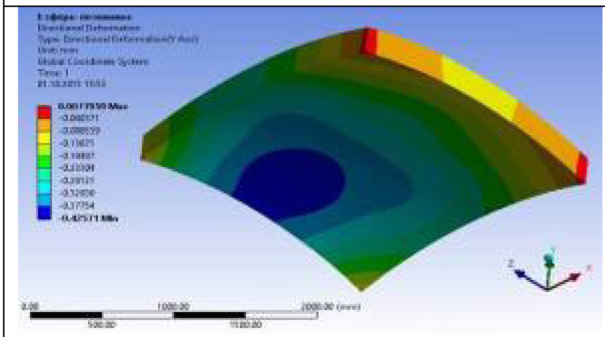
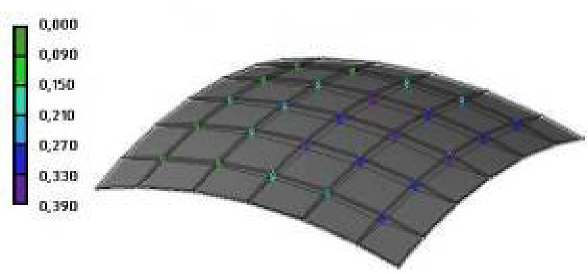
<tr><td colspan="2">a) over the entire structure surface</td></tr>
<tr><td></td><td>
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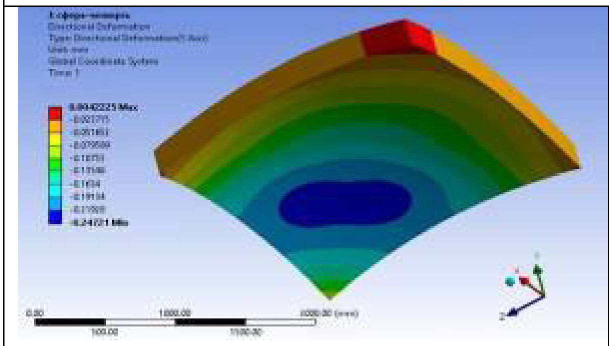
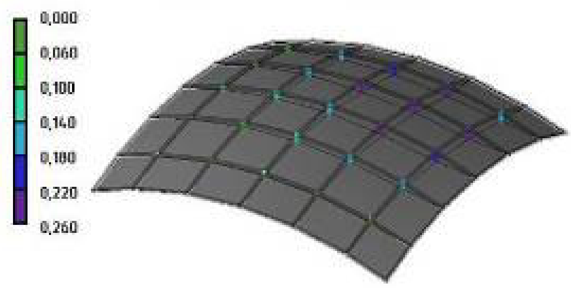
<tr><td colspan="2">b) over 1/2 of the structure surface</td></tr>
<tr><td></td><td>
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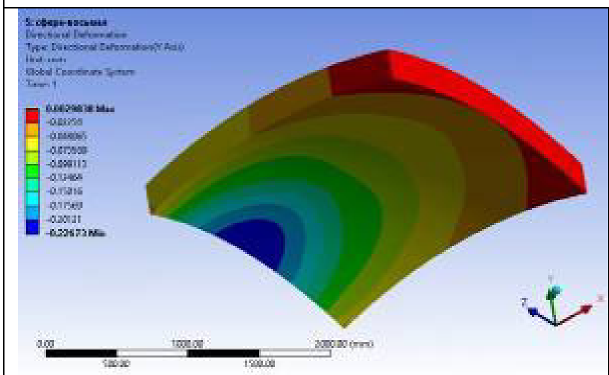
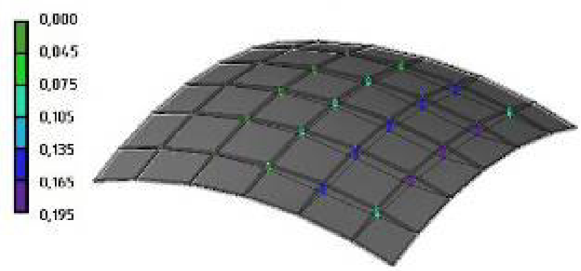
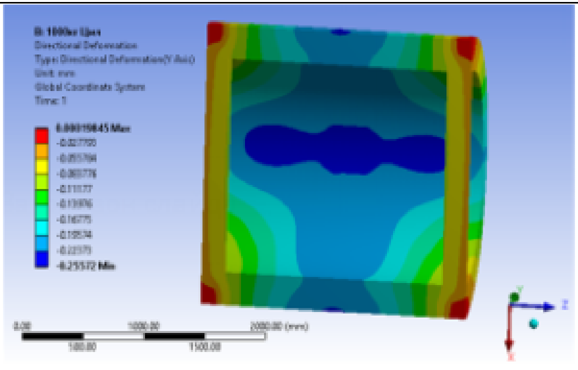
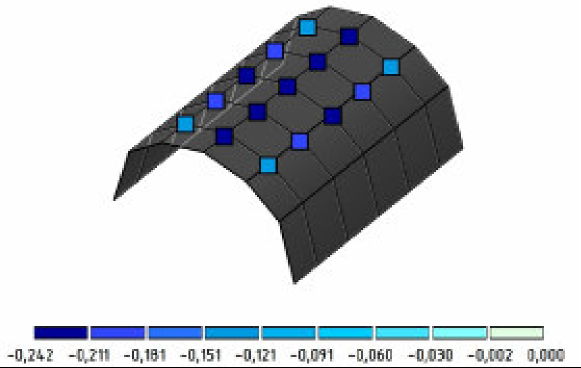
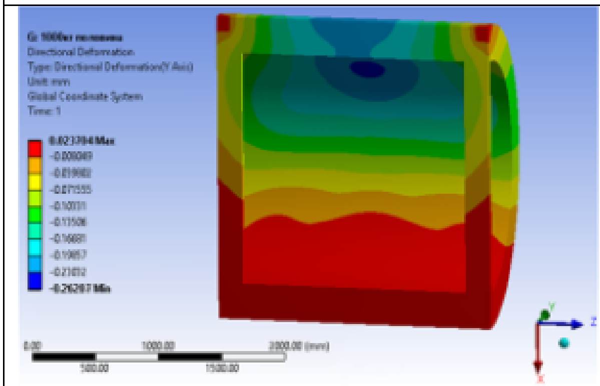
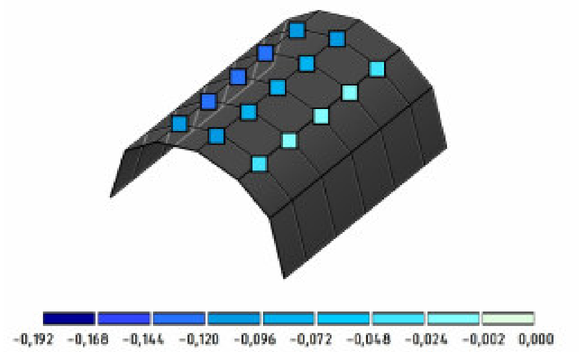
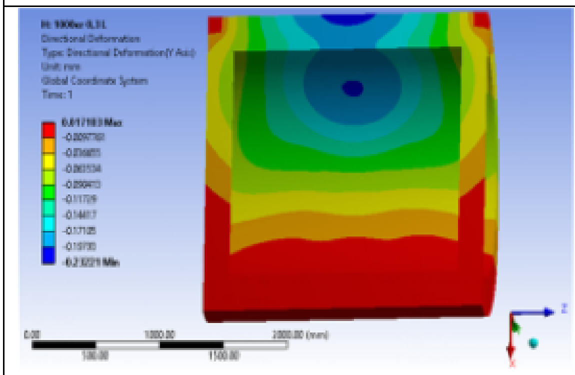
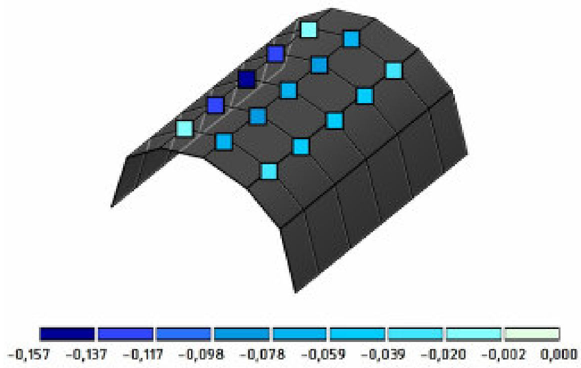
<tr><td colspan="2">c) over 1/4 of the structure surface</td></tr>
<tr><td></td><td>
</td></tr>
<tr><td colspan="2">г) over 1/8 of the structure surface</td></tr>
</table>

Table 4 – Comparison of theoretical and research experimental cylindrical shell

Theoretical displacements	**Experimental displacements**
	
a) over the entire structure surface	
	
б) **over 1/2 of the structure surface**	
	
в) **over 1/3 of the structure surface**	

Research conclusions, perspectives and further development. The following research shows that structural elements possess all necessary strength and rigidity properties of bearing members of monolithic reinforced concrete buildings.

In general it should be noted that the completed research defines «Monofant» system representativeness and opens new capabilities of cast-in-place construction that is proved by complex design in Kharkiv (fig. 24).

Figure24 – Design («Monofant» system)

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Received 02.04.2017

UDC 69.059.7:725.4 (477.53)

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FIBRE-REINFORCED POLYMER BARS IN PRECAST SLABS FOR ROADS TO OIL AND GAS EXTRACTION COMPLEXES

Considered an example of the application of fibre-reinforced polymer bars in precast slabs for temporary roads to oil and gas extraction complexes. Found that samples of products by geometric requirements similar Ferro-concrete products, but reinforced fibre-reinforced polymer bars accessories instead of metal. The results of replacing metal fittings on the fibre-reinforced polymer bars in the experimental samples. Given the comparative assessment of the conformity of prototypes of requirements on indicates the ability and crack resistance from. Found that fibre-reinforced polymer bars valves may be used in the construction of a prefab temporary roads without reducing their carrying capacity. It is proven that the use of this rebar for reinforcement of structures that work on resilient basis, both at the stage of manufacture and operation. It is shown that the resulting experience can be used in the planning and in the design, manufacture and test prototypes, and the analysis of the obtained results allow you to identify opportunities for the implementation of this direction in Ukraine.

Keywords: *fibre-reinforced polymer bars, road slab, modulus of elasticity, crack resistance.*

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ПОЛІМЕР-КОМПОЗИТНА АРМАТУРА В ЗБІРНИХ ПЛИТАХ ДЛЯ ДОРІГ ДО НАФТОГАЗОВИДОБУВНИХ КОМПЛЕКСІВ

Розглянуто приклад застосування полімер-композитної арматури в збірних плитах для тимчасових доріг до нафтогазовидобувних комплексів. З'ясовано, що дослідні зразки виробів по геометричним параметрам аналогічні залізобетонним виробам, але армовані полімер-композитною арматурою замість металевої. Наведено результати заміни металевої арматури на полімер-композитну арматуру в дослідних зразках. Надано порівняльну оцінку відповідності дослідних зразків вимогам по несучій здатності та тріщиностійкості. Встановлено, що полімер-композитну арматуру можливо застосувати в збірних плитах для тимчасових доріг без зниження їх несучої спроможності. Доведено, що застосування цієї арматури для армування конструкцій, які працюють на пружній основі, доцільно як на стадії виготовлення так і при експлуатації. З'ясовано, що отриманий досвід можна використовувати при плануванні досліджень та проектуванні, виготовленні і випробуванні дослідних зразків, а одержані результати проведеного аналізу дають змогу визначити можливості для впровадження такого напряму в Україні.

Ключові слова: *полімер-композитна арматура, плита для доріг, модуль пружності, тріщиностійкість.*

Introduction. On the present development of oil and gas extraction requires the reduction of cycle construction drilling, speeding up putting them into operation and reducing the cost of the construction of the wells. One of the components, which significantly affects the data indicators – reduction of time for installation of temporary access roads and roads and reducing their cost, through the use of modern materials.

Commissioning of new wells objectively due execution of drilling in new areas, necessitating the rebasing of drilling companies with drilling one district to another. Among new areas, yak rule, anew create a number of production objects to which it is necessary to lay new driveways and roads.

To speed up the commissioning of drilling rigs importance is performing a compulsory minimum training of construction works on the new platforms [1]. The issue of the rapid installation of temporary access roads and highways might solve thanks to the use of precast slabs using non-metal composite rebar, which is significantly cheaper for the metal. In addition, the replacement of the metal fittings on the nemetalevu removes the possibility of damage to the structures as a result of corrosion of the steel and the destruction of the protective layer that allows you to reduce operating costs. These boards have successfully can be reused when the development of new areas for drilling.

Analysis of recent studies and publications sources. Application of non-metal composite rebar construction designs is a progressive direction in the construction [2 – 8]. However, the application of polymer-composite cable and significantly limits the number of flaws and features composite fittings that do not allow direct replacement of reinforcement in composite:

– low modulus of elasticity;
– lower fire products, reinforced composite reinforcement;
– low strength at transverse loads;
– the difficulty in making bent rebar products;
– the complexity in the manufacture of prestressed structures.

One of the perspective directions of use of non-metal composite rebar is the reinforcement of structures that work on resilient basis.

In 2014-2015 by specialists of the Kazan State Architectural Construction University together with JSC «Tatneft» was working on the introduction of fibre-reinforced polymer bars for reinforcement of prefabricated elements, which are used for the extraction of petroleum-slabs, the slabs and walls under machines-rocking, plate under drives cepnì (fig. 1, a, b) [9 – 11].

a b

Figure 1 – Beam and slab under machines-rocking
a *– Beam under machines-rocking:* ***b*** *– Plate under machines-rocking*

The nature of such products is virtually the same, they will be subject to a uniformly distributed load and work on resilient basis. With the possibility of applying developments in Ukraine, the most important is to work on the introduction of polymer-composite rebar for reinforcement of precast slabs for temporary roads to oil extraction.

Parts of general problem unsolved before. At present in Ukraine not manufactured at industrial level and does not apply to prefabricated slabs using non-metal composite fittings. This occurs as a result of insufficient awareness about innovative projects that exist in the world and the lack of development in this direction.

Problem formulation. The main purpose of this work is the analysis of the results of research on the application of polymer-composite rebar at armuvannì prefabricated panels for temporary roads and estimation of prospects of development of this direction in Ukraine.

The basic material and results. Opitnij sample team plates for temporary roads 2P 30.18-30 using polymer-composite fittings have been, manufactured and tested [9 – 11]. This plate is similar to reinforced concrete plates, given the fact that its geometric parameters are accepted according to [12]. In this case the metal fittings used in reinforced concrete slabs according to [13], was replaced by a polymer-composite on the criterion – of the strength longitudinal.

It was also complied with the preservation of the total number of cores and save the locations of them in opalubcì. The process of reinforcement plate shown in figure 2.

Given the great corrosion resistance of composite reinforcement, protective layer top and bottom plate was taken 20 mm instead of 30 mm. The class is concrete and technology of concreting was performed by [13] without changes. On the basis of measurements at the load plate was built the schedule according to the «load-your backbends». Fixing strain was used to load 11 t., thus destroying the plates with a load of 14 t took place [10].

Figure 2 – Reinforcing plate

With a load of 11 t the magnitude of deflection equal to 50 mm, indicating significant your backbends plate. Raised deformativnìst′ plates under load and the occurrence of cracks in concrete structure due to the zone stretched polymer composite, namely low elasticity module. This fact is compensated for by corrosion resistant polymer-composite reinforcement and the occurrence of cracks in concrete will not cause corrosion destruction of structures, as in the case with metal reinforcement.

As a result of the tests established that the plate 2P 30.18-30 (fig. 3, b) meets the criteria of strength – by controlling stress fracture plates. Plate 2P 30.18-30 also meets the criteria of crack resistance from (fig. 3, a) when controlling the load plate width of the disclosure of the cracks smaller than 0.2 mm.

a b

Figure 3 – Test plates 2P30.18-30
a *– Test the load on crack resistance from 3,8 t,*
the width of the disclosure of the cracks smaller than 0.2 mm;
b *– Test the load on strength of 7,0 t, the destruction of the plates do not happen*

The results confirmed the possibility of polymer-composite rebar in the construction of temporary roads without reducing their carrying capacity. As a result of technical-economic evaluation of applying polymer-composite reinforcement in the slabs found that the cost of manufacturing reinforcing mesh is reduced by 12%, thus decreasing labor costs on the reinforcement plates as a result of significantly lower weight of reinforcing products from polymer-composite cable compared with metal. In addition, concrete slabs, reinforced polymer-composite accessories have increased corrosion resistance and consequently increased the term of operation. Thus the application of polymer-composite rebar in the construction of temporary roads economically feasible as at the stage of manufacture and operation.

To implement this direction in Ukraine need to perform research, similar to the above, with the use of regulations, the existing in Ukraine. In Ukraine at the present time operates a normative document regarding the design and manufacture of concrete structures with composite with accessories [14], which contains information about the scope of application, General provisions of design elements with composite reinforcement, structural requirements for these elements. In this document these characteristic parameters for composite reinforcement with links to [15, 16] for the reinforcement of concrete structures with basalt fixtures and accessories based on glass rovìngu respectively. But there is no documentation (DSTU) on requirements to composite cable, there are no common technical terms. No standardized methods of calculation of concrete of composite reinforcement, methods for determining the minimum percent reinforcement. Not enough studied experience of operation of composite products. Given the above indicated, a very important and urgent task is to develop recommendations regarding the practical application of polymer-composite reinforcement in precast slabs for temporary roads. It is advisable in this case to use the world experience [8, 11, 17 – 19].

Work in this direction was launched in Poltava National Technical University named after Yuri Kondratyuk – were conducted experimental research of mechanical characteristics of skloplastikovoï fittings. The characteristic of the composite samples of the fittings listed in table 1. Skloplastikova fittings in accordance with the provisions of [14] is a nemetaleva composite fittings, produced on the basis of continuous glass rovìngu. According to tests, the characteristic values of resistance stretching skloplastikovoï rebar is 500 MPa, and the modulus of elasticity is 33 000 MPa (average value of the three samples). According to table 6.2 [14] the characteristic value of the resistance to stretching for composite reinforcement based on glass rovìngu should be equal to 600 (800) MPa, and the modulus of elasticity is 50 000 Psi while steel must meet the requirements [16]. For comparison, table 2 shows the mechanical characteristics of skloplastikovoï fittings, which were obtained in an experimental trial and what the steps in the how to document [14]. Relationship between straight deformation of composite reinforcement and tensions are in figure 4.

Table 1 – Characteristics of composite rebar

The name	Geometric parameters	The mass of the 1 p. m., g	The density of the g/cm³	The amount of binder, %	The degree of polymerization, %
FCS	Ø 5,0mm A=7,065mm²	16,27	1,75	26	93

The characters in the notation of composite fittings have the following explanation:
F – fittings; C – composite; S – skloplastikova fittings on the basis of a glass rovìngu.

Table 2 – Mechanical characteristics of composite rebar

The name	FCS, DSTU [14]	FCS, sample № 1	FCS, sample № 2	FCS, sample № 3
Characteristic value of resistance strength, MPa	600 800	510	480	510
Modulus of elasticity has been worked out, MPa	50 000	30 000	35 000	34 000

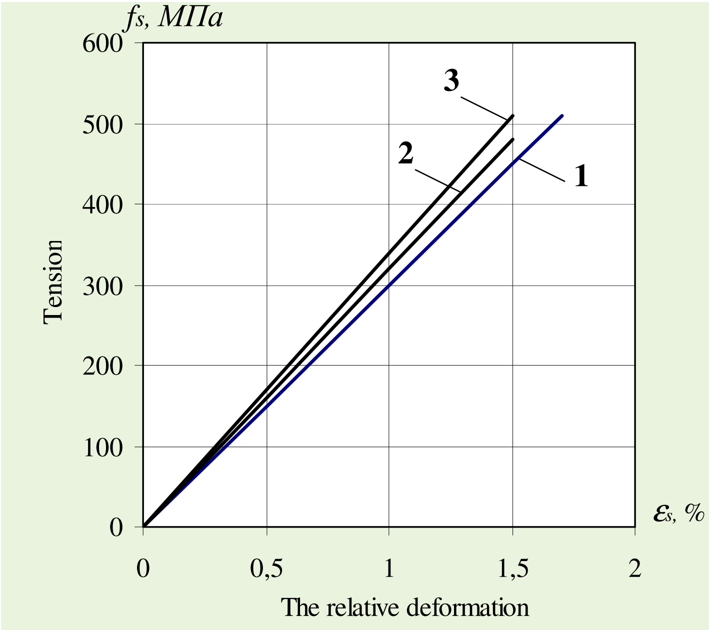

Figure 4 – Graph of dependencies between straight deformation of composite fittings and tensions
1 – sample № 1; 2 – sample № 2; 3 – sample № 3

Based on the results of the tests, was given the variability of the characteristic values of skloplastikovoï reinforcement – characteristic value of resistance strength and modulus of elasticity is significantly lower than stated in the rules [14]. The reason is that the skloplastikova steel is a rod and profile are made with continuous reinforcing glass fiber (glass rovìngu) and the termoaktivnogo binding. Physical and mechanical characteristics of such fittings depends on the fibers, percentage «fiber – binder» and production technology.

Today in the market of building materials appeared in many large and small producers who work for their own technical, mechanical characteristics of skloplastikovoï fittings vary widely.

Feature of the application of this reinforcement is the fact that it is very important to have technical specifications on each batch of composite reinforcement used in the designs. At performance of works using a composite cable, you need to determine its mìcnostnì and deformation value, and validate these values [14]. When the disparity of these values need to calculate the design for the real values of the characteristics of strength and deformability, engaging for this specialized laboratories that have the corresponding certificate. Special attention should be paid to the quality of the fittings. Not allowed: chipped, bundles, conch, gusts of winding, dents from mechanical impact damage fibers.

For the reliability of the results that will be obtained when conducting research on the use of polymer-composite rebar in the construction of temporary roads will be required to produce and test a minimum of 3 samples. Put into operation in Ukraine DSTU [20] (with the abolition of [12]) and the entry into force of DSTU [21] (with the abolition of [13]). Imposed national standards [20, 21], [12] and [13] in addition to the normative references cited in appendix A. So when planning your experiment, you can use the technique listed above in the example.

Geometric parameters, class of concrete and technology of concreting of prototypes of plates for temporary roads P-1, P-2, P-3 be taken like slabs of reinforced concrete 2P 30.18-30 [20]. In this case the metal rebar used in concrete slabs replaced by polymer-composite on the criterion of equal strength longitudinal.

By calculations using these tests (table 2), is an example of replacing the metal fittings on the polymer-composite (table 3).

Table 3 – Replacement of reinforcement for polymer-composite rebar

№	The diameter of the metal rod, mm	Diameter polymer-composite of rod, mm
1	Ø8F400	Ø6FCS
2	Ø10F400	Ø8FCS
3	Ø12F400	Ø10FCS
4	Ø14F400	Ø12FCS
5	Ø16F400	Ø14FCS

Determined that the longitudinal steel Ø10F400 in research samples P-1, P-2, P-3 can be replaced by a composite Ø8FCS, of the metal rods, that located along the short side plates Ø8F400 – on the composite Ø6 FCS. Diagram of the reinforcement of research designs P-1, P-2, P-3 take in accordance with [21, 22], while maintaining the total number of rods and their locations in the opalubcì.

Opitnih test samples necessary to carry out the static load by a technique for evaluation of their strength and compliance with regulatory requirements. In this case, the main goal of the tests is to research the destruction and deformation of plates reinforced fibre-reinforced polymer bars.

The final stage of data research is the development of technical specifications for the manufacture of prefabricated plates with the use of a composite cable.

Conclusions. The use of plates with fibre-reinforced polymer bars leads to a reduction of the cycle of building wells, their accelerated commissioning, that gives the effect in increasing production and reducing the cost of the construction of the wells.

You can use the experience gained in the planning and the design, manufacture and test prototypes, and the analysis of the obtained results allow you to identify opportunities for the implementation of this direction in Ukraine.

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Received 21.02.2017

UDC 624.012.45

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METHODS OF MODELING OF COMPOSITE MATERIALS AND COMPOSITE STRUCTURES ON «LIRA-SAPR»

This paper provides detailed suggestions for the process of structural reinforcement modeling by composite materials on the software package «LIRA-SAPR». It also provides the implementation of bearing capacity checks for reinforced elements on the program called «ESPRI». The article offers an algorithm for calculation of the construction objects in case of design situation changing, considering the modeling of the composite structure reinforcement. It considered the modeling process of reinforcement of structures using classical methods, such as using of metal casing. It also investigated a numerical modeling example of the frame structure reinforcement, with the selection and verification of the composite material.

***Keywords**: stress-strain state, composite material, strains, deformations, material nonlinearity, software «LIRA-SAPR».*

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МЕТОДИ МОДЕЛЮВАННЯ КОМПОЗИТНИХ МАТЕРІАЛІВ І КОМПОЗИТНИХ КОНСТРУКЦІЙ В ПК «ЛІРА-САПР»

У статті розглянуто процес моделювання підсилення конструкцій композитними матеріалами в програмному комплексі «ЛІРА-САПР» зі здійсненням перевірки несучої здатності підсилених елементів в програмі «ЕСПРІ». Запропоновано алгоритм розрахунку об'єктів будівництва при зміні проектної ситуації з урахуванням моделювання підсилення конструкції. Досліджено процес моделювання підсилення конструкцій металевою обоймою. Наведено приклад чисельного моделювання підсилення рами, з підбором та перевіркою композитного матеріалу.

***Ключові слова**: напружено-деформований стан, композитні матеріали, деформація, фізична нелінійність, програмний комплекс «ЛІРА-САПР».*

Introduction. While in operation of buildings and structures, if change of loads takes place, during reconstruction or when a building structural scheme changes, it becomes necessary to increase load bearing capacity of some structural elements. Therefore, the question about the bearing capacity increasing of constructions is very relevant. One of the most effective innovative methods to strengthen the reinforced concrete structures is the application of composite materials (fiber reinforced polymers (FRP)).

Analysis of the latest research sources and publications. The first composite material of modern type is deemed to be unidirectional glass-fibre reinforced plastic created by A. Burov in the 30s. In the 60s in the UK the carbon fibers were created, which provided impetus for the development of new generation of composite materials, which are highly used in the design and reconstruction of buildings nowadays. In consequence of steady increase of the interest in composite materials (fiber reinforced polymers) and composite reinforced structures a lot of works have recently been devoted to the development of this issue: many works created by Storozhenko L. [6, 7], Lapenko O. [7, 9, 13], Ermolenko D. [4], Hvozdeva A., Klyueva S., Kurlapova D. [12], Khayutina Yu. [15], Chernyavskii V. [15, 16] and others. A lot of works of Kuznetsov V., Vatin N., Hrigorieva Ya. were dedicated to the development of calculation methods of reinforced concrete structures strengthened by fiber reinforced polymers. Many works devoted to the topic were created abroad, for example, some works written by Belarbia A. [18], Acunb B., David D. [19], Grace N. [21] and others.

Identification of previously unresolved areas of a common problem. The analysis of the latest works on the considering subject showed that up to date there are a lot of works devoted to experimental researches and analysis of experimental data on the reinforced concrete structures strengthening. But, the question about the computational modelling of reinforced structures and the analysis of its stress-strain state is still quite undecided.

Problem formulation. The goal of the article is mathematical modeling of construction by the finite element method (FEM), taking into account the work of the strengthening system from composite materials. And also the creation of computer modeling of construction work before and after strengthening. The implementation of the goal was carried out on the basis of the software package «LIRA-SAPR» and using the engineer electronic reference book «ESPRI».

Main material and results. Materials that consist of two or more components or phases are called composite materials. A continuous phase is called matrix, and components are called filler or a reinforcing phase. The role of fillers is that they change the properties of the continuous phase in the desired direction. Such materials may have a polymeric, metallic or ceramic continuous phase. The mechanical properties of composite material depend on the used continuous phase (matrix). Polymer phases are characterized by relatively low strength and modulus of elasticity. Ceramic phases have high strength and rigidity, but their disadvantage is high brittleness. Metal phases are characterized by intermediate values of strength, modulus of elasticity and they are more plastic than ceramic phases.

Composite materials based on fiber, which used in reconstruction and strengthening of buildings or structures, are made of microfibers. They combined within polymer and thus are pieced. The most common types of fibers: aramid, carbon and fiberglass. The physical and mechanical properties of composite materials are determined by the type and number of fibers which are used.

Supporting of the load-bearing ability and stability of any construction is laid on the basis of computer and numerical analysis of their stress-strain state.

So let's consider a procedure of structure analysis in SP «LIRA-SAPR», taking into account the increasing of carrying capacity of its elements.

In the event of changing of the design situation (it may be the load increasing at the object, defect identification within any structural elements, changing the object's purpose or reconfiguration of the building object), first of all, the static analysis of the construction is made, taking into account the impact of the new loads and some changes within the elements. Based on the results of the static analysis the necessary reinforcement for reinforced concrete elements can be matched.

Taking into account the selected reinforcement, the analysis of the structure take into consideration the physical nonlinearity is performed. As a result of the analysis one determines damaged structural elements, in which cracks appear. Consequently elements that are in need of strengthening are determined.

At the following step of the design the parameters of the fiber reinforced polymer material are to be selected which will be used to strengthen the construction. The bearing capacity of the strengthened element could be checked in «ESPRI». As a result of the check in «ESPRI» new reduced rigidity of the strengthened element is got.

After selection of the composite material and testing the load-bearing capacity of the strengthened element, on the SP «LIRA-SAPR» the new stiffness to the finite elements of the analytic model is set. To do this, the type of the finite elements, which are in need of being strengthened, is to be changed. The new type of these finite elements must be the physically nonlinear universal three axes bar finite element №210. After setting the new stiffness (taking into account the strengthening of the construction by fiber reinforced polymers materials) a new calculation has to be made. After the evaluation of the obtained stress-strain state of the analytic model decision about the additional strengthening of the other structural elements has to be sold.

After the strengthening, in the analytic model the redistribution of the forces takes place. So if desired, in «ESPRI» it is possible to re-test the load-bearing capacity of the strengthened structural elements, with the new forces obtained on the SP «LIRA-SAPR» in these elements.

Consequently, the calculation procedure of the analytical model in the SP «LIRA-SAPR» and «ESPRI», taking into account the strengthening, can be displayed in the form of the following algorithm, which is shown in Fig. 1.

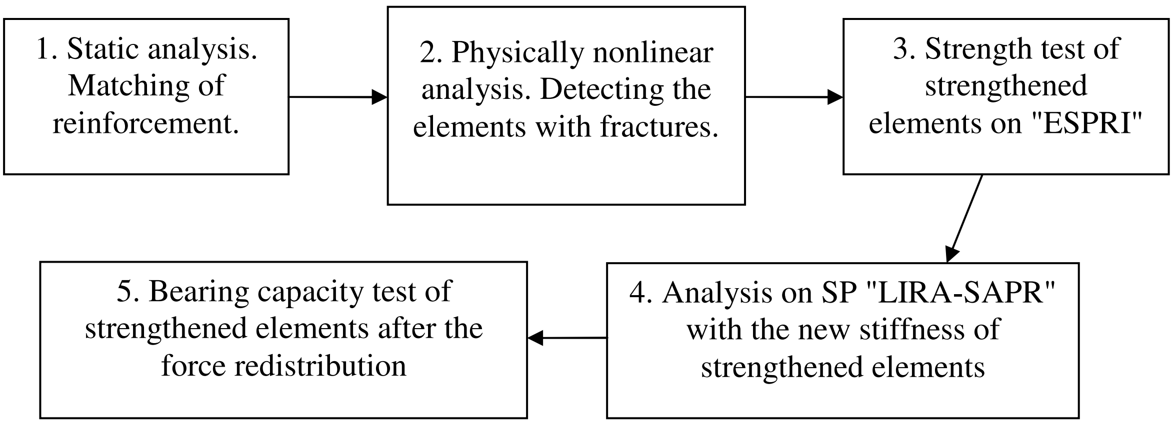

Figure 1 – The analysis algorithm of strengthened structures on SP «LIRA-SAPR».

Letus consider the procedure for obtaining the parameters to model the strengthen of a structure on SP «LIRA-SAPR» by the example of the frame analytic model (Fig. 2).

After the static analysis, for the considered analytic model the appropriate reinforcement is selected.

Then the physically nonlinear analysis was carried out, in which the corresponding parameters of the concrete deformation (Fig. 3) and reinforcement were specified.

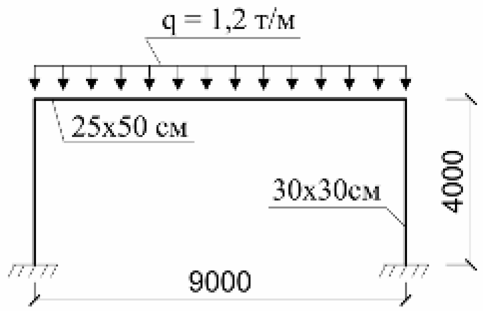

Figure 2 – Frame analytic model

Parameters	Values
Ecm(-)	3.25E+010
Ectm(+)	2.9E+010
fcm(-)	-38000000
fctm(+)	2600000
Epscu(-)	-0.0035
Epsc(-)	-0.0022
Epsctu(+)	0.00012

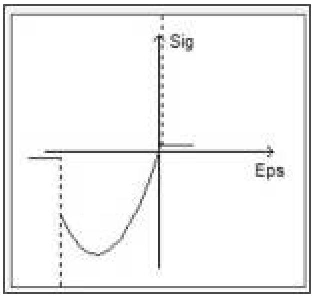

Figure 3 – Characteristics of nonlinear concrete deformation, Pa

During the physically nonlinear analysis the parameters to calculate the deformation of the structure, taking into account creep of concrete are specified. The change of the creep coefficient in time was determined with the 44th piecewise creep law.

The creep coefficient $\varphi(\tau)$ was calculated with formulas:

$$\varphi(\tau) = \varphi(t')\, f(t - t'), \tag{1}$$

$$\varphi(t') = C_0 + \sum_{k=1}^{m} \frac{A_k}{(t')^k}, \tag{2}$$

$$f(t - t') = \sum_{k=0}^{m} B_k e^{-\gamma_k (t - t')}, \tag{3}$$

where t is a point of time for which deformation is determined;

t' is a point of applying the stress elementary increment;

B_k and γ_k are constants which depend on the concrete, and also $B_0 = 1$, $\gamma_0 = 0$ and $\gamma_k > 0$.

The value C_0 is the extreme value of the creep coefficient; A_k is characteristic, which depends on the properties and ageing conditions of the using concrete.

After the physically nonlinear analysis the schemes of the stress-strain state of the frame is received (Fig. 4).

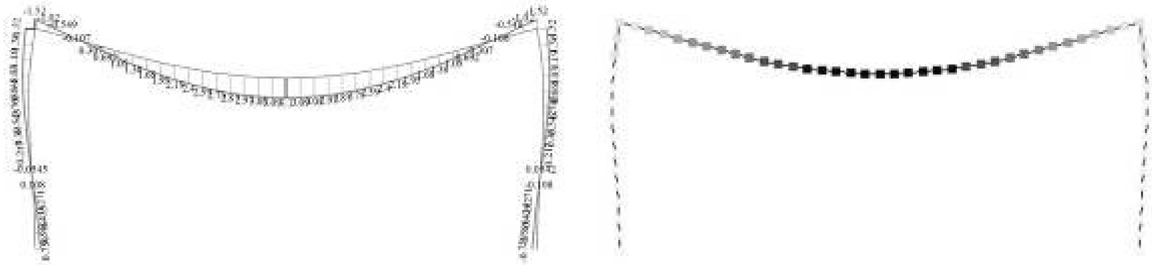

Figure 4 – Stress-strain state schemes

As a result of the analysis, it was determined that on the third stage of creep deformation manifestation the cracks appear in some elements of the analytic model. Figure 5 shows the diagrams of stresses and strains within the cross-sections of the beam and columns, before and after appearance of the cracks.

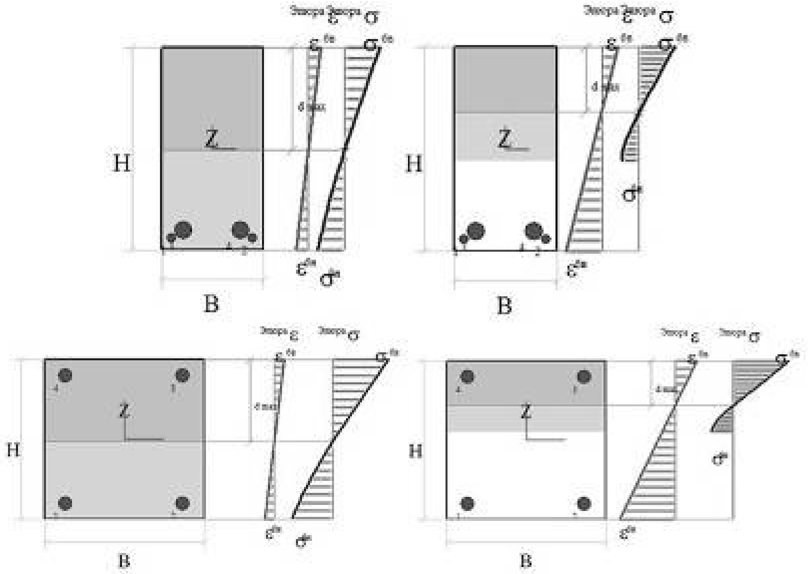

Figure 5 – Stress-strain diagrams within the crosss-sections of the analytic model

To strengthen the frame elements in which cracks appeared, we selected the fiber reinforced polymers Aslan 400 CFRP Laminate with the following stiffness characteristics is selected:

– monolayer thickness: 1,4 mm;
– modulus of elasticity, E: 131000 MPa;
– strength of a material: 2400 MPa;
– deformation at rupture: 0.0187 %.

The work condition coefficient was assumed equal to unity.

After selection the composite material for strengthening, on «ESPRI» the load-bearing capacity of the strengthened frame element was tested.

For strengthened elements which work on compression, the load-carrying capacity check is performed using the following formulas:

– when strengthening by external reinforcement longitudinally:

$$Ne \le f_{cd}bx(d-0{,}5x)+f_{scd}A_s'(d-a')+f_{fd}A_f a\,, \tag{4}$$

– when strengthening by external reinforcement edgeways:

$$Ne \le f_{cd3}bx(d-0{,}5x)+f_{scd}A_s'(d-a')\,. \tag{5}$$

For elements that work on bending, the check is performed on a bending moment:

– for rectangular cross-sections:

$$M \le f_{cd}bx(d-0{,}5x)+f_{scd}A_s'(d-a')+f_{fd}A_f a\,, \tag{6}$$

– for T-section, if the compressive zone bound is through the girder web:

$$M \le f_{cd}bx(d-0{,}5x)+f_{cd}(b_f'-b)h_f'(d-0{,}5h_f')+f_{scd}A_s'(d-a')+f_{fd}A_f a\,. \tag{7}$$

In the formulas (4), (5), (6) and (7) the following symbols are used:

f_{cd} – design value of the cylinder compressive strength of concrete,
f_{cd3} – axial design value of the cylinder compressive strength of concrete,

f_{scd} – design value of the compressive strength of reinforcing steel,

f_{fd} – design value of the cylinder strain strength of fiber reinforced polymer,

d – cross-section effective depth,

A_f – cross-section area of fiber reinforced polymer reinforcement,

$h_{f'}$ – depth of the flange of a T-bar (I-bar) cross-section in the compressive zone,

$b_{f'}$ – width of the flange of a T-bar (I-bar) cross-section in the compressive zone,

b – cross-section effective depth.

After the strengthening material matching and the strength test of the reinforced structural elements, the reduced rigidity of the strengthened elements is obtained. To calculate the analytic model, taking into account the new stiffness, the FE №10 with the FE №210 is replaced, which was given the new reduced stiffness characteristics.

One of the most common classic variants to increase the bearing capacity of the structure is the increasing of its rigidity, by installing metal casing. Let us consider a calculation example of the strengthening of a column of the considered frame, using the metal casing. To do this, instead of selecting composite materials, choose the dimensions of the metal angle bar (or plates) with which should be to strengthen the elements of the structure. The verification of the strengthened elements whish also are implemented on the «ESPRI» program. For this purpose, in the subroutine «Check of the steel concrete column cross-sections» (section Reinforced Concrete Structures) it is necessary to choose a check of the steel concrete cross-sections with metal angle bars and set the parameters of the angle bars in such way that they model the metal casing around the column.

The check of the composite cross sections could be carried out by limit states and also by deformation model, as well as by the two-line and three-linear deformation diagrams which are represented in figure 6.

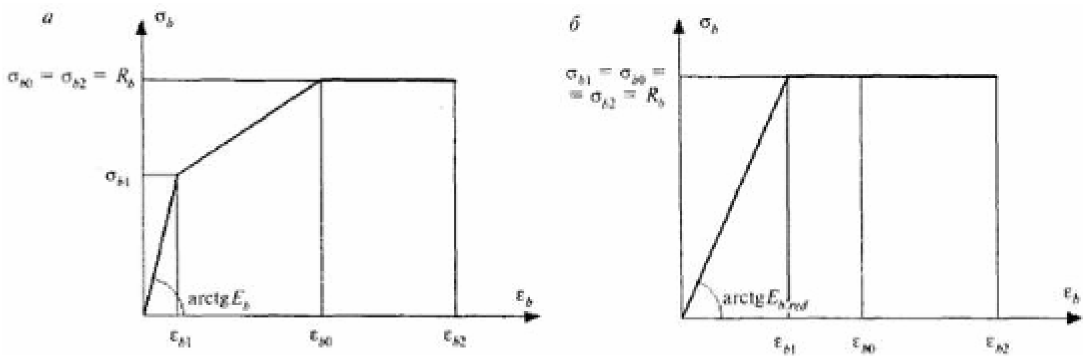

Figure 6 – Deformation diagrams of steel concrete elements.

Comparison of the frame stress-strain state at different design stages, taking into account the strengthening, is shown in Table 1.

Table 1 – Comparison of efforts and displacements of the frame at various formulation of the problem

	Lineal elastic calculation		Physical nonlinearity		Together with the strengthening	
	Disp-lacements Z, mm	Efforts M_y, t*m	Disp-lacements Z, mm	Efforts M_y, t*m	Disp-lacements Z, mm	Efforts M_y, t*m
Column	0	±1,65	-0,1	±1,89	0	±2,06
Beam	-2,8	2,96	-7,6	2,7	-7,3	2,26

Also, on the program «ESPRI» it is possible to calculate other cross sections of the steel concrete structures, which examples are shown in Fig. 7.

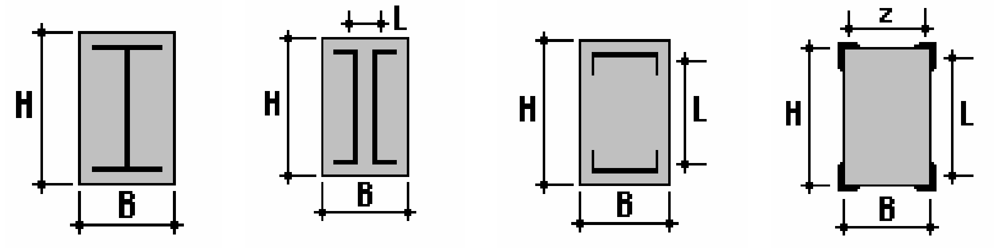

Figure 7 – Cross-section types of steel concrete structures which could be calculated on the "ESPRI" program

Conclusion. Usage of fiber reinforced polymers to strengthen structures allows significantly to increase the load-bearing capacity of the elements of buildings and structures. It also allows to extend their service life, prevent or eliminate the emergency situation, correct construction or design mistakes, and the most important that it allows to ensure reliable operation and durability of the structures.

In this paper, the model technique for strengthen structures by fiber reinforced polymer materials is proposed. The variant of modeling the strengthening of a construction by metal casing is considered. The values of the element stress-strain state within the analytic model are obtained.

The result of the research is the evaluation of the stress-strain state of the structure while simulation its strengthening by fiber reinforced polymer with the physically nonlinear formulation of the problem.

The results of this work can be used for wider application to increase the bearing capacity of buildings and structures.

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Received 30.11.2016

UDK 624.071.34

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SPATIAL STRUCTURES OF CLOSED PROFILES

The article presents a new design of steel space trusses, arches, frames of complex application, the main advantages of which are lightness, durability of plane, cost efficiency, reliability, reduction of construction period and investment cycle, energy savings at manufacture, transportation and construction. Based on the previously performed experimental and theoretic studies, the expediency of using box-shaped section parallel chord trusses has been proved. Therefore, new constructive decisions for light efficient trusses, arches, frames that can be used at erection of civil and industrial objects are presented below. The suggested efficient structure of a steel truss with spatial upper chord, using tubes or rectangular hollow box-shaped sections, steel trifurcate spatial truss construction, spatial arch truss, spatial arch. Considered the use of closed cross-sections for spatial arch elements. It has been revealed that using the two-hinges spatial frame the construction cost of a building can be reduced due to reducing of the basement's dimensions.

***Keywords:** steel truss, arch, frame, box-shaped section.*

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ПРОСТОРОВІ КОНСТРУКЦІЇ ІЗ ЗАМКНЕНИХ ПРОФІЛІВ

Подано нові ефективні сталеві просторові конструкції ферм, арок, рам комплексного застосування, переваги яких полягають у легкості, стійкості щодо площини, економічності, надійності, скорочення строків будівництва й інвестиційного циклу в цілому, економії енерговитрат при виробництві, транспортування та зведення. На основі проведених раніше експериментальних та теоретичних досліджень доведено доцільність використання ферм з коробчастими перерізами з паралельними поясами, наведено нові конструктивні рішення легких ефективних ферм, арок, рам, які можна застосовувати в будівництві об'єктів цивільного та промислового призначення. Запропоновано ефективні конструкції сталевої ферми з просторовим верхнім поясом з використанням труб або замкнених коробчастих перерізів, сталевої тригілкової просторової ферми, арочної просторової ферми та просторової арки. Розглянуто застосування замкнених перерізів для просторових арочних елементів і просторових перерізів із замкнених профілів для рамних конструкцій. Доведено, що при використанні варіанта двошарнірної просторової рами можна зменшити вартість споруди за рахунок зменшення розміру фундаменту.

***Ключові слова:** сталева ферма, арка, рама, коробчастий переріз.*

Introduction. In the design of steel span structures of civil and industrial destination, spatial steel structures have lately acquired prevalence due to their substantial advantages in comparison with the traditional ones. The main advantages of the above type constructions are: considerable reduction of the buildings' weight, erection durability and simultaneous saving of operation costs. In general, the construction decisions efficiency is aimed to obtain the most feasible indices of material consumption, manpower efforts and capital intensity of construction. Efficiency indices can be raised due to light efficient box-shaped sections and new contours of trusses, arches and frames with space lattice.

At the current stage of the metal structures market development, ever more popularity is being gained by new types of cross-sections, such as curved box-shaped, profiled and perforated sections. Therefore, popularization of such structures is inducing development of efficient constructive elements and the respective standard regulations for calculation of the given type constructive forms. At present, under the conditions of the nationwide resource saving program, the topical issue is designing new constructive forms and structures' contours, using present-day efficient materials, raising their corrosion resistance and durability. In the building construction industry there is a necessity of using innovative technological developments at erection of public, administrative, storage and industrial buildings and structures.

Latest sources of studies and publications review. The integrated assessment and systematization of the existent building structures referring to the said type of the civil and industrial objects has been the subject matter of the research papers [1 – 6]. Results of the authors' research, namely combined arch-core elements in flat structures, are described in [7]. Patent projects, where the structures under study consideration has been started, are presented in publications [8, 9]. Foreign research developments, presenting a large number of light metal structures, such as trusses, arches, frames, and specifying the advantages and faults of the above type structures, are referred to in the following papers [10 – 15].

Part of the problem not solved before is the fact that at the fast rates of building construction development in our country, the demand of searching new efficient, high-tech, resource saving constructive forms and cross-sections of light metal structures.

Aims of the study. Based on the foreign and national experience of using light steel structures, aim of the study has been formulated as: development of new constructive decisions for trusses, arches, frames, and determining their main advantages compared to the traditional decision variants.

Basic material presentation. Based on the previously performed experimental and theoretic studies [7], the expediency of using box-shaped section parallel chord trusses has been proved. Therefore, new constructive decisions for light efficient trusses, arches, frames that can be used at erection of civil and industrial objects are presented below. The suggested efficient structure of a steel truss with spatial upper chord, using tubes or rectangular hollow box-shaped sections (Fig. 1, a) [9].

Structural peculiarity of the above trusses lies in the fact that the truss's upper chord is made trifurcate, thus providing its stability both in the plane and out of the truss's plane (Fig.1, b, c). In all the cases, the cross-sections are made of tubes or rectangular profiles. The distance between the chord's branches makes 1/6 – 1/8 of the truss's height and is determined by the calculation of upper chord's stability. The truss's height depends on the parallel chord trusses' span and provides the necessary stiffness of the structure.

As the following stage of using the rectangular hollow sections, let us consider the steel trifurcate spatial truss (Fig. 2). Efficient operation of this structure is determined by the truss's spatial rigidity both in the plane and out of the truss's plane. Disengagement of the truss's upper chord with joists, on which a profiled decking is mounted, forming a rigid decking disk. The lower chord of a spatial truss is disengaged by a system of braces, providing the trusses' stability out of the plane. Using rectangular cross-sections is technologically more expedient than using round tubes that require additional equipment for shape-cutting. The above described structures cover a definite surface and are more economically efficient when comprehensively used for covering. Spatial trusses have less construction height than flat ones that sometimes sufficiently reduces the construction costs.

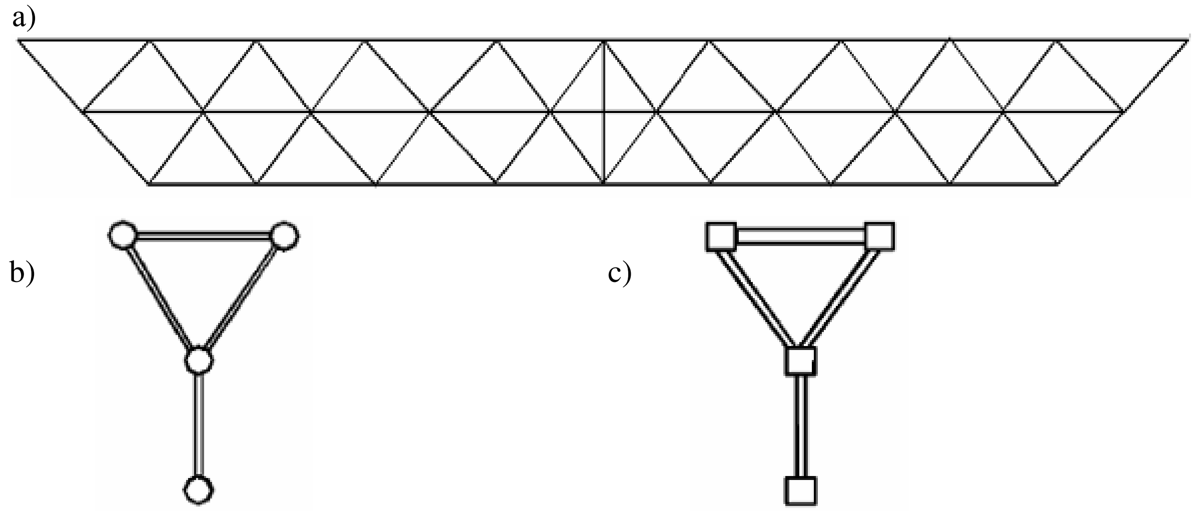

Figure 1 – Truss with a spatial upper chord:
a) diagram of a truss with a spatial upper chord;
b) cross-section of trusses with a spatial upper chord made of tubes;
c) cross-section of trusses with a spatial upper chord made of rectangular profiles

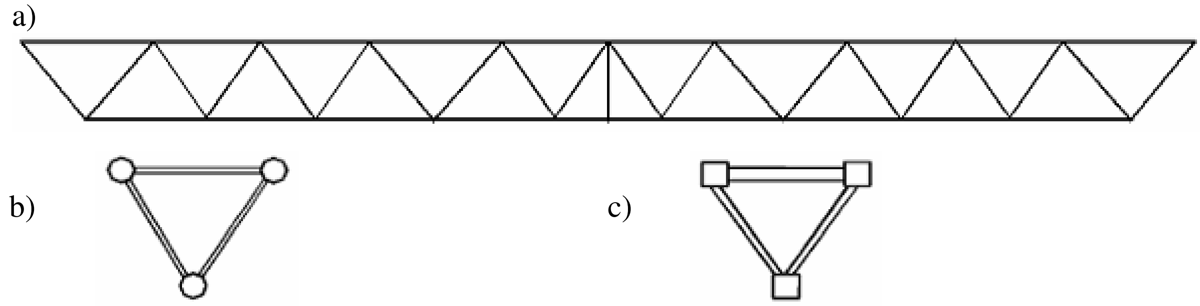

Figure 2 – Spatial truss made of closed hollow profiles:
a) diagram of a truss made of closed hollow profiles;
b) truss cross-sections made of tubes;
c) truss cross-sections made of rectangular profiles

Let us consider the use of closed cross-sections for spatial arch elements. Figure 3, a describes an arch spatial truss and a spatial arch (Fig. 3, b). As to the metal consumption, arch covering is more economizing than the beam systems.

In the course of the study performed it has been determined that the most expedient height of the arch lies within 1/4 – 1/6 of the span. The arch contour should maximally correspond to the pressure curve. The pressure curve in the arch has a parabolic contour due to the persistent pressure, therefore, the most frequent arch form is recognized to be parabolic. However, for ease of manufacture, arch elements are often delineated in a circular arc. In flat (depressed) arches the circular arc is almost coinciding with the parable, whereas in higher arches it is reasonable to substitute the parable with a combination of circular arcs of different radii.

a)

b)

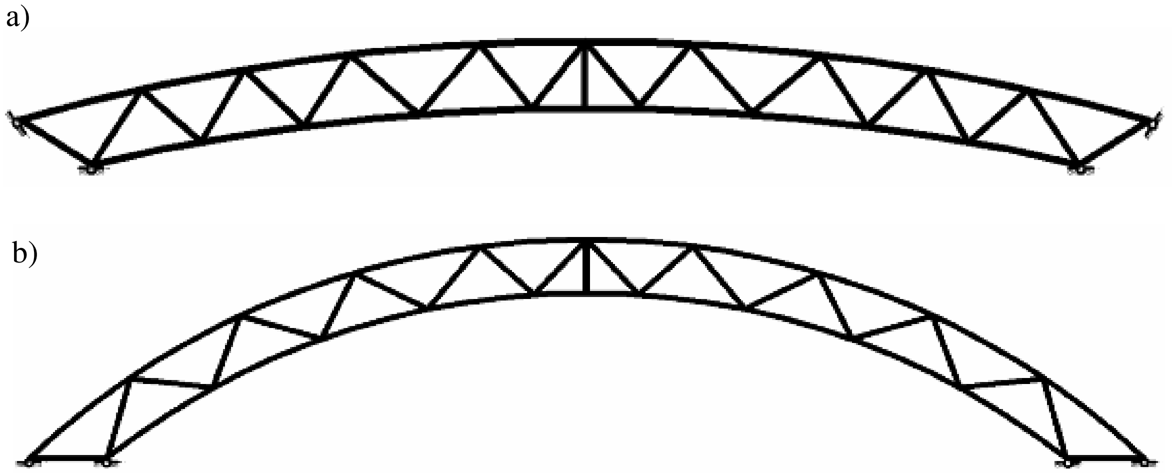

Figure 3 – Steel spatial arch structures:
a) spatial arch truss; b) spatial arch

The arch section's height depends upon the span and the ratio of the dead and live load values and is assumed for braced arches within 1/30 – 1/60 of the span, for solid sections it makes 1/50 – 1/80 of the span.

Spandrel aches cross-sections are recommended to be of a fixed height, which is meeting the nature of the efforts length changes to the fullest extent. Meanwhile, in many cases, variable height cross-sections are used, for example, crescent type arches in two- and three-hinged arch covers.

Using spatial sections of closed profiles for frame structures will be considered in the context of traditionally contoured hingeless (Fig. 4, a) and two-hinges frames (Fig. 4, b).

a)

b)

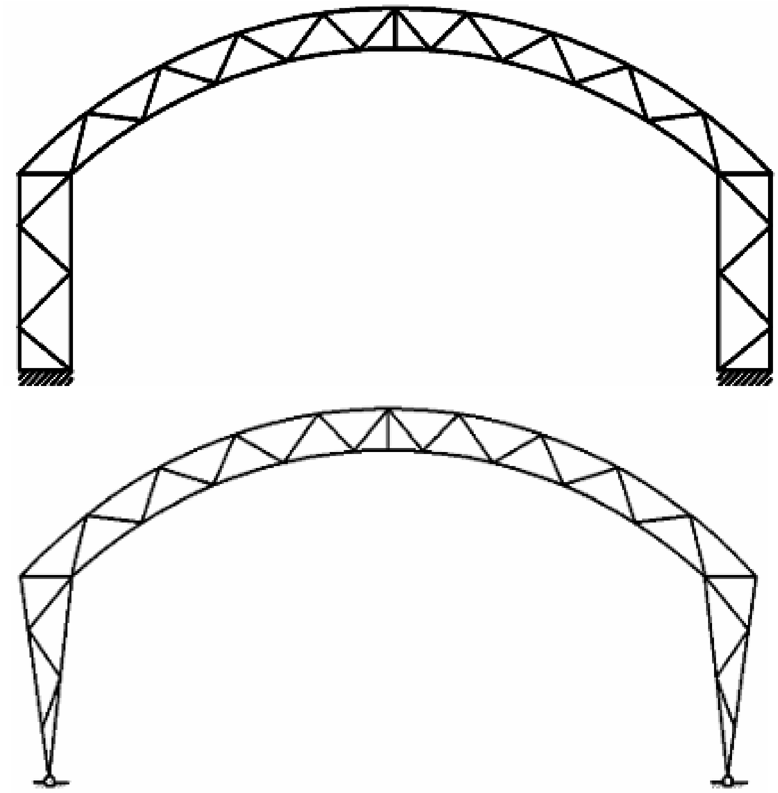

Figure 4 – Steel spatial frame structures:
a) hingeless spatial frame; b) two-hinges spatial frame

The trifurcate system's spatial behaviour provides rigidity in the plane and out of the frame's plane. The main advantages of the frame covers compared to the beam ones are: less weight, high rigidity and less frame beam's height. Frame beams' sections are mostly designed built-up for spans up to 60 m long, particularly when the frame's beam has a broken contour. Frame structures are efficient when the columns' and beams' rigidity is equal, thus permitting to redistribute the vertical loads' efforts and to considerably lighten the beams: in this case the braced beam's height can be assumed equal to 1/12 – 1/20 of the span.

Manpower effort's growth at spatial structures manufacturing is exceeded by their reduced material consumption thus giving the possibility of obtaining more economizing structures.

As a result of the performed study, it has been revealed that using the two-hinges spatial frame (Fig. 4, b) the construction cost of a building can be reduced due to reducing of the basement's dimensions.

Conclusion. The suggested new constructive decisions for steel spatial trusses, arches and frames, having the properties of high load bearing capacity and architectural expression, minimize material and manpower expenses. The described type structures demonstrate the increased indices on the general stability of separate elements and of the whole system both in the plane and out of the plane.

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Received 25.10.2016

UDS 624.014:697.112.3

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PROBABILISTIC ANALYSIS OF THERMAL PERFORMANCE OF THE WALL FROM LIGHT-GAUGE THIN-WALLED STEEL STRUCTURES

The article is devoted to determining of the thermal reliability rates of CFS wall panels based on three thermo-technical failure criteria - reduced heat transfer resistance, exceeding the values of the temperature difference between the reduced temperature of the inner surface of structure and internal air temperature above the permissible temperature values by the sanitary requirements and criteria of reduction of local values of the inner surface temperature to the temperature of vapor-liquid condensation.

With increasing coefficient of variation of thermal conductivity from 2.28% to 20%, the probability of refusal of wall panels of light steel thin-walled structures, under the criterion of the specified heat transfer resistance, is increasing from $9{,}85\times10^{-7}$ to 0,015 respectively.

***Keywords:** light-gauge thin-walled steel structures, wall, thermal conductivity, linear heat transfer coefficient, thermal resistance, thermal reliability.*

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ІМОВІРНІСНИЙ АНАЛІЗ ТЕПЛОТЕХНІЧНИХ ПОКАЗНИКІВ СТІНИ З ЛЕГКИХ СТАЛЕВИХ ТОНКОСТІННИХ КОНСТРУКЦІЙ

Розглянуто визначення показників теплової надійності стінової панелі із легких сталевих тонкостінних конструкцій (ЛСТК) при зміні фізичних характеристик за трьома теплотехнічними критеріями відмови – приведеним опором теплопередачі, перевищенням значень перепаду температур між приведеною температурою внутрішньої поверхні конструкції та температурою внутрішнього повітря над значеннями температури, допустимими за санітарно-гігієнічними вимогами, та за критерієм зниження локальних значень температур внутрішньої поверхні до температури конденсації пари повітря.

З'ясовано, що ймовірність відмови стінової панелі із ЛСТК за критерієм приведеного опору теплопередачі зі збільшенням коефіцієнта варіації теплопровідності від 2,28 до 20% підвищується від $9{,}85\times10^{-7}$ до 0,015 відповідно.

***Ключові слова:** легкі тонкостінні конструкції, стіна, теплопровідність, лінійний коефіцієнт теплопередачі, опір теплопередачі, теплова надійність.*

Introduction. One of the important factors of economic growth in Ukraine is the development of energy saving and energy efficiency in the construction. Reducing the costs of heating buildings is possible by using both renewable energy and increasing requirements for enclosing structures. One of the points of energy efficiency measures for the household sector is the improvement of building regulations and standards (including the provision of annual increase in the number of newly constructed buildings close to zero energy consumption). Considering that Ukraine has raised the standards for enclosing structures almost for two times [1, 2], while the number of temperature zones decreased from 4 to 2, there is quite acute problem of thermal reliability for buildings that are in the same temperature zone, but geographically located at great distances (from north to south).

Enclosing structures of frame buildings of light steel thin-walled structures have many heat-conducting inclusions, through which there are significant heat loss of the building as a whole. However, many designers in the deterministic calculations do not include these inclusion and offer only constructive solutions of eliminate the influence of thermal bypass to enhance of thermal reliability. At the same time Enclosing structures is calculated as thermally homogeneous. This may have a negative effect to frame buildings of light steel thin-walled structures.

Review of recent research and publications sources. The concept of thermal reliability in Ukraine was first introduced by G. Fareniuk. According to [3], thermal reliability is a property of the object (enclosing structure) to store in time within the established value of parameters which characterizing the ability to perform necessary functions in specified modes and conditions of use. In other words, it must maintain its thermal performance within permissible limits given in the lifetime of the building. To solve the problem of evaluation of thermal reliability of enclosing structures G. Fareniuk proposed four conditions of thermal failures [4].

In the future research area was continued by V. Pashynskyi [5]. By his participation was made DBN V.1.2.-14-2009: «System to ensure reliability and safety of building objects. General principles of ensure reliability and structural safety of buildings, building structures and foundations» [6]. In it noted that the reliability of construction object is a property of the object to perform specified functions within a specified period of time. Methods for numerical evaluation of the probability of failure of similar enclosing structures were proposed by V. Pashynskyi on the following criteria: failure to achieve a sufficient level of thermal resistance through the variability of geometric and thermal characteristics of materials enclosing structures; exceeding the maximum acceptable value of density of heat flow through the enclosing structure [7].

Further research in the area of thermal reliability of enclosing structures, in particular enclosing structures of light steel thin-walled profiles was performed by V. Semko and S. Pichugin [8 – 10].

Methods for determining the probability of thermal failures of enclosing structures of cold formed steel elements in three thermo-technical parameters were proposed and developed by V. Semko – the provided thermal resistance, exceeding the values of the temperature difference between the provided temperature of internal surface of the construction and the temperature of internal air above the permissible temperature values for the sanitary requirements and criteria for reducing local temperature values of internal surface to the temperature of vapor condensation air.

Selection the general problems are not resolved before. The results of previous studies of higher mentioned scientists and our own developments [11] show that at performance of deterministic calculations of wall panels of light steel thin-walled structures and its specific nodes, in most cases the three main thermal criteria are fully ensured, while indicators of its faultless operation may be low under these same criteria.

In cases where the design scheme provides enough qualitative insulation of heat-conducting inclusions and wall constructions calculations performed for ideal (theoretical) conditions of their production and operation, the faultless operation performance can be quite high in three main thermo-technical criteria. But as practice shows, in real conditions of building the defects of structures are often observed in the following way: thermal insulation of thermal bypass is not performed quite efficiently, especially in the joints of several metal elements, the accumulation of moisture in the insulation is also possible and others.

Therefore the aim of the article was offer to perform the probabilistic analysis of the main thermal indicators of the wall of light steel thin-walled structures under change of physical characteristics.

Main material and results. Construction of exterior wall panels of light steel thin-walled structures (Fig. 1) with its structural nodes (Fig. 2) was chosen for numerical experimental research.

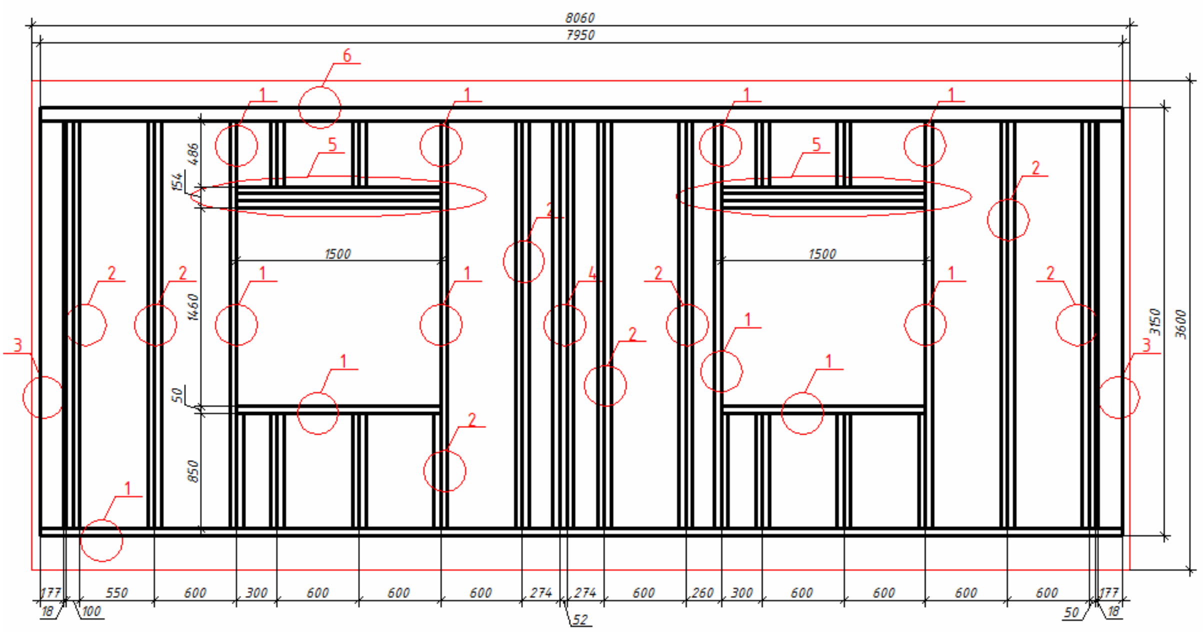


Figure 1 – Construction scheme of exterior wall of light steel thin-walled structures: 1-6 – numbering of structural nodes

Research of influencing factors on the value of the local temperature at the site of heat-conductive inclusion, for each of the experimental nodes, showed that the most influential indicator is the thermal conductivity of insulation in the construction of wall of light steel thin-walled structures. That is why was proposed to perform probabilistic analysis of the main thermal indicators of the wall with different coefficients of variation of heat conduction of thermal insulation material.

Experimental research in [12] has allowed to receive a coefficients of variation of heat conduction of mineral wool V=3,52%. According to previous research [13-16] this value ranged from V=2,28% to V=12,7%. These results enabled to estimate the probability of thermal failure of (by three main thermo-technical criteria's) a nodes and a whole panel wall of light steel thin-walled structures under changing the coefficient of variation of heat conductivity of mineral wool from V=2,28% to V=20%.

In determining the probability of faultless operation under the criterion of reducing local values of temperature of internal surface to the temperature of water vapor condensation (Table 1), calculations were made for the climate of Poltava and Kropyvnytskyi (1-st climatic area) with different values of relative humidity of internal air (the values of relative humidity of internal air are shown from below for each schedule in Table 1, %).

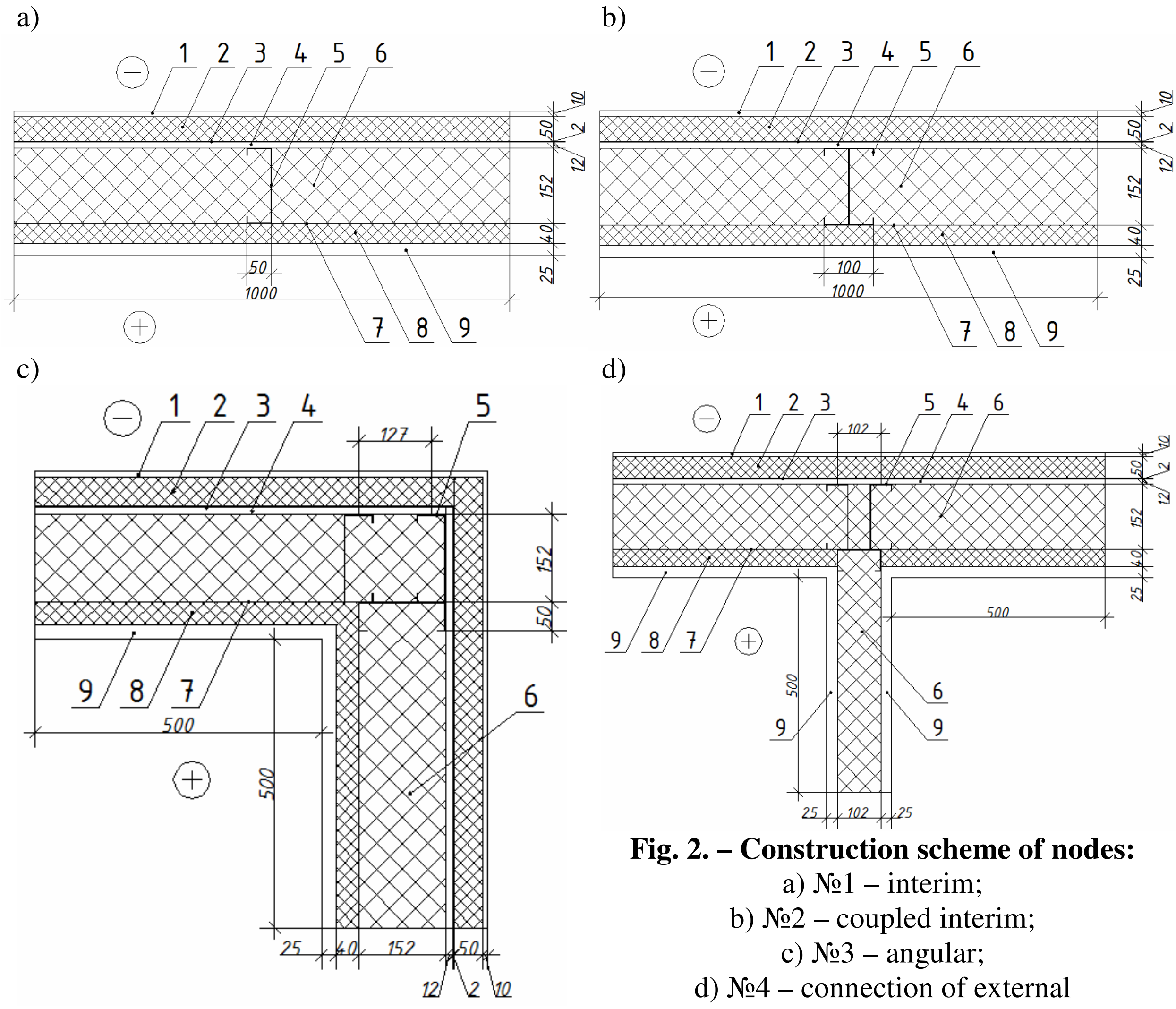


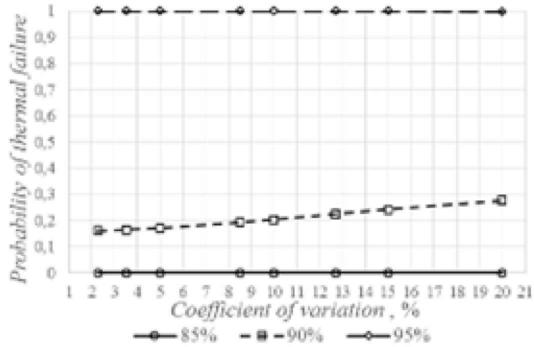
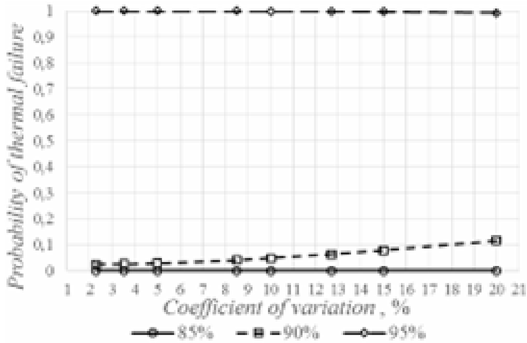
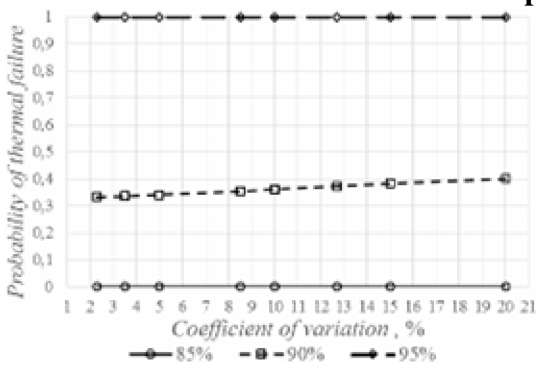
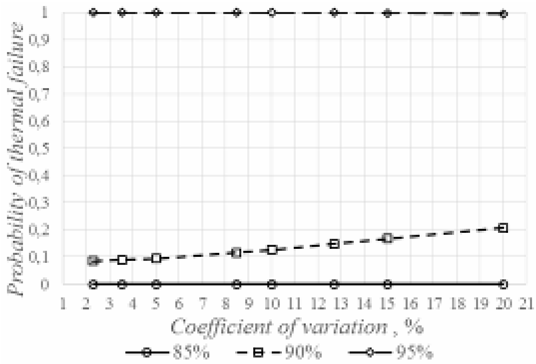
Fig. 2. – Construction scheme of nodes:
a) №1 – interim;
b) №2 – coupled interim;
c) №3 – angular;
d) №4 – connection of external wall with internal one

1 – plaster layer: 10 mm; 2 – insulation: 50 mm; ρ=135 kg/m³; λ(V)=0,042 W/m·K;
3 – polymer adhesive for thermal insulation: 2 mm; 4 – slab OSB-3: 12 mm;
5 – metal frame (profile C152,4×50,8×1,5; t=1,5 mm); 6 – insulation: 150 mm; ρ=45 kg/m³;
λ(V)=0,043 W/m·K; 7 – vapor insulation; 8 – insulation: 40 mm; ρ=135 kg/m³;
λ(V)=0,042 W/m·K); 9 – gypsum plasterboard: 25 mm; ρ=800 kg/m³; λ(V)=0,21 W/m·K.

The probability of thermal failure of wall panels of light steel thin-walled structures is almost zero under the criterion of values exceeding of temperature difference between the specified temperature of internal surface of the construction and the temperature of internal air above the permissible temperature values by sanitary requirements at change of coefficient of variation of heat conductivity of mineral wool from V=2,28% to V=20%. This is according to obtained values of safety coefficient, which are rather large.

The probability of thermal failure of wall enclosing constructions under criteria of specified heat transfer resistance and overall resistance of heat transfer, shown in Figure 3, can be divided into three stages: 1) from V=2,28% to V=10% – probability of thermal failure remained almost zero; 2) from V=10% to V=15% – probability of thermal failure increased for 15.3 times; 3) from V=15% to V=20% – probability of thermal failure increased f or 4,8 times compared to the previous interval. At the same, the condition in terms of specified resistance of heat transfer is fully implemented under the deterministic approach.

Table 1 – The probability of thermal failure under changing coefficient of variation of heat conductivity

<table>
<tr><td colspan="2">City</td></tr>
<tr><td>Poltava</td><td>Kropyvnytskyi</td></tr>
<tr><td colspan="2">№1. Interim node</td></tr>
<tr><td>
</td><td>
</td></tr>
<tr><td colspan="2">№2. Coupled interim node</td></tr>
<tr><td>
</td><td>
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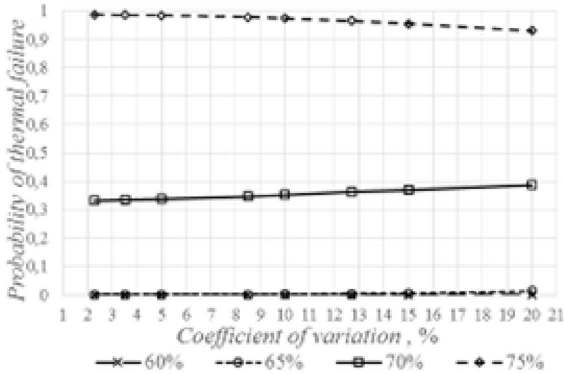
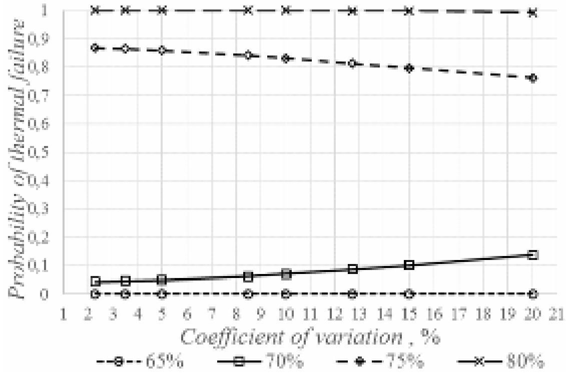
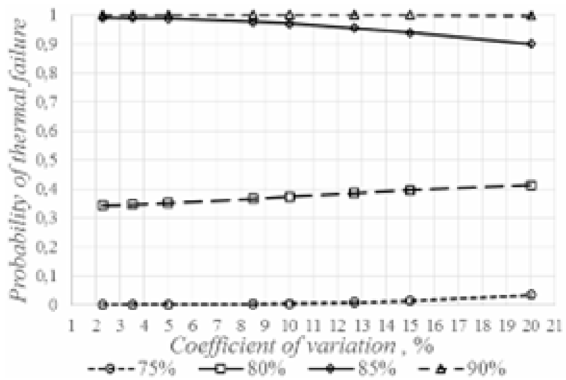
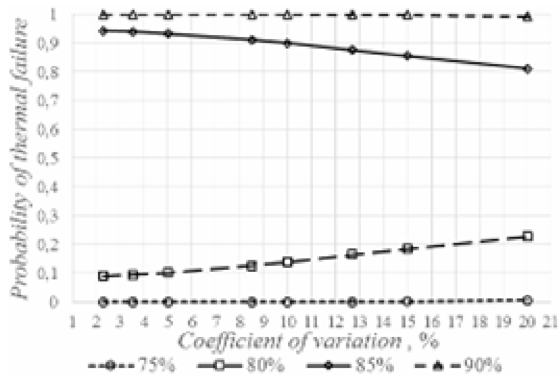
<tr><td colspan="2">№3. Angular node</td></tr>
<tr><td>
</td><td>
</td></tr>
<tr><td colspan="2">№4. Node of connection of external wall with internal one</td></tr>
<tr><td>
</td><td>
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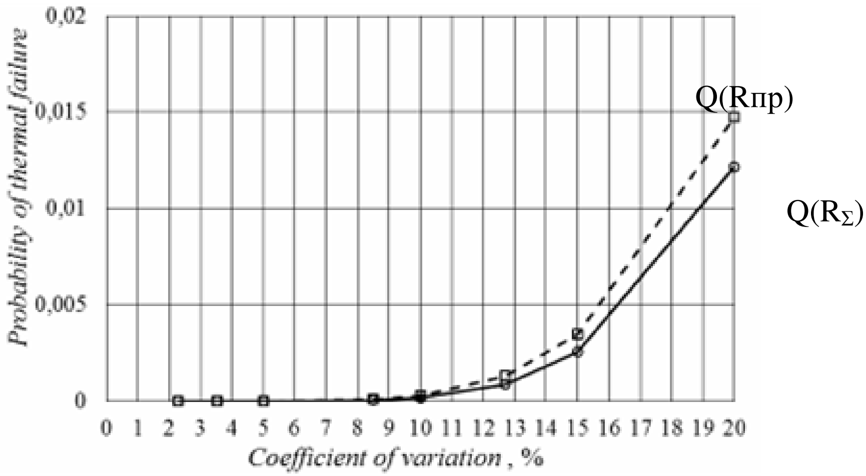


Figure 3 – Graph of changing probability of thermal failure depending on the coefficient of variation of heat conductivity of mineral wool

The following condition in the research of thermal reliability of wall of light steel thin-walled structures: coefficient of thermal conductivity of insulation material was taken as a constant λ=0,0423 (W/m·K) with providing 0,95 (Table 2). The coefficient of variation of thermal conductivity of mineral wool is ranged from V=2,28% to V=20%.

Table 2 – The coefficient of thermal conductivity under changing coefficient of variation

V, %	2,28	3,52	5	8,5	10	12,7	15	20
λ, (W/m·K)	0,0439	0,0448	0,0457	0,0482	0,0492	0,0511	0,0526	0,0561

Table 3 – Indicators of reliability of thermal conductivity functions with providing 0,95

	R_{Σ}	$R_{пр}$
M	5,035	4,128
S	0,391	0,219
β	4,438	3,790
Q	$4{,}54x10^{-6}$	$7{,}53x10^{-5}$

The data in Table 3 prove that the probability of failure under the criterion of specified heat transfer resistance is increased for 2.2 times when using value of thermal conductivity of mineral wool with providing 0.95 in the experimental wall constructions of light steel thin-walled profiles.

Another physical characteristic that influences on probabilistic analysis of the main thermal indicators of the wall of light steel thin-walled structures is a linear heat transfer coefficient.

The probability of faultless operation of wall panels of light steel thin-walled structures (under criterion of specified heat transfer resistance) was defined by specified conditions, in particular, that linear heat transfer coefficient for each of the structural nodes changes depending on thermal conductivity of insulation material (Fig. 3). The coefficient of variation of thermal conductivity of mineral wool is ranged from V=2,28% to V=20%.

The graph in Figure 4 shows that the probability of failure-free operation under criterion of decreasing of local values of temperature of internal surface to the temperature of vapor condensation is reduced with increasing of variability of linear heat transfer coefficient for each of the structural nodes. At the same, the probability of thermal failure (Fig. 5) (under criterion of specified heat transfer resistance) of wall panels of light steel thin-walled structures for the 1-st temperature area is increased comparatively to the variability of linear heat transfer coefficients: for V=5% –0,6 times; foe V=10% – 4,4 times; for V=15% –25,7 times; for V=20% – 93,6 times.

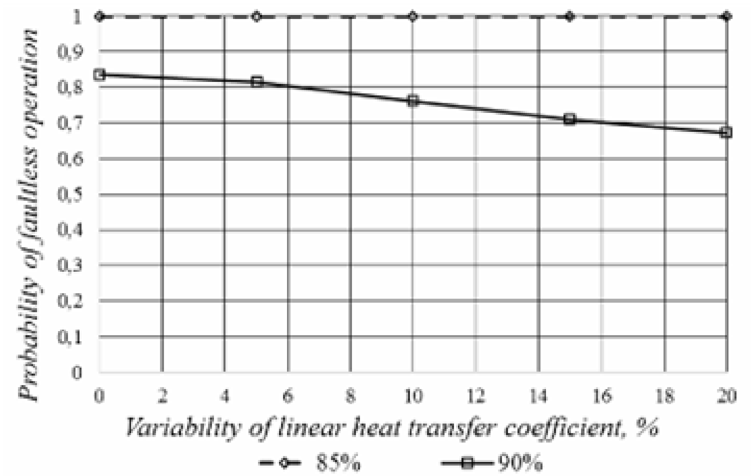


Figure 4 – Probability of faultless operation of constructive node №1 with variability of linear heat transfer coefficient at various humidity of internal air for Poltava

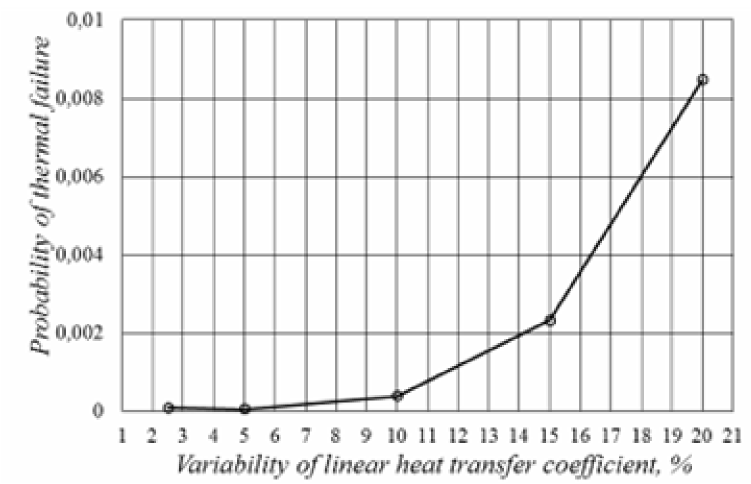


Figure 5 – Graph of changes the probability of thermal failure under criteria of specified heat transfer resistance with variability of linear heat transfer coefficient

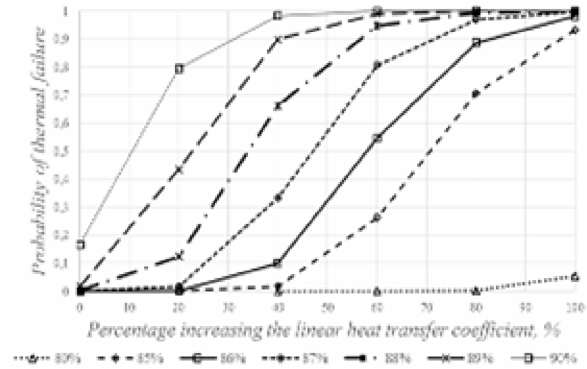
Conductive inclusions have a great impact on the thermal reliability of walls of light steel thin-walled structures. Theoretically determined values of linear heat transfer coefficient in place of heat-conductive inclusion can be greater in practice because of defects and errors during installation. That is why necessary to consider a possible increase value of the linear heat transfer coefficient.

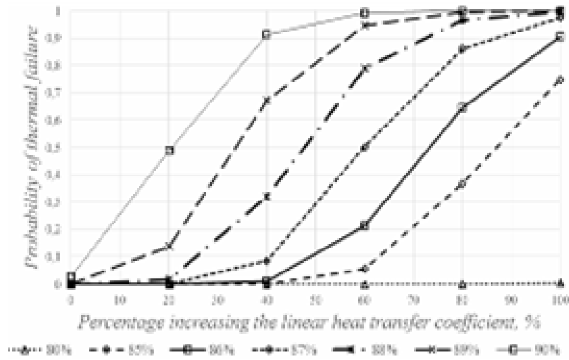
During the probabilistic analysis of basic thermal indicators of the walls of light steel thin-walled structures, the conditions for calculation remained unchanged under criterion of reduction of local temperature values of internal surface to the temperature of condensation of water vapor – for city: Poltava and Kropyvnytskyi, which are in the same climatic area with different values of relative humidity of indoor air (Table 4).

Table 4 – Probability of thermal rejection under changing the linear coefficient of thermal conductivity

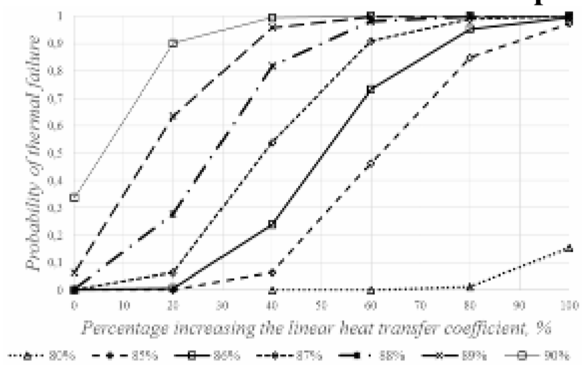
City	
Poltava	Kropyvnytskyi

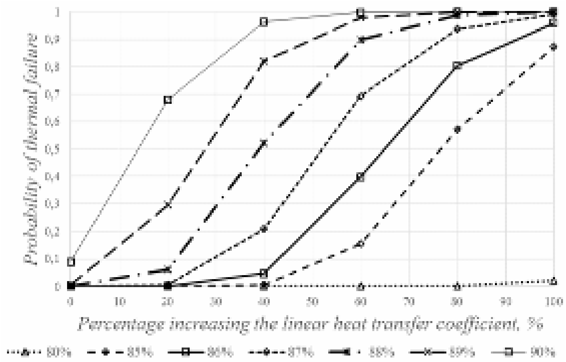
№1. Interim node



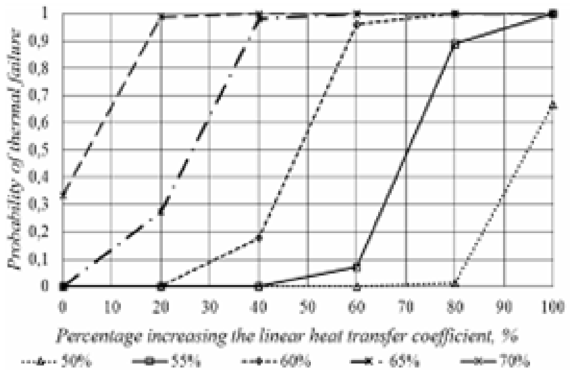


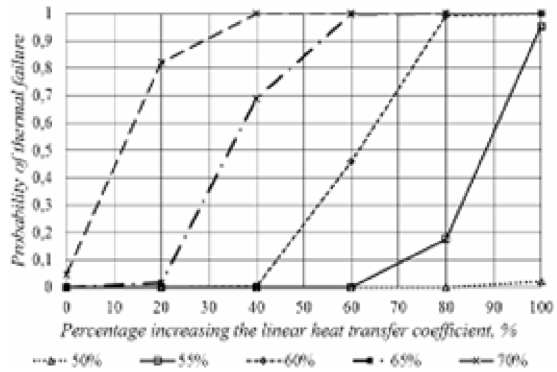
№2. Coupled interim node



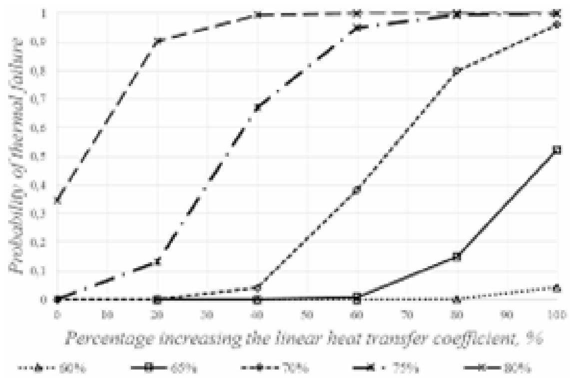


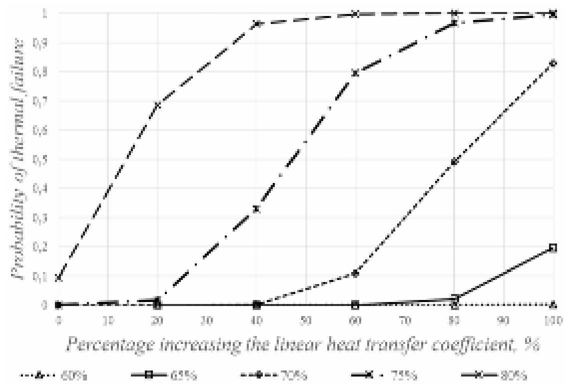
№3. Angular node





№4. Node of connection of external wall with internal one





At increase of value of the linear heat transfer coefficient for 20% 40%, 60%, 80% and 100%, the probability of thermal rejection for each of considered nodes№1-№4 (Fig. 2) of wall of light steel thin-walled structures (Fig. 1) increases with increase of humidity of indoor air. The probability of thermal failure presented in the form of vapor condensation in place of heat-conductive inclusion on the wall.

Probability of thermal failure for nodes №1 and №2 for Poltava is possible only when the humidity of internal air is 85% and higher, for Kropyvnytskyi – 86% and higher. At increase of linear heat coefficient for 20%, the probability of thermal failure (under internal air humidity – 90%) will increase for 4.8 times for Poltava and for 18,7 times for Kropyvnytskyi. At increase of value of the linear heat transfer coefficient for 40%, 60%, 80% and 100%, the probability of thermal rejection is almost 100%.

For example, Probability of thermal failure for angular node №3 is possible when the humidity of internal air is 60% and higher for Poltava (Table 4) and for Kropyvnytskyi – 65% and higher. At increase of value of the linear heat transfer coefficient for 20%, the probability of thermal failure (under internal air humidity – 65%) will increase from 0,0005 to 0,27 for Poltava and from $1{,}1\times10^{-6}$ to 0,017 for Kropyvnytskyi; for 40% – it will increase plus for 6.3 times for Poltava and for Kropyvnytskyi – for 41 times. At increase of value of the linear heat transfer coefficient for 60% and higher, the probability of thermal failure (under internal air humidity – 65%) is almost 100%. The values of relative humidity of internal air are shown from below for each schedule in Table 4, %.

Thermal reliability (faultless operation) of wall panel of light steel thin-walled structures under criterion of specified heat transfer resistance is reduced depending on a linear coefficient of thermal conductivity. By its increasing for 20% the faultless operation of wall panel is 99,8%, for 40% – 97,2%, for 60% – 85,1%, for 80% – 60,0%, for 100% – 32,2%. This is evidenced by the graph on Figure 6.

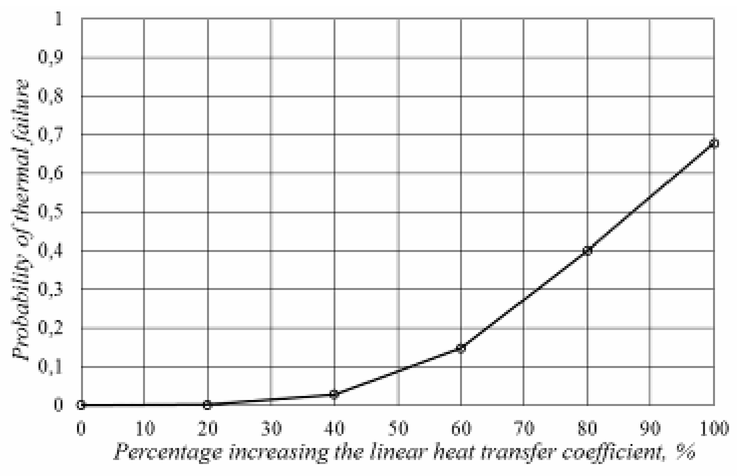


Figure 6 – Probability of thermal failure under criterion of specified heat transfer resistance depending on the a linear coefficient of thermal conductivity

The results of probabilistic calculations have shown that the effect of the variability of heat conduction of insulation and linear heat transfer coefficient of heat-conductive inclusion significantly affect the thermal reliability of walls. That is why it is necessary to recommend determining the thermal conductivity of insulation on large series of samples to establish the values of coefficient of thermal conductivity variation. It is appropriate to use a value of thermal conductivity with providing 0.95 for samples with a high thermal conductivity variability (V>10%) for theoretical calculations. Also the control of conformity theoretical values of linear coefficient of thermal conductivity to actual values is recommended for serial constructions. It is necessary to prevent increasing losses of heat in areas of linear temperature inclusions because of installations defects by increasing the controls on the construction plant or steel production plant.

So, as a result of the research, we can make the following **conclusions**:

1. The probability of thermal failure is calculated in three thermo-technical criteria for frame buildings, enclosing structures of which have many temperature inclusions. Thermo-technical criteria are following: the specified thermal resistance, exceeding the values of the temperature difference between the specified temperature of internal surface of the construction and the temperature of internal air above the permissible temperature values for the sanitary requirements and criteria for reducing local temperature values of internal surface to the temperature of vapor condensation air.

2. Determination of probability of failure of enclosing structures under the criterion of local values decrease of temperature of internal surface to the temperature of vapor condensation showed that the probability of thermal failure can vary from 2 to 7 times for different structural nodes of walls of light steel thin-walled structures, which were designed in different places of the same temperature zone.

3. Changing the coefficient of thermal conductivity variation of insulation material leads to changes in the level of thermal reliability of wall constructions. The probability of failure of wall panels of light steel thin-walled structures under the criterion of specified heat resistance increases from 0,015 to 9,85x10^{-7}while increasing the coefficient of variation from 2.28% to 20% respectively.

4. Changing the linear heat transfer coefficient of heat-conductive inclusion leads to changes in the level of thermal reliability of wall construction in general. The probability of thermal failure for the angular node (projected in Poltava) under the criterion of specified heat resistance increases from 0,05% to 27% while increasing the linear heat transfer coefficient from 20% to 40% respectively.

5. This approach can be used not only to assessment the thermal reliability for an enclosing structures of cold formed steel profiles but also for any other enclosing structures with heat conductive inclusions.

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Received 07.02.2017

UDC 624.131

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FEATURES OF THE MATHEMATICAL MODELING OF FOUNDATIONS INTERACTION WITH COMPACTING SOILS, WITH ANISOTROPIC PROPERTIES

Features of the mathematical modeling of foundations interaction with compaction soils with anisotropic properties, by ultimate elements method in the physically and geometrically non-linear presentation are confirmed. To prove this fact it was obtained henomenological soil model that describes its state during building, work of compacted subsoil. Famous physical correlations of orthotropic medium were used in model. Modeling of cast work -in-situ pile with leading borehole and enlarged base for isotropic basic and for transversely-isotropic medium were compared. Reality of decisions obtained by modeling are provided by properties of ultimate elements, by sizes of calculated field, by choice of design schemes of soil compaction, by conformity of model state soil parameters.

Keywords: *soil natural and directed anisotropy, foundation, soil compaction zone, orthotropic medium, method of ultimate elements, axis-symmetrical problem.*

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ОСОБЛИВОСТІ МАТЕМАТИЧНОГО МОДЕЛЮВАННЯ ВЗАЄМОДІЇ ФУНДАМЕНТІВ З УЩІЛЬНЕНИМИ ҐРУНТАМИ, ЩО МАЮТЬ АНІЗОТРОПНІ ВЛАСТИВОСТІ

Викладено особливості моделювання методом скінченних елементів у фізично й геометрично нелінійній постановці взаємодії фундаментів з ущільненими ґрунтами, що мають анізотропні властивості. Для цього розроблено феноменологічну модель ґрунту, що описує його стан при влаштуванні та роботі ущільнених основ. У моделі використано відомі фізичні співвідношення ортотропного середовища. Наведено порівняння результатів моделювання роботи набивної палі з лідируючою свердловиною й розширенням для ізотропної основи та для трансверсально-ізотропного середовища. Достовірність рішень, отриманих моделюванням, забезпечено властивостями скінченних елементів, розмірами розрахункової області, вибором розрахункових схем ущільнення ґрунту, відповідністю параметрів моделі стану ґрунту.

Ключові слова: *природна та наведена анізотропія ґрунту, фундамент, ущільнена зона, ортотропне середовище, метод скінченних елементів, вісесиметрична задача.*

Introduction. Classic calculation methods are tested for common foundations with compacted subsoil. They are not universal because of soil conditions difference and new structural and technologic solutions, which during designing often leads to necessity of additional natural tests. In the same time current level of soft allows to use methods of bases and foundations stress-strain state (SSS) modeling on solution of practical geotechnical problems. Current practice [1] recommends to use tested program complex of finite elements method (FEM), especially in the physically and geometrically non-linear presentation.

Analysis of recent sources of research and publications. O. Bugrov, F. Gabibov, N. Zotsenko, V. Lushnikov, Yu. Osipov, E. Sergeev, L. Timofeeva, G. Cherny, O. Shkola, B. Amadei, A. Bishop, I. Duncan, G. Gazetas, H. Kulatilake, K. Lo, J. Magnan, M. Oda, H. Seed and other [1 – 7] investigated anisotropy of mechanical soil properties. I. Boyko, G. Geniyv, M. Goldshteyn, S. Klovanych, O. Кorobova, V. Кovtun, O. Petrakov, S. Thimbal, P. Allahverdizadeh and other [8 – 11] developed the advantages of anisotropic model over isotropic.

Because of specificity for clay deposits with aqueous origin, loess, strip clays primary (natural) mechanical (deformation, strength) anisotropy are caused by their natural structure (ordered structure with priority parallel orientation of particles or pores in certain direction), its origin, formation condition, (including the process of sedimentation), etc., and secondary anisotropy, the nature and the laws depending on the natural structure of the soil and on the particular technologies of foundations (the direction of displacement of soil working body piles, blocks of sizes between Base Area) [2 – 7 11] – it allows to use physical models in their relations anisotropic, and firstly orthotropic environment.

But presented environmental variables in the formation of its foundations and bases of determining soil compaction simulation of the transient processes include soil compaction by different technologies, with FEM physically and geometrically nonlinear statement [10 – 15].

Identification of general problem parts unsolved before. Today, even with help of modern program complex of FEM in the physically and geometrically non-linear presentation fair evaluation of SSS of soil foundations arranged with soil compaction with anisotropic properties, were not investigated.

That is why the **goal** of this article is to improve the methodic of modeling in the condition of axis-symmetrical problem of FEM in the physically and geometrically non-linear presentation of foundations (or piles) interaction with compaction soils, which have anisotropic properties.

Basic material and results. The authors investigated the mechanical anisotropic soil properties by taking soil samples by cutting rings, oriented in different angles (α = 0; 45; 90^{o}) to horizontal plane (it's taken as isotropic plane), with tests in the odometers, direct shear apparatuses, penetrometers. Penetration was performed directly within the array by penetrometers PD-2M and MV-2 perpendicular to the sites cleaned in different directions to the plane of isotropy. In array of points for all research areas at the plane isotropy has coefficient of variation values of soil mechanical properties submitted in the form of quadrants or travel time, which is graphical representation of the behavior of the soil mechanical characteristics from the angle α [5 – 7].

The coefficients of mechanical anisotropic properties were defined:

$$n_{E,\alpha} = E_{\alpha} / E_{-} \ ; \tag{1}$$

$$n_{c,\alpha} = c_{\alpha} / c_{-} \ ; \tag{2}$$

$$n_{\varphi,\alpha} = tg\varphi_\alpha / tg\varphi_{\_} \;; \tag{3}$$

$$n_{R,\alpha} = R_\alpha / R_{\_} \;, \tag{4}$$

where $E_{\_}$ – modulus of soil deformation in isotropic plane from impact of stresses in the same plane (orientation of rings under angle $\alpha = 0^o$ relatively to horizontal plane);

E_α – the same, respectively for plane, inclined to isotropic plane on angle α, $c_{\_}$, c_α, $\varphi_{\_}$;

φ_α – unit cohesion and angle of internal soil friction in plane of shift respectively is parallel ($\alpha = 0^o$) and inclined to isotropic plane on angle α;

$R_{\_}$ and R_α – unit penetration resistance according to $\alpha = 0^o$ and $\alpha \neq 0^o$ to isotropic plane.

In massif of loessial loam (W_L = 0,29 – 0,33; W_P = 0,18 – 0,20; e = 0,83 – 0,99; w = 0,13 – 0,23) established that the highest values of mechanical properties are character for the samples, which are taken for angle $\alpha = 0^o$ to horizontal plane, the smallest – for angle $\alpha = 45^o$ (n_α = 0,6 – 0,9). With depths soil anisotropy increased: in depth 1 m from surface $n_{\alpha=90}$ = 0,85 – 0,9 and in depth 4,0 – 4,5 m – $n_{\alpha=90}$ = 0,7 – 0,8. Under water saturation and silicatization loess has isotropic properties ($n_\alpha \to 1,0$).

Bulk loams for 10 – 40 years of compaction under own weight gained anisotropic properties ($n_{\alpha=90}$ = 0,65 – 0,95). Its options of anisotropy depend on time of own weight compaction. After soil compaction priority directions of locus answered to displacement directions of the soil by pile working body or prefabricated elements (n_α = 0,5 – 2,0) [6, 7].

For conditions, when coefficients of soil anisotropy differ significantly from n_α = 1,0, calculation accuracy of the soil base SSS could be increased by its using in the model of physical correlations of orthotropic or transversely-isotropic medium. Parameters describing these bodies (in the cylindrical coordinate system – which scheme is presented in Figure 1)are: modulus of deformation in isotropic plane E_r and E_Θ, and also transverse direction E_z; respectively Poisson's ratios $v_{r\Theta}$, v_{rz}, $v_{\Theta z}$. In the case of model of transversely-isotropic medium using in calculation shows that $E_\Theta = E_r$.

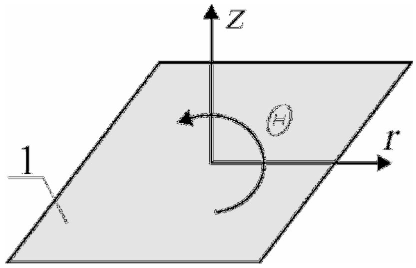

Figure 1 – The cylindrical coordinate system: 1 – isotropic plane

According to classification of compaction methods and phenomenological elastoplastic soil model were created program complex «PRIZ-Pile». It is realized in form of axis-symmetrical problem solution by FEM in physically and geometrically non-linear form [10, 11]. Geotechnic modelling:

1) different by geometry, scheme of soil displacement, character and speed of pressure transition processes of foundations arrangement with compacted subsoils, which result is SSS of massif and presented values of soil properties;

2) further work of such soil basis and foundations under load. Eight-nodal isoparametric FE can change its shape and volume, which allows using rectangular and curved mesh. In presenting of the soil isotropic medium physical correlations of SS in matrix form with form:

$$\begin{Bmatrix} \sigma_r \\ \sigma_\theta \\ \sigma_z \\ \tau_{rz} \end{Bmatrix} = \frac{E}{\Omega} \begin{bmatrix} 1 & \frac{\nu}{1-\nu} & \frac{\nu}{1-\nu} & 0 \\ \frac{\nu}{1-\nu} & 1 & \frac{\nu}{1-\nu} & 0 \\ \frac{\nu}{1-\nu} & \frac{\nu}{1-\nu} & 1 & 0 \\ 0 & 0 & 0 & \frac{1-2\nu}{2(1-\nu)} \end{bmatrix} \begin{Bmatrix} \varepsilon_r \\ \varepsilon_\theta \\ \varepsilon_z \\ \gamma_{rz} \end{Bmatrix}; \tag{5}$$

$$\Omega = [(1+\nu)(1-2\nu)]/(1-\nu), \tag{6}$$

where σ_r, σ_Θ, σ_z, τ_{rz} – normal and tangential stresses in UE y cylindrical coordinates;
E – modulus of soil deformation those FE;
ν – Poisson's soil ratio of FE;
ε_r, ε_Θ, ε_z, γ_{rz} – axial and angle components of strains in FE.

In presenting soil by anisotropic (orthotropic) medium physical equations of SS in matrix form with form:

$$\begin{Bmatrix} \sigma_r \\ \sigma_\theta \\ \sigma_z \\ \tau_{rz} \end{Bmatrix} = \frac{1}{\Omega} \begin{bmatrix} E_r(1-\nu_{\theta z}\nu_{z\theta}) & E_r(\nu_{r\theta}+\nu_{rz}\nu_{z\theta}) & E_r(\nu_{rz}+\nu_{r\theta}\nu_{\theta z}) & 0 \\ E_\theta(\nu_{\theta r}+\nu_{zr}\nu_{\theta z}) & E_\theta(1-\nu_{rz}\nu_{zr}) & E_\theta(\nu_{\theta z}+\nu_{rz}\nu_{\theta r}) & 0 \\ E_z(\nu_{zr}+\nu_{\theta z}\nu_{z\theta}) & E_z(\nu_{z\theta}+\nu_{r\theta}\nu_{zr}) & E_z(1-\nu_{rz}\nu_{\theta r}) & 0 \\ 0 & 0 & 0 & \Omega G_{rz} \end{bmatrix} \begin{Bmatrix} \varepsilon_r \\ \varepsilon_\theta \\ \varepsilon_z \\ \gamma_{rz} \end{Bmatrix}; \tag{7}$$

$$\Omega = 1 - 2\nu_{\theta r}\nu_{rz}\nu_{z\theta} - \nu_{r\theta}\nu_{\theta r} - \nu_{\theta z}\nu_{z\theta} - \nu_{rz}\nu_{zr}, \tag{8}$$

where E_r, E_Θ, E_z – modulus of soil deformation for relevant directions;
$\nu_{r\Theta}$, ν_{rz} $\nu_{\Theta z}$ – respective Poisson's ratios, which are calculated as follow:

$$\nu_{\theta r} = \frac{E_r}{E_\theta}\cdot\nu_{r\theta}; \qquad \nu_{zr} = \frac{E_r}{E_z}\cdot\nu_{rz}; \qquad \nu_{z\theta} = \frac{E_\theta}{E_z}\cdot\nu_{\theta z}. \tag{9}$$

For transversely-isotropic medium $E_\Theta = E_r$.
G_{rz} – shear modulus, according to formula of S. Lekhnitcky:

$$G_{rz} = \frac{E_r E_z}{E_z + E_r(1+2\nu_{rz})}. \tag{10}$$

For each layer of the soil (stiffness) is defined:

– initial stiffness – depending on the characteristics consideration of anisotropy. In the case of soil presentation by isotropic medium the stiffness characteristics presented in the form of entry modulus of deformation and Poisson's ratio ν. In presenting of the soil by orthotropic medium as stiffness characteristics modulus of deformation E_r, E_Θ, E_z and respective Poisson's ratios $\nu_{r\Theta}$, ν_{rz} $\nu_{\Theta z}$ are considered Acceptance of the hypothesis of transversely-isotropic medium [2] is possible. Then: $E_\Theta = E_r$; $\nu_{\Theta z} = \nu_{rz}$;

– correlations of deformation modulus from volume (or porosity) of soil $E_i/E_\circ = f(V_i/V_\circ)$ in the form of analytical expression [11] or table. For the first phase of this dependence, it is considered the rate of applied load, corresponding to foundation arrangement technology, and the second - static load;

– correlations of soil resistance shift from normal stress $\tau = f(\sigma)$;

– soil unit weight γ.

Except geometric dimensions, which are result of the first stage, as initial parameters of foundations on the second stage physical and mechanical properties of materials (unit weight, angle of internal soil friction, unit cohesion, modulus of deformation) and lateral earth pressure coefficients λ and the impact of sliding side surface foundation on modulus of soil deformation are considered (from 0 to 1). When the foundation has several components, these parameters are determined for each foundation component.

Calculation in complex is performed in two stages. On the first stage is simulating the formation of cavity under foundation (pile, artificial base). The cavity axis coincides with a symmetry axis of the designed field. External influence is given in a form of forced vertical and horizontal displacements of unit of FE net, which lay on axis of rotation on the highest limit of designed field or occupy in it another position, which simulates the process of pressing out the soil by pile (rammer). These displacements lead to decreasing FE value and thus to reduction of soil porosity and the increase of its deformation modulus and strength though the process of soil boiling is possible. Because the forced displacements coincide with FE sizes of calculated scheme it is corrected on each stage by correction unit coordinates accounting displacements of previous step. With the change of coordinates, the FE values change and it gives the possibility to correct sol modulus of deformation in each FE for speed of loading, which corresponds to technology of the foundation arrangement. Soil void ratio in each FE is

$$e_i = e_\circ - \left(1 + e_\circ\right)\left(1 - V_i / V_\circ\right). \tag{11}$$

The result of the first stage (and its steps) are new coordinates of FE units, the given soil characteristics, displacements of units of FE net, strength in the massif which are given in a form of tables, charts, isolines. Considering the fact that calculation on first stage are connected with step by step solution of the problem of defined moving and are deformed by the scheme at every step, then, as a rule, here is significant change in the shape of FE, which can lead to degeneration of the FE. Therefore, it should be considered moving the nodes less than the magnitude of product size element and soil void in this FE or appropriately select the size of FE.

Calculated soil characteristics and SSS massif enable to pass to the second stage – simulation of behavior of pile (foundation) under load. The hollow received by pressing out the soil is filled in by construction material, its characteristics are designed and new FE simulating foundation is introduced. So in contrast to models with fixed value of deformation modulus, this model describes its changes in volumetric strains especially in compaction, according to the change of soil porosity and transmission speed of pressure on it.

Peculiarity of the model on second stage is that in complicated SSS (compression with shear) general strains include linear (elastic) and plastic parts, thus plastic part of strain appears after SSS reach of strength limit according to Mises-Shleikher-Botkin. The compaction of soil is considered, its transfer into plastic stage with the reach of strength limit according to condition of strength, the possibility of sliding of side surface of foundation relatively the soil. The latter is realized by controlling tangential stresses in FE soil which are in contact «pile – soil». The task is checked by

$$\tau_{rz} \le (\sigma_r + \gamma h \lambda) tg \varphi + c \ , \tag{12}$$

where σ_r – radial normal stresses;
h – distance from surface;
λ – lateral earth pressure coefficient;
φ – angle of internal friction;
c – unit cohesion.

With further loading on the surface of the FE, adjacent to soil, it was applied uniformly distributed load by friction forces on the foundation by soil $p = \gamma h \lambda \ tg\varphi$. Results of stage are: dependence of foundation settlement on load; displacements of each FE node; stresses in massif; soil transition in the fluid state in some FE; there soil properties are presented.

Examples of initial design scheme area division of the FE after the formation of leading borehole with diameter 0.5 m and depth 5.0 m for modeling SSS of cast-in-situ pile with enlarged base with crushed stone are presented in Figure 2 (borehole limited by nodes 865, 867 and 1186). The scheme has 369 FE with dimensions from 0,25×0,25 up to 0,8×1,0 m and 1204 nodes (150 fixed). Calculated region – cylinder with diameter 9,1 m and height 15 m. To depth 1.5 m is laying loam heavy silty, stiff (ρ_d = 1,41 g/cm^3), the range 1,5 – 3,5 m – loam light silty, stiff (ρ_d = 1,49 g/cm^3; E = 5.8 MPa), lower – clay light silty, stiff (ρ_d = 1,54 g/cm^3; E = 14 MPa). The enlarged base is created by soil compaction with crushed stone V_{cr} = 1,5 m^3 ($V_{cr.1}$ = 0,25 m^3) by the rammer with diameter 430 mm. The enlarged base has form of ellipsoid with half-axes: horizontal r_{br} = 0,65 m and vertical h_{br} = 0,70 m [16]. Its formation is modeled by applying forced horizontal and vertical displacements the 8 nodes (from 865 to 983) located on a path lower part of the borehole. The fragment of scheme deformation basis for constructing the enlarged base is presented in Figure 3.

The higher level of soil compaction is near and under the enlarged base. On the distance 0,15 m from the lateral surface of the enlarged base the value ρ_d increased from 1.54 g/cm^3 to 2,07 g/cm^3. Radius of a sufficient compaction zone, where ρ_d = 1,60 g/cm^3, according to modeling is $r_s \approx$ 1,00 m, and from [16] r_s = 0,96 m. Modulus of soil deformation in borders of the sufficient compaction zone increased by 2,3 times its size in the middle of the zone – in 3.5 times. The above parameters around the pile soil are used for modeling of pile work under the loading. The cavity, obtained by drilling and compacted, «fill» by construction materials (crushed stone and concrete) also additional FE and nodes are inserted (there are respectively
13 and 43), they are modelling pile body and the enlarged base. The load is applied by steps (firstly – 300 кN, and after – steps by 100 кN) in the form of concentrated force to axial node upper side of pile.

As output data of this task for soil base SSS modeling, which could be presented as transversely-isotropic medium, correlations of soil deformation modulus values according to previous investigations were taken [6, 7, 11] on the areas which are composed by loess soils with natural humidity, $n_{E,\alpha=90}$ = 0,8: for loam, placed over the enlarged base, $E_\Theta = E_r$ =4,65 MPa; E_z =5,8 MPa ; for clay, placed under the enlarged base, $E_\Theta = E_r$ =11,2 MPa; E_z =14,0 MPa. Comparison of the «load – settlement» behavior $S = f(P)$ of cast-in-situ pile for isotropic soil base (position 2) and for the transversely-isotropic medium (position 3) is presented in Figure 4.

Figure four presents that settlement according to modeling is on 15 – 20% exceeded results of static tests, but with increasing of the load this difference significantly reduce. The value of pile settlement in the 2 case, if $E_\theta = E_r < E_z$, is about for 10% bigger when $E_\Theta = E_r = E_z$.

So the result doesn't deny famous patterns of soil mechanics and conclusions of other researchers [2, 3, 8]. Also it should be noted that according to modeling as for isotropic medium and for anisotropic soil in any FE don't reach its ultimate limit state.

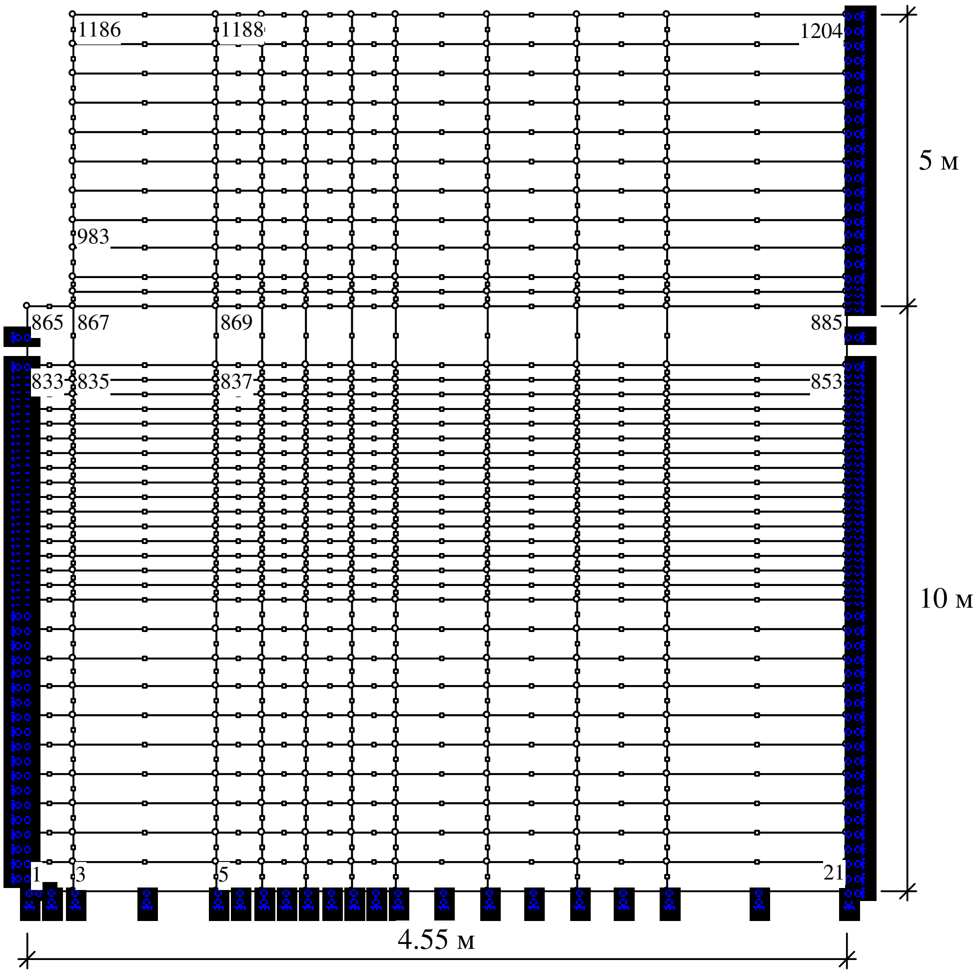

Figure 2 – Examples of initial design scheme area division of the FE after the formation of leading borehole and before the expansion placement

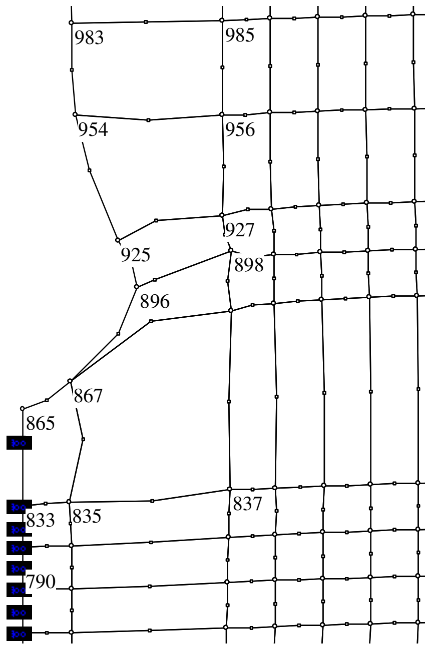

Figure 3 – The fragment of scheme of first stage modeling of pile soil base deformation with leading borehole after arrangement of an enlarged base

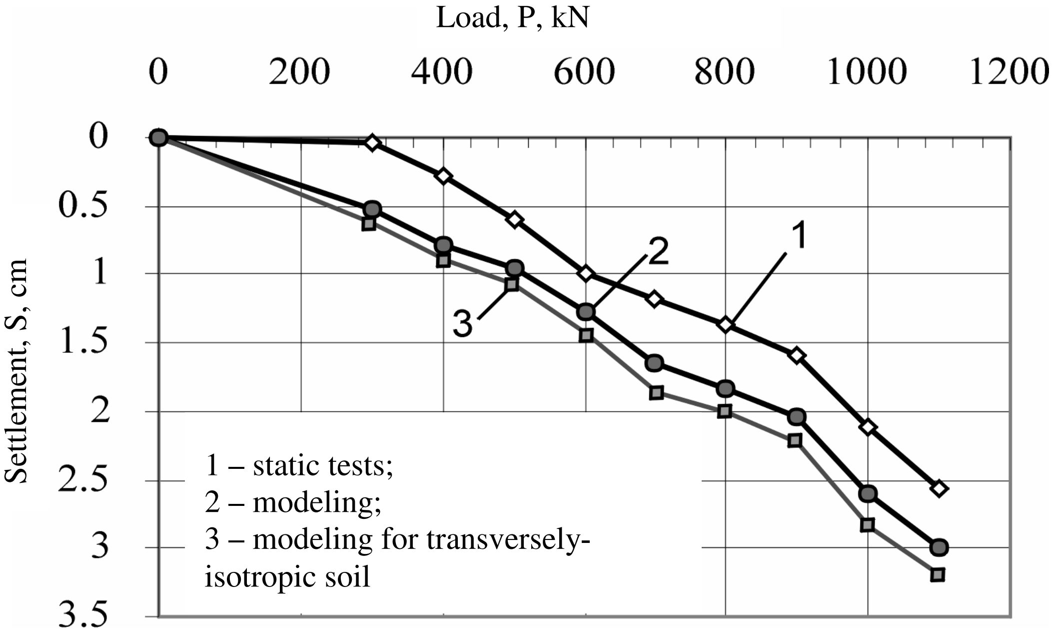

Figure 4 – Graphs of depending the settlement on loading onto cast-in-situ pile with leading borehole and enlarged base:
1 – static tests; 2 – modeling; 3 – modeling for transversely-isotropic soil $n_{E,\,\alpha=90^\circ} = 0{,}8$

Conclusions. For values of the soil deformation in isotropic plane E_r and E_θ in transverse plane E_z for axis-symmetrical problem of FEM in the physically and geometrically non-linear presentation it is possible to estimate SSS of orthotropic soil base of foundation, which are arranged or work with soil compaction.

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Received 10.02.2017

UDC 625.12

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COMPLEX ANALYSIS OF SUBGRADE STRESS-STRAIN STATE WITH COMBINED STRENGTHENING

The paper highlights combined techniques of strengthening that include geotextile laying as well as other related advanced technologies. Subgrade construction analysis and its modification, reinforced with the different types and options of combined strengthening were conducted. To justify strengthening of subgrade a series of numerical calculations were made. Simulation with software package SCAD has confirmed the experimental results. From obtained results one can conclude that minimum horizontal displacements are observed in the version with deepening of geotextile at 1m and vertical ones at 0.4 m. Based on simulation results it is possible to make recommendations concerning modernization of existing subgrade and construction of new one in complex engineering-geological conditions.
***Keywords**: subgrade; combined strengthening; geotextile; stress-strain state*

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КОМПЛЕКСНИЙ АНАЛІЗ НАПРУЖЕНО-ДЕФОРМОВАНОГО СТАНУ ЗЕМЛЯНОГО ПОЛОТНА З КОМБІНОВАНИМ ПІДСИЛЕННЯМ

Розглянуто комбіновані способи підсилення, які включають в себе не тільки укладання геотекстилю, а й інші супутні прогресивні технології. Проведено аналіз конструкції земляного полотна та його модифікації, укріплені різними типами і варіантами комбінованого підсилення. Для обґрунтування підсилення земляного полотна проведено серію чисельних розрахунків. Моделювання в програмному комплексі SCAD підтвердило результати експериментальних досліджень. З отриманих результатів слідує, що мінімальні горизонтальні переміщення у варіанті з заглибленням геотекстилю на 1 м, а вертикальні – на 0,4 м. На підставі одержаних результатів моделювання можливо надати рекомендації щодо модернізації існуючого та будівництва нового земляного полотна у складних інженерно-геологічних умовах.
***Ключові слова**: земляне полотно, комбіноване підсилення, геотекстиль, напружено-деформований стан.*

Introduction. Nowadays construction volumes of railway tracks are significantly increased as well as the repair and reconstruction of the subgrade, including for the introduction of high-speed traffic. In addition, taking into account the development in the field of construction technologies, it should introduce new technical solutions to reduce the strain of subgrade and, accordingly, increase the period of its operation.

According to state target-oriented programs and programs for high-speed implementation on railways major objectives are crucial improvement the technical level of railway infrastructure, production organization of high-speed rolling stock and other railway equipment as well as development of new materials and technologies [1 – 2].

Placement of artificial substructures in complex engineering-geological conditions let reduce construction costs, prevent possible emergence of strains as a result of uneven subsidences and increase service life.

To justify strengthening of subgrade series of numerical calculations was conducted. Optimal characteristics provide the highest stability, strength and stability that are associated with the general stress-strain state (SSS). Calculation of stress-strain state for constructions was conducted using the finite element method with the help of software package Structure CAD for Windows, v.7.31 R.4 (SCAD).

Analysis of recent sources of research and publications. A great number of studies are devoted to strengthening of subgrade, in particular [2 – 5]. Theoretical questions of subgrade reinforcement with geosynthetics make up the major part of research in this field [6]. However, less attention is focused on determining the optimal option of strengthening in accordance with their impact on the stress-strain state. Prior research have their shortcomings –unsufficient consideration of combined action «soil-reinforcing net» and the use of geosynthetics that are no longer applied [7 – 8].

Highlighting of unsolved before aspects of the common task. Insufficient knowledge about mechanics of the process, lack of specific parameters for usage in various particular engineering-geological conditions, uncertainty of the design methods and so on hinder to widespread introduction of the combined strengthening on the rail transport. Comparison of various technologies integration for strengthening allows making an assumption about possibility of using the different technologies and comprehensive strengthening to enhance the subgrade in different engineering-geological conditions.

Formulation of the objective. Unreinforced subgrade without protective layer is applied in conditions when its construction is structured from noncohesive drainage soil of sufficient bearing capacity, excluding frost-heaving process and having strain modulus at least of 40 MPa (Fig. 1). This type is a major one when overhaul work or modernization at areas where basic platform has no defects and does not require strengthening. When using the machine type RM-80 cross slope of 4 ... 5 ‰ is prepared in one direction.

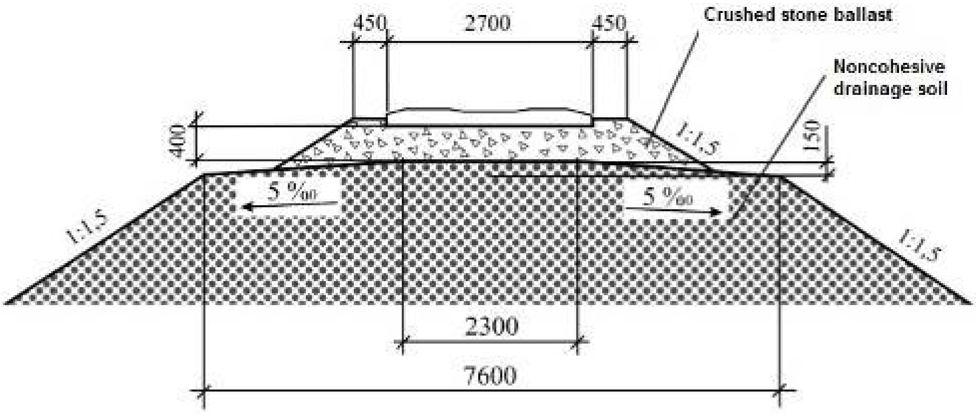

Figure 1 – Subgrade design for single track embankment

When construction repair with the removal of permanent way without the use of ballast cleaning machine, type RM-80 main platform is released from contaminated ballast, leveled by a bulldozer, inequalities are filled up with sand and subgrade is placed.

By modification of formation design (Fig. 1) the option is considered, providing the use of geosynthetics for separation of ballast and subgrade (Fig. 2).

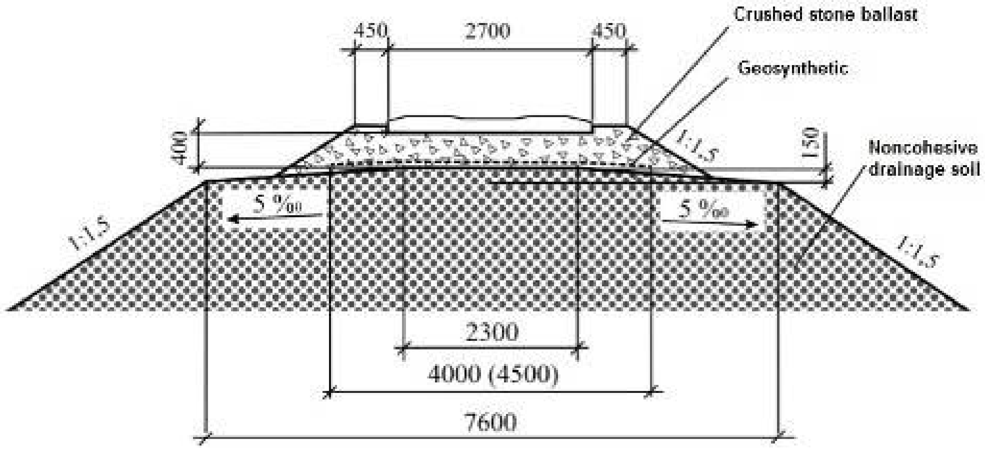

Figure 2 – Subgrade construction with geotextile or geomesh

This design is recommended for subgrade from draining soil, presented with sandy loam or crushed stone mix with elastic modulus E not less than 35 MPa.

Work technology in application of this option is similar to the work technique when using the first option. Geosyntheticsare placed on the prepared basic platform with strain modulus corresponding to project values. Then crushed-stone ballast and rail-sleeper grid are laid over them. The use of geosynthetics with width of 4.2 m is recommended.

When using the machine type RM-80 geosynthetics are placed in mechanized manner by special device. Geosynthetics, used for reinforcement and strengthening of subgrade in order to improve its reliability, have to ensure efficient operation throughout the whole service life (from 50 up to 100 years). This is ensured by correct selection of geosynthetics; pointing (accordingly to the project) of appropriate constructions of strengthening; Regulations for work technology [8, 9].

Constructions for strengthening of subgrade using geosynthetics have to be designed on the basis of engineering-geological and engineering-geodesic surveys. When designing the preset (accordingly to the regulatory requirements) level of reliability of subgrade by strength, stability and deformation capacity throughout the service life of the construction has to be provided. Design solutions must include required calculations justifying structural concept as well as technical-and-economic assessment of their application.

Calculated characteristics of geosynthetics should be accepted in view of their deterioration in design lifetime, including through their aging or damages during laying and operation as well as climatic and biological effects. When designing and calculating constructions with usage of geosynthetics one should take into account the category of the track.

The load on constructions with geosynthetics should be specified taking into account the factor of a possible overload. At this the load from the rolling stock must be taken in view of promising operating conditions of the railway.

The most effective is the construction of strengthening of subgrade using geotextile with bends, inside there are old crashed stone and soil mix (Fig. 3). This design is justified with experimental studies [5, 6] and patented [10].

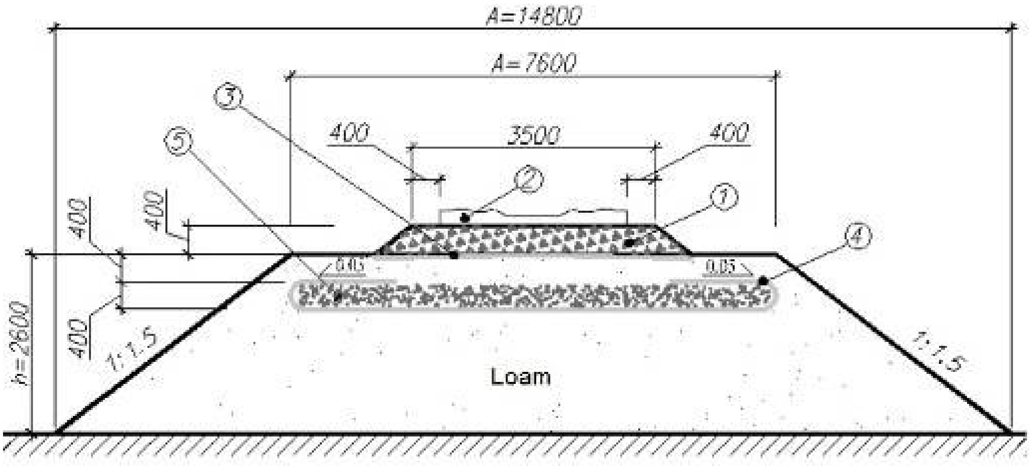

Figure 3 – Combined construction of strengthening of subgrade:
1 – ballast layer; 2 – sleeper; 3 – geotextile under the ballast layer;
4 – strengthening layer of geotextile with bends;
5 – crashed-stone and soil mix

Presented constructions of subgrade can be implemented when conducting overhaul on defective sections of the track and ensure their safe operation. But in the future more attention will be focused on the exactly combined construction of strengthening of subgrade using geotextile and an old layer of crushed stone as the most effective one.

Major material and results. Work technology involves strengthening of deformed embankments by their reinforcing with system of soil-cement piles that are being built accordingly to jet-grouting technology. Such technology of continuous and periodic efforts transmission on stiffest soils excludes unpredictable deformations (subsidences). The theoretical basis for choosing this technology may be the following factors:

– layered character of embankment from soil with various porosity, density and mechanical composition;

– soil layers thickness can vary along the length of the embankment;

– in a base of the embankment there are subsiding soils to be strengthen;

– inability to reuse of soil.

Reinforcement of such embankments is performed in two stages. First, a vertical screen is formed with soil-cement piles of external embankment reinforcement at a composite feeding pressure of 0.1 ... 0.15 MPa. Then soil-cement piles of internal massif are formed at pressure of 0.3 ... 0.3 MPa. This technology eliminates the need of development, transportation and storage for large amounts of material of functioning embankment, exploration and research of soil for the new embankment as well as placing of this soil and layered one, rammer and new embankment compacting.

SSS of reinforcing element is determined on its work as composite, since open-ended shell from geotextile with filler in the form of soil-cement mix (SCM) can be viewed as a three-layer package with significantly different properties of shell and filler.

It should be noted that open-ended shell may be considered as fictitiously closed, it corresponds to the nature of its strain. When increasing the load on the major platform of the model, emerging tensions affect the bends of shell, what is more the topsoil with the additional load leads to restraining effect. Sufficiently long bends allow developing significant friction forces both on outer surface (soil of subgrade is geotextile) and on inner surface (geotextile is SCM). Restraining effect increases proportionally to the load on the main platform of the model and consequently friction increases on pointed surfaces.

In experimental studies [6] in cases of open-ended shell and closed one, it was found that the geotextile surface locking practically did not change SSS of the model, due to the fact that friction at some length of shell ending is enough to develop significant friction and prevent the geotextile from drifting and pulling out. Friction share that in the center of shell, has not so much impact on this effect, therefore open-ended shell with bend is well enough understood as fictitious closed shell (Fig. 4).

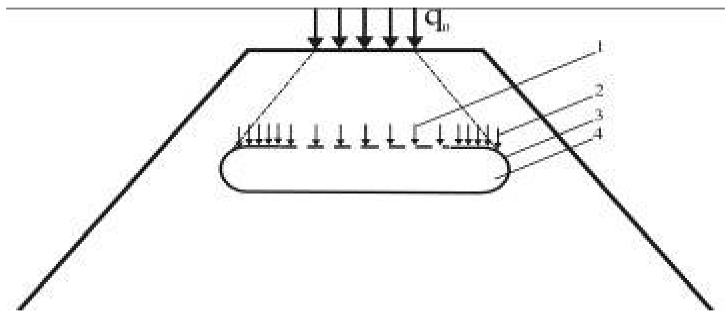

Figure 4 – Diagram of power distribution in view of restraining effect:
1 – load transmitted to the reinforcing element (load portion of the train and net weight);
2 – load transmitted to the bend of shell;
3 – fictitiously closed shell from geotextile; 4 – filler (SCM)

Strain figure of reinforcing element at load impact of the train is changed as follows. Filler as SCM with a high density and is considered substantially incompressible, being separated from the matrix of subgrade with shell is unable to change the volume. Accordingly, the reinforcing element is deformed only with the matrix, and its basis is subgrade with elastic properties (Fig. 5).

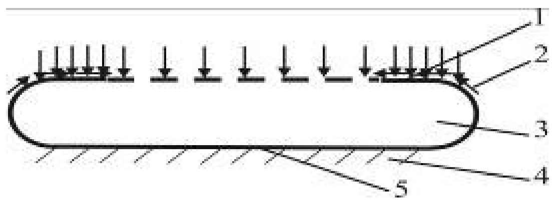

Figure 5 – Diagram of power distribution and friction forces in the reinforcing element:
1 – clamped bend of geosynthetic shell; 2 – friction forces;
3 – compression zone of filler; 4 – elastic foundation;
5 – tensile fibre of geosynthetic shell

The positive effect from strengthening with the combined method is in the fact that almost all the energy of strain goes for an attempt of strain of the reinforcing element. This element due to its design dissipates or distributes energy on the friction forces and uniform distribution of strain on its basis.

Performed calculation of this strengthening in the software package SCAD (elastic setting [11, 12]) allows comparing the experimental results and mathematical simulation. Stress-strain properties[13, 14] accepted for the calculation are presented in Table 1.

After presentation of all necessary parameters model calculation with multu-frontal method was conducted. On completion it was provided the calculation report (SCAD report [15]) concerning successful performance, then results of the calculation are subject to detailed analysis.

Table 1 – Stress-strain properties of solid component of the model

Model element name	Modulus of elasticity, kN/m^2	Poisson number	Volume weight, kN/m^3
Base	10000…35000	0.3	20
Subgrade	10000…35000	0.3	20
Sand	50000	0.2	20
Crushed stone	100000	0.2	20
Sleeper	$3{,}91 \cdot 10^7$	0.2	24.5
Rail	$2{,}1 \cdot 10^8$	0.3	77.0

Geotextiles is used in three variations: 1) flat; 2) with bend; 3) with bend and placement of soil-cement mix in shell. Geotextile is located at a depth of 0.6 m from the slope point of the main platform (Fig. 6). Height of bend is equal to 0.2 m and the length of upper portion of geotextile bend is equal to 0.4 m (Fig. 7).

Characteristics of these geotextiles are as follows: volume weight – 11 kN/m^3; modulus of elasticity – 0.8 MPa; Poisson number– 0.35; plate thickness – 4 mm.

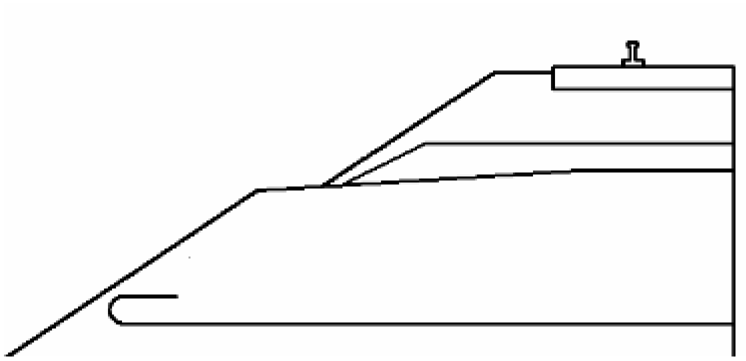

Figure 6 – Placement of geotextile with bend in subgrade

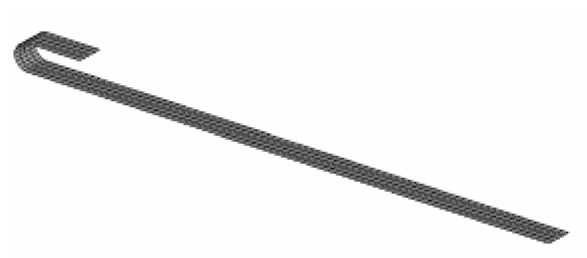

Figure 7 – Model view fragment of geotextile with bend

From obtained results one can conclude that minimum horizontal displacements are observed in the version with deepening of geotextile at 1m and vertical ones at 0.4m. Options of models with geotextile with bend and mineral mix are also under consideration, where position of geotextile changes by edge of the main platform deeper at: 0.4m; 0.6m; 0.8m; 1m.

A typical distribution of stress-strain state when changing the use of geotextile with mineral mix shows that the displacements distribution in all four options is not changed fundamentally, especially as opposed them to options for models with geotextile with crashed stone. The line of added displacements along the location of geotextile was formed in all options (Fig. 8). Distribution of such displacements decreases with the deepening of geotextiles into subgrade.

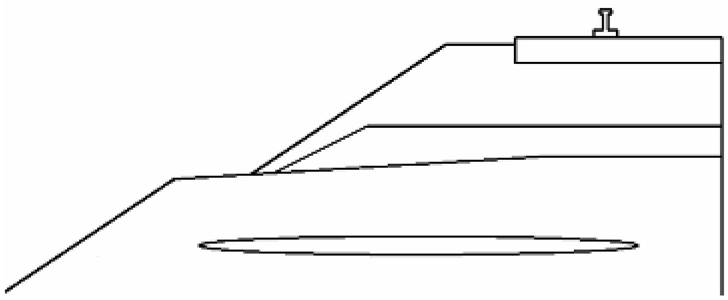

Figure 8 – Typical placement of added horizontal displacements in options of models with geotextile and with mineral mix

All results of maximum horizontal displacements are in Table 2.

Table 2 – Results of strain state by depth of placement

Depth of placement	Maximum horizontal displacements, mm	
	Subtract	Added
0.4	-9.15	3.69
0.6	-8.92	3.76
0.8	-8.65	3.83
1.0	-8.36	3.88

A typical distribution of strain state when changing the use of geotextile with mineral mix also shows that distribution of vertical displacements in four options of models is the same. Isolines distortion is observed in all options of models in the location of geotextiles with mineral mix.

Obtained results when using models with geotextile with bend and mineral mix are follows: the maximum subtract displacements by depth of placement are equal to: 0.4 – 65.98 mm; 0.6 – 66.73 mm; 0.8 – 67.3 mm; 1 – 67.73 mm.

For visual comparison of two options effectiveness for filling of geotextile, graphs of displacements dependency were built in two versions (Fig. 9 – 11).

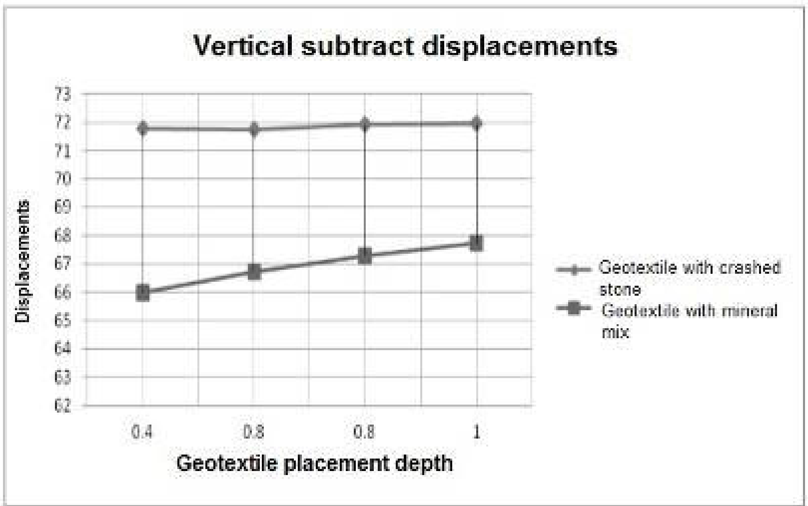

Figure 9 – Dependence of vertical subtract displacements (mm) on geotextile placement depth (m)

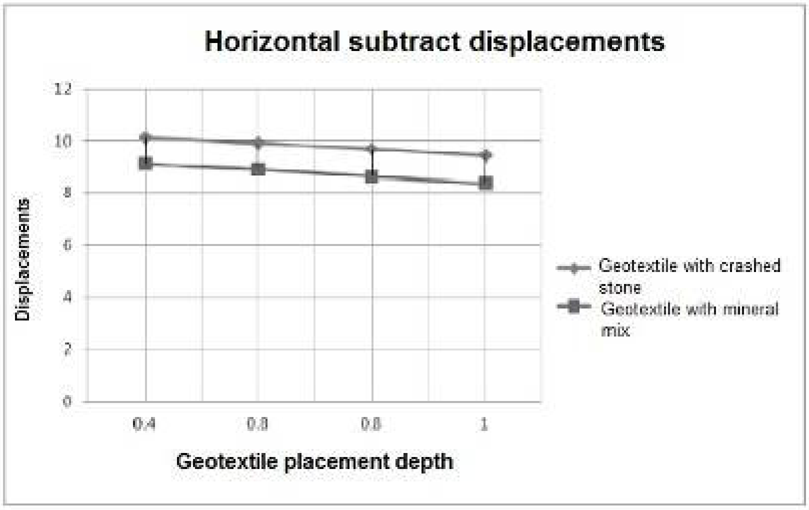

Figure 10 – Dependence of horizontal subtract displacements (mm) on geotextile placement depth (m)

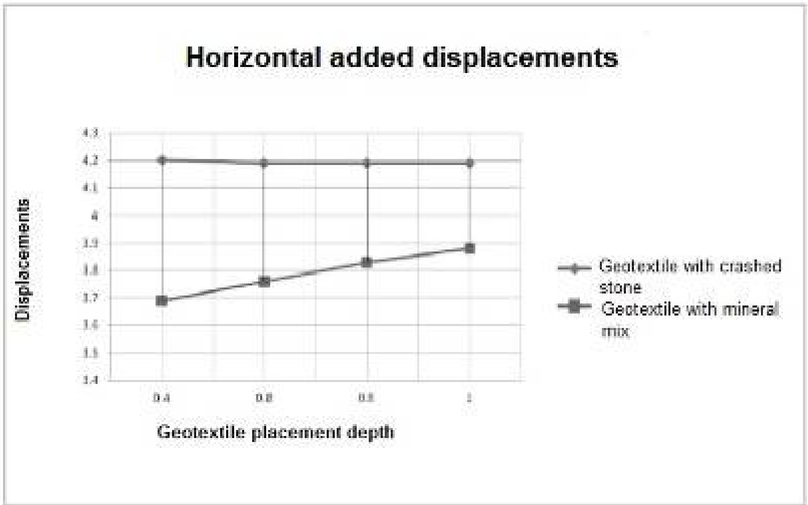

Figure 11 – Dependence of added horizontal displacements (mm) on geotextile placement depth (m)

Conclusions. A few models of strengthening of subgrade differing by depth of cover of geotextile and filler were proposed. Calculations of soil models in the software package SCAD verified themselves properly as actual research.

From obtained results one can conclude that minimum horizontal displacements are observed in models with deepening of geotextile at1 m and vertical ones at 0.4 m. Analysis of grapfs confirms that dependences are linear ones and their nature indicates that the option of geotextile with bends and mineral mix when location at various depths reduces deformation parameters more effectively.

Based on simulation results it is possible to make recommendations concerning modernization of existing subgrade and construction of new one in complex engineering-geological conditions.

One of the promising directions for future research is improvement of strengthening of subgrade based on obtained knowledge at experimental research and simulation. A series of experiments separately with strengthening using soil-cement piles and combined method is planned to perform.

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Received 10.02.2017

UDC 622.692.23:622.691.4

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DESIGN PECULIARITIES OF OIL STORAGE TANKS IN COMPLEX GEOTECHNICAL CONDITIONS AT SEISMIC EFFECTS

Problematic issues of construction and operation of oil storage vertical steel tanks in complex geotechnical conditions, including the seismically unstable territories are systematized. The technique of seismic danger decreasing (increasing the seismic stability of the ground) for ensuring the accident-free operation of tanks during earthquakes of various intensities is proved.

The practical experience of design solutions of the highly effective systems «man-made grounds – foundation – tank» in complex geotechnical conditions for static and dynamic effects (earthquakes, emergency technogenic loadings, etc.) is given.

Keywords: *seismic effects, seismic resistance, oil storage tank, complex geotechnical conditions, man-made grounds, soil-cement elements.*

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ОСОБЛИВОСТІ ПРОЕКТУВАННЯ НАФТОВИХ РЕЗЕРВУАРІВ У СКЛАДНИХ ІНЖЕНЕРНО-ГЕОЛОГІЧНИХ УМОВАХ ПРИ СЕЙСМІЧНИХ ВПЛИВАХ

Систематизовано проблемні питання будівництва та експлуатації нафтових вертикальних сталевих резервуарів у складних інженерно-геологічних умовах, у т. ч. на сейсмічно небезпечних територіях. Обґрунтовано методику зниження сейсмічної небезпеки (підвищення сейсмічної стійкості ґрунтової основи) з метою забезпечення безаварійної експлуатації резервуарів у разі землетрусів різної інтенсивності.

Наведено практичний досвід проектних рішень високоефективних систем «штучна основа – фундаменти – резервуар» у складних геотехнічних умовах для статичних і динамічних впливів (землетрусів, аварійних техногенних навантажень тощо).

Ключові слова: *сейсмічні впливи, сейсмостійкість, нафтовий резервуар, складні інженерно-геологічні умови, штучна основа, ґрунтоцементні елементи.*

Introduction. The development of the oil and petrochemical industry is connected with the necessity of building a significant number of storage tanks for raw materials and completed products. Vertical steel tanks (VST) are the constructions that provide not only the storage of oil and oil products in raw material bases, refineries but the safety and continuity of supply of products in main trunk pipeline system also. At the same time tanks are constructions of an increased danger. The accidents with tanks are followed by a flood of huge mass of liquid that can lead to catastrophic consequences with losses of human lives, due to violation of normal modes of operation the objects of transportation and storage of oil and oil products, and to significant environmental pollution and serious economic consequences also.

Construction areas for VST are often characterized by complex geotechnical conditions. For example, on soft soils which are widespread in the territory of Ukraine. The increasing of volumes of VST construction has been observed. At the same time pressure which is transferred to the ground also considerably increased. Therefore, the cost of modern VST has considerably increased in complex geotechnical conditions. The possibility of maintaining the design production requirements also becomes more complicated as its operation due to ground differential settlements. That's why it is necessary to create new geotechnical technologies that would minimize risks and provide accident-free operation of modern VST, especially in complex geotechnical conditions.

In the territory of Ukraine, including its platform part, there are danger local strong earthquakes reaching more than 5 points (more than 6 points on MSK-64 scale) according to the modern seismological researches [1]. It creates additional danger of operation of the existing and new oil storage tanks.

During the design of oil storage tanks which are objects of the increased responsibility according to Ukrainian codes [2] it is necessary to consider 1% probability of excess the calculated seismic intensity within 50 years. This factor increases the possibility of accident-free operation and respectively a cost and complexity of construction of these engineering constructions. It is necessary to carry out additional calculations and to develop the relevant constructive decisions on minimization of risks during accidents in case of an earthquake.

Analysis of recent sources of research and publications. The analysis of world and domestic experience of usage the various methods of decreasing the dynamic and vibration effects on soft soil [2 – 8] has shown that the most effective option for its transformation is cementation by the means of jet or mixing technologies. The primary feature of these technologies is that they allow strengthening practically all soil types. At the same time there is destruction and simultaneous soil mixing with cement in the «mix-in-place» mode. During soil reinforcing there are strong connections between firms particles are being established. These connections increase the soil strength and reduce the soil compressibility.

The effect of such soil reinforcing is that a certain volume of soft soil is replaced by low compressibility material (soil-cement with big module of deformation, E=70–200 MPa). The natural soil is clamped between vertical soil-cement elements also raises its mechanical characteristics due to impossibility of lateral expansion. The module of deformation of the man-made grounds is considered to be average value between soil-cement and nature soil [5, 7, 11, 12]. Its value can be regulated due to change of distance between such elements.

Identification of general problem parts unsolved before. Nowadays there is almost no experience of oil storage tanks operating in the complex geotechnical conditions on man-made grounds, especially under the action of dangerous geological phenomena like earthquakes.

Building the responsible structures in complex geotechnical conditions taking into account seismic effects is one of the most difficult tasks of geotechnics. Therefore, **the aim of work** is the analysis of geotechnical solutions of construction of RVS on collapsible and soft soil in seismic areas and development of an effective type of the seismic resistance man-made grounds.

Basic material and results. VST in the normal mode of operation are in complex stress-strain state (SSS). The SSS of the VST elements arises at the stage of construction and installation works. Further stress increasing in tank elements is a consequence of the operational loadings (hydrostatic, overpressure, vacuum, snow, wind and temperature load). Consequence of stress increasing is the ground differential settlements on area and perimeter of tank foundation. Therefore, for providing production operational requirements for oil storage tanks on soft soil it is necessary to use the pile foundation or different types of man-made grounds.

During the design of tanks in seismic areas with intensity higher than 6 points it is necessary to consider the additional requirements: 1) use of tanks with lower height; 2) in tanks with a floating roof or a pontoon to apply locks of soft type; 3) in case of usage the tanks with a stationary roof it is necessary to carry out calculation of the maximum height of filling of the tank with liquid to avoid hydrodynamic blow in a roof the wave arising in the tank from a horizontal push, and others. For tanks with a floating roof or a pontoon one should consider horizontal inertial forces from a floating roof or a pontoon.

Tanks need to be calculated on a resistance to overturning and displacement from wind loads, on differential settlements of ground and on seismic effects. Tanks foundation calculated for two groups of limit states: 1) ultimate limit state – on the bearing capacity for check of stability of tanks on overturning; 2) serviceability limit state – on deformations (absolute vertical settlements of the center and a contour circle of the foundation, differential settlements of ground taking into account local moistening of collapsible thickness, tilt). Calculation of ground bearing capacity, the vertical settlements, tank tilt is similar to other buildings and structures according to requirements of norms [9]. Medium settlements of the contour circle of the foundation for tanks up to 30000 m^3 should be no more than 20 cm, for tanks with volume 30000 m^3 and above – no more than 30 cm [10].

The total design scheme for determination seismic resistance of the tank is shown in Fig. 1.

In earthquakes conditions there is an addition to external vibrations further load of the product on the wall and bottom of the tank, namely: 1) hydrostatic loads and loads of overpressure; 2) impulsive (inertial) component of hydrodynamic pressure; 3) convection (kinematic) component of hydrodynamic. The impulsive component of pressure arises from a part of the product moving in an earthquake together with a tank wall. Fluctuations of liquid in the tank create convective pressure and leads to emergence of waves on a product surface. Vertical fluctuations of a tank ground also induce additional load of his wall.

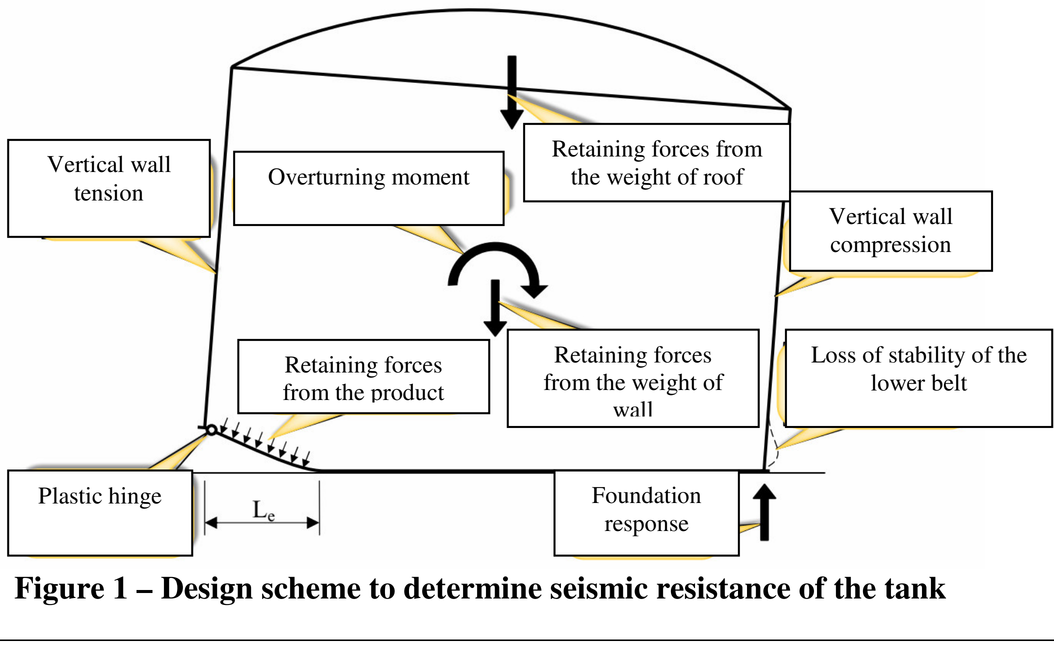

Figure 1 – Design scheme to determine seismic resistance of the tank

Tank's seismic resistance is consider to reached of: a) the tank doesn't overturn during an earthquake (overturning criterion is the limit state at which on the external radius of the lifted part of the bottom a full plastic hinge appears); b) stability of the lower belt of a wall at action of longitudinal and cross loading is provided; c) durability condition for all bearing elements of the tank is provided.

The main author's idea is development of the universal man-made grounds which will be able to provide standard and production requirements for oil storage tanks on soft soil as for static service conditions and in case of action of seismic influences of various intensity.

Reduction of dynamic load influence by a superstructure in case of earthquakes can be reached due to reduction of the ground acceleration and vibration amplitude. One of options of reduction the seismic intensity is an increasing the seismic rigidity $V_s\rho$ of an active soil layer due to increase the speed of distribution in it seismic waves. Such effect can be reached due to increase the ground elastic deformation characteristics using soil mixing technology [4, 8]. At such approach it is possible to raise the ground module of elasticity to 500 – 2000 MPa, the speed of waves distribution to 600 – 1000 m/s at the constant density.

For example, there are given geotechnical solutions of the oil storage tank. Technological parameters of the tank are given in Table. 1. Geometrical tank parameters: 1) nominal volume 20000 m^3; 2) gross space 20956 m^3; 3) wall height 17,926 m; 4) inside diameter 39,9 m; 5) product-surface area1250,4 m^2.

The foundation diameter is about 40,5 m. The pressure under the foundation at hydro testing is 168,14 kPa, at operation – 180,86 kPa. Uniformly distributed load on a base contour at hydrotesting is 31,65 kN/r.m., at operation – 40,33 kN/r.m., at wind load – ±6 kN/r.m., at seismic influence – +353,73/ -268,67 kN/r.m. The value of the seismic horizontal forces that transmitted to the tank foundation is 65500 kN.

Table 1 – Technological parameters of oil storage tank

Parameters	Value
Product density (oil), t/m^3	0,89
Expected level of product filling, m	16,2
Water level at hydrotest, m	16,75
Internal overpressure	missing
Standard internal vacuum	missing
Operation rate (cycles per year), min/max	20/100
Characteristic value of snow load, kg/m^2	102
Characteristic value of wind load, kg/m^2	51
Seismic intensity, points	8
The temperature of the coldest days with the use factor 0,98, ^{0}C	– 24
Maximum temperature of oil storage, ^{0}C	+ 25
Design service life, years	40
Design wave altitude of oil at seismic loadings, m	0,32
The size of an allowance for corrosion for sheets of a wall, mm	0

Two sites with complex geotechnical conditions are considered. Level of ground waters is at a depth of 8,6 – 8,8 m from Earth's surface level. Complexity of the first site is characterized by soft loams and sandy loams (the module of deformation is E = 3 – 5 MPa) of 12 – 13 m thickness at 7,6 m depth. Below a depth of 21,7 – 23,9 m there are solid and semi-solid clay (Fig. 2). Soft soil during seismic influences could be liquefied (thixotropic properties), to receive additional consolidation therefore, there will be additional deformations of the foundation tank. Complexity of the second site lies in the existence of collapsible thickness more than 5 m. Soil seismic properties category is III. Standard seismic intensity of construction sites is 8 points (according [2]), calculation – 9 points.

In such conditions such geotechnical options were considered: 1) penetration of soft and collapsible soil thickness using piling foundation (a section of 350×350 mm), dynamic deep soil compaction in space between piles (a diameter of elements is 300 mm), concreting pile cap of 0,7 m thick, piles connection with a cap is hinged; 2) the same, as in the first option, but over the piles is compacted gravel for the purpose of damping of the tank fluctuations and for avoidance of transfer of horizontal loads on piles; 3) reinforcing of soft and collapsible soil by vertical soil-cement elements with a diameter of 500 – 650 mm (man-made grounds using soil mixing technology), further the same, as in the second option.

Perception of horizontal seismic loading due to work of piles is provided only at their significant amount (~ 1000 pieces). It is economically inexpedient as vertical load of piles will make no more than 35% of admissible. Therefore this option wasn't compared further.

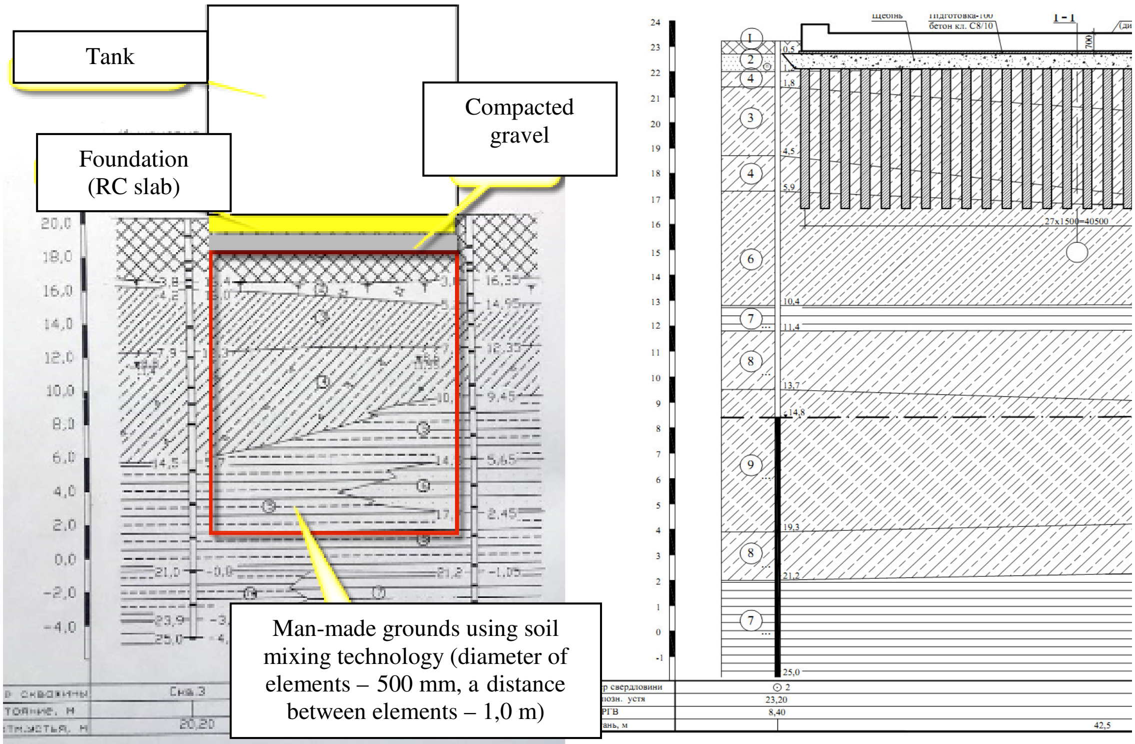

Figure 2 – Soil profiles (left – soft loams and sandy loams of 12 – 13 m thickness; right – collapsible thickness more than 5 m)

Option with vertical soil-cement elements was much cheaper and it was possible to implement it faster.

Length and diameter of soil-cement elements and distance between it were defined by an iterative method. Providing smaller critically admissible values of the settlement of center and extreme foundation points, tilt, and the ground bearing capacity at seismic influences was the main criterion of calculation. As a result of calculations it is established that optimal diameter of elements – 500 mm, a distance between elements – 1,0 m (2d). Informative geotechnical model of the tank is shown in Fig. 3.

The SSS modeling of the «soil – reinforced ground – compacted gravel for damping – foundation – tank» system is carried out. The problem was solved in 2D («axisymmetric») scheme using the finite elements method (FEM), taking into account seismic effects. The soil, the man-made grounds and crushed-stone pillow are clusters with the appropriate characteristics. The foundation is a beam element with the corresponding flexibilities EI and normal rigidness EA for a plate 0,7 m thick. The tank is beam elements; oil is a cluster with the appropriate characteristics. The design axially symmetric scheme is provided on Fig. 4.

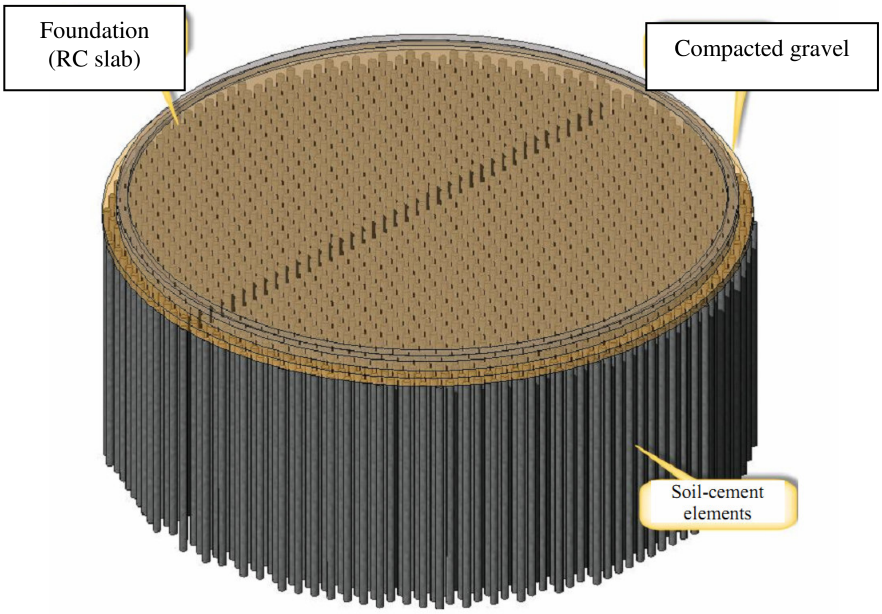

Figure 3 – 3D design scheme of the system «reinforced base – crushed stone pillow – foundations»

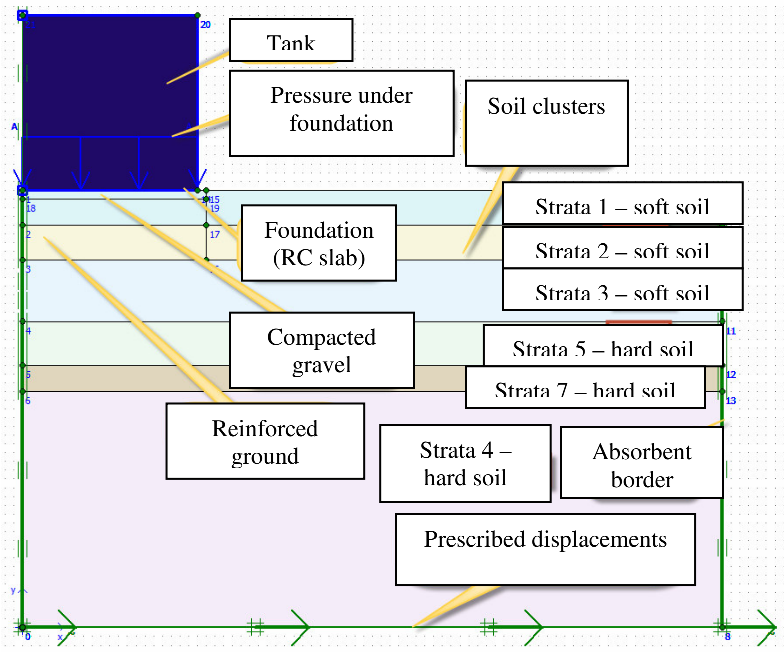

Figure 4 – Design axially symmetric scheme of the system «soil – reinforced ground – damping compacted gravel – foundation – tank»

The sizes of a calculation zone around a construction were accepted from a condition of prevention of its influence on results. The depth of the calculation zone is 50 m (on this depth there is a rocky or semi-rocky deposits). The fluctuations at the bottom boundary in the form of an earthquake acceleration diagram are set (with the parameters corresponding to intensity of seismic influences in 9 points). At the far vertical boundaries, absorbent boundary conditions are applied to absorb outgoing waves. In this way the boundary conditions as described above are automatically generated. For soil was using linear model. Influence of hydrostatic water pressure using appropriate ground water level. Damping of the building and the ground is simulated by means of Rayleigh's coefficients. The horizontal prescribed displacements is set to $u_x = 0{,}01$ m at the bottom boundary. The vertical component of the prescribed displacement is kept zero $u_y = 0$.

According to the analysis of results of calculations, modeling and comparison of options it is possible to make the following **conclusions:**

1. The foundation on the man-made grounds, using soil mixing technology which turns soft and collapsible ground into composite material, is a seismic resistance option. It cost less than pile foundation and at the same time it is more technological. All production and standard requirements imposed to tanks operation are satisfied.

2. Due to the reinforcement of soft and collapsible soil the tank's vibration amplitude decreases, the soil accelerations decrease in the bottom of foundation. Such result is provided with increase in speed of distribution of seismic waves in the man-made grounds using soil mixing technology, and also due to increase in strength characteristics and the module of deformation.

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

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Received 21.02.2017

UDC 624.131

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OPTIMIZATION OF SLABS REINFORCEMENT DEPENDING ON THE DEGREE OF BASE COMPLIANCE

In the work it was made analysis of stiffness influence of upper basis construction and compliance of basics on stress–strained state of monolithic slabs and capabilities to simplify the design scheme of slabs. Two building options were considered: the one with different design: frame and frameless, the second with homogeneous and heterogeneous bases. It is established that in frame buildings only incomplete modeling of slabs shows adequate quality and does not make significant changes in constructive solution in comparison with the full simulation. For frameless buildings ignoring rigidity of upper basis and constructions compliance structures and foundations not only leads to some excess reinforcement plates, but also to the quality discrepancy reinforcement, particularly in patchy basis.

***Keywords:** system building–foundation–base, collaborate, building rigidity, foundation yielding, subsidence, tensely deformed state (TDS).*

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ОПТИМІЗАЦІЯ АРМУВАННЯ ПЛИТ ПЕРЕКРИТТЯ В ЗАЛЕЖНОСТІ ВІД СТУПЕНЯ ПІДДАТЛИВОСТІ ОСНОВИ

Проведено дослідження впливу жорсткості надфундаментної конструкції та піддатливості основи на напружено–деформований стан монолітних плит перекриття і можливості спрощення розрахункової схеми плити. Були розглянуті два варіанти будівель з різним конструктивним рішенням: повнокаркасна і безкаркасна, а також два варіанти основи: однорідна і неоднорідна. Встановлено, що для каркасних будинків неповне моделювання тільки плити перекриття дає адекватну якісну картину та не вносить суттєвих змін до конструктивного рішення у порівнянні з повним моделюванням. Для безкаркасних будинків ігнорування жорсткості надфундаментних конструкцій і піддатливості основи може призвести не тільки до деякого переармування плити, але і до якісної невідповідності армування, зокрема при неоднорідній основі.

***Ключові слова:** система будівля–фундамент–основа, сумісна робота, жорсткість будівлі, піддатливість основи, осідання, напружено–деформований стан (НДС).*

Introduction. Elements stresses in buildings and bases deformations are recommended to be determined by calculation of joint foundations, bases and overlying structures work, considering basics depth and plan heterogeneity [1, 2]. These calculation results are achieved in more accurate simulation of the stress–strain state (SSS) of ground base and overlying structures. As building stiffness influence on SSS and the base settling, as the base compliance and nonlinear properties affect the stresses redistribution in foundations and overlying structures. Therefore, relationship among calculation model of building and soil base is important.

In works [3–6] it is noted that the need for joint calculations of the building with the foundation is especially important for the current level of building science development. By the introduction of modern calculation methods and the newest materials, it could be possible to design building structures with minimal strength reserves. In such conditions, slight increase in stresses due to the joint operation of the buildings and bases can lead to the cracks appearance and reduction in the overall reliability of the structures.

But such calculation is associated with considerable time consuming for compiling computational model, and also requires significant computer memory resource. Sometimes, in order to simplify the calculation of monolithic slabs, only the slab itself is modeled with the elements on which it relies, without considering the work of other overlying structures and the compliance of the base.

Research results. Joint calculations are performed using computer technology, usually the finite element method. In many cases, buildings are complex in plan configuration and stiffness distribution, so usually it is quite difficult to assess the impact of various factors (stiffness of different above–ground structure elements and foundations) for calculation results.

For the analysis two primary design scheme of houses are selected:

– frameless with bearing walls of brick;

– frame with monolithic bearing structures.

For both schemes complex joint calculation of above–ground buildings, foundations and soil base are executed in different variants of soil layers:

– location of soil layers close to horizontal;

– there is a pinch of soil layers with different modules deformation.

As basis for the simulation of design schemes basic variants, the design documentation for the frameless residential section of size 25×15 m in the axes was selected. Total height of 9–storey section with basement and technical floors is 30 m. The main walls are from brick, the walls of the ground floor are from concrete block, foundations are monolithic tape up to 3.6 m widths.

In simulation of framework scheme bearing walls above the 0.000 level have been replaced by monolithic columns. Basement and foundations were left unchanged.

As homogeneous base with horizontal placement of the layers were made of random geological conditions of the construction site with the average characteristics of soils. Characteristics of soil area are presented in Table 1. For the simulation of heterogeneous soil base wedging out, soil layer with low deformation modulus of E = 8 MPa was used.

Calculation of 3D computer models is performed using the software package «Lira–CAD» which is a computer system for structural analysis and design. The program was developed by the Scientific Research Institute of Automated Systems in Construction (NIASS), Kyiv, Ukraine. The complex allows to perform spatial calculations of building systems considering the heterogeneity of the base in plan, its depth, influence of neighboring buildings and structures.

For each variant, calculations of monolithic slab are performed both for 3D computer model and simplified slab plate model, taking into account support conditions with the determination of the stress–strain state and the reinforcement design.

Table 1 – Physical and mechanical characteristics of the building site ground

Soils name	Layer power , m	Specific gravity γ, kN/m^3	Humidity			Specific gravity of ground particles γ_s, kN/m^3	Porosity ratio e	Degree of humidity S_r	Plasticity number I_P	The flow rate I_L	Specific adhesion c, kPa	Ext. friction angle φ, degree	Modulus of deformation E, MPa
			w	w_L	w_p								
Vegetative layer of soil	0,9	16,4	0,13	–	–	–	–	–	–	–	–	–	–
Sandy loam silty	4,8	18	0,17	0,2	0,15	26,8	0,70	0,62	0,05	0,4	11	21	25,8
Soft clay loam	2,7	17,5	0,17	0,21	0,12	26,6	0,78	0,58	0,09	0,55	5	18	22,6
Medium size sand	5,2	20	0,18	–	–	26,5	0,56	0,88	–	–	2	38	39

General view of computer models of frameless and frame construction in the program «Lira–CAD» is shown in Fig. 1, 2.

For the modeling of the soil massif, the connection among the «Lira–CAD» and the sub–program «Grunt» («Soil») was used. To do this, before the simulation began, soil characteristics were introduced into the «Grunt» program and «boreholes» were created setting the layers thickness.

Fig. 3, 4 shows the models of ground base with horizontal placement of seams and with the wedging of seams with different deformation modules.

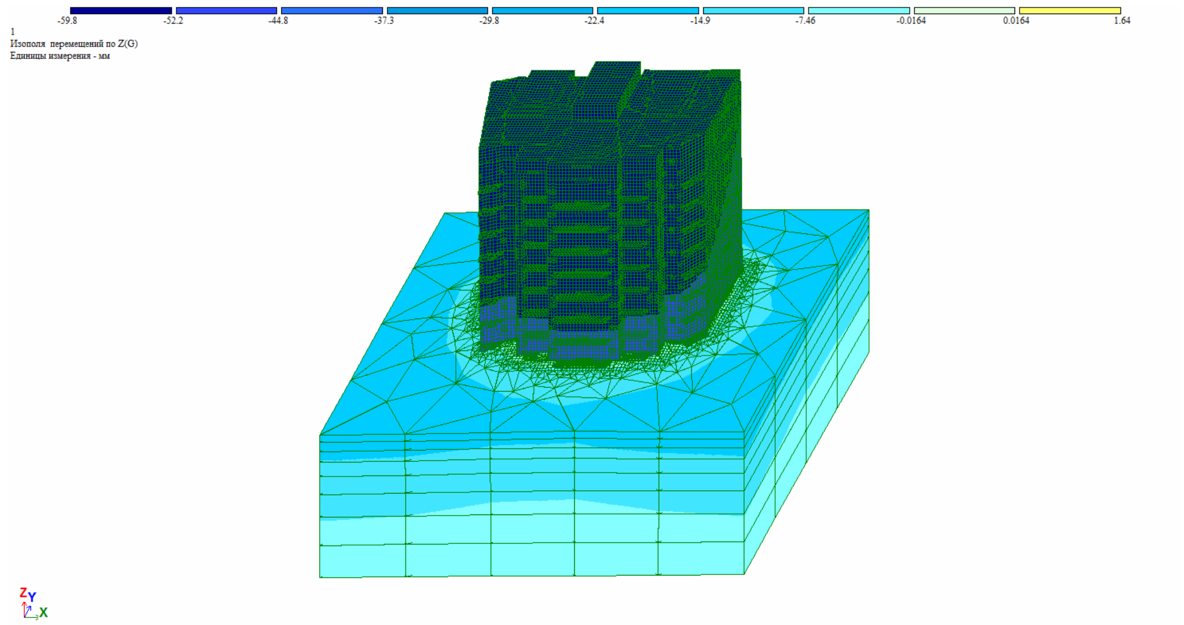

Figure 1 – General view of the computer model of a frameless building

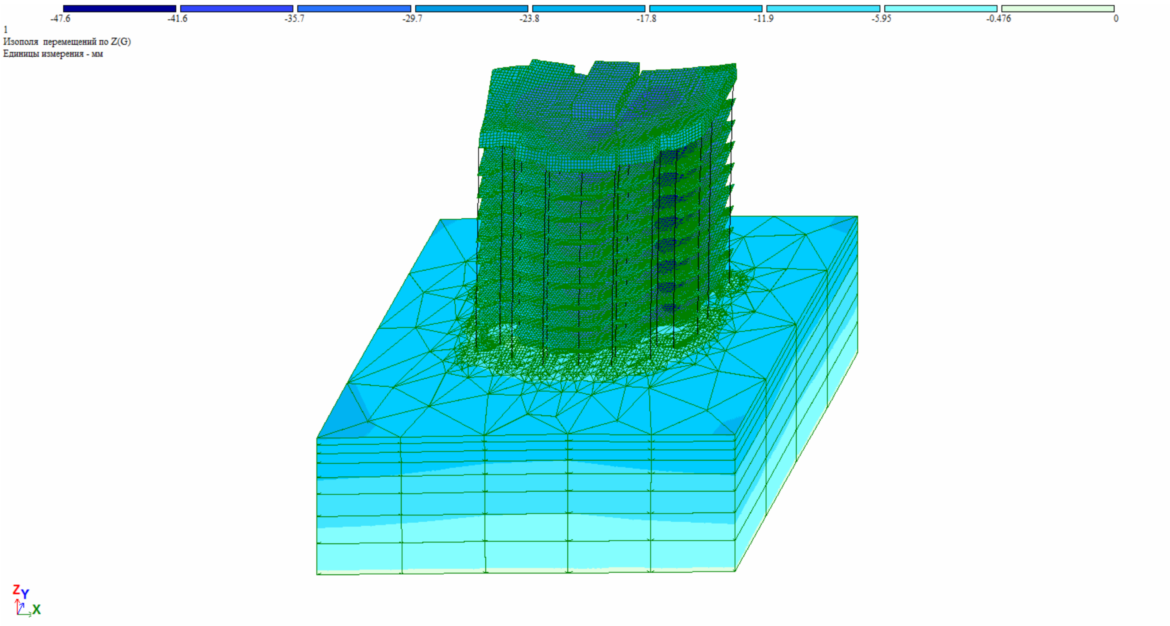

Figure 2 – General view of the computer model of building with full frame

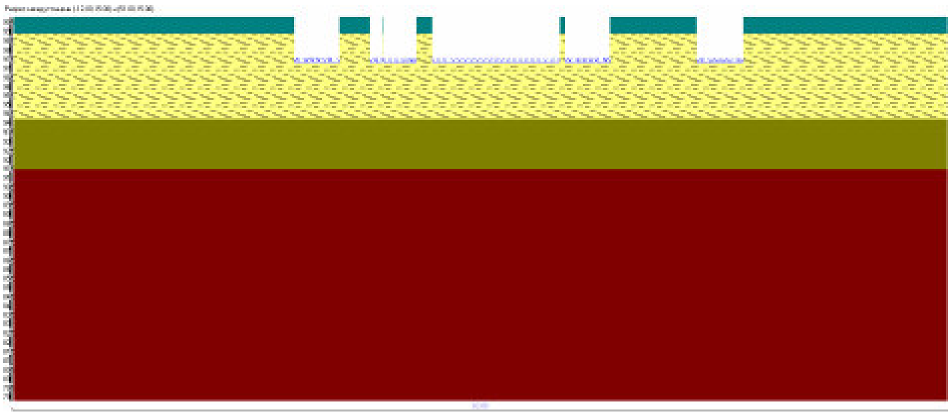

Figure 3 – Model of soil base with horizontal arrangement of seams

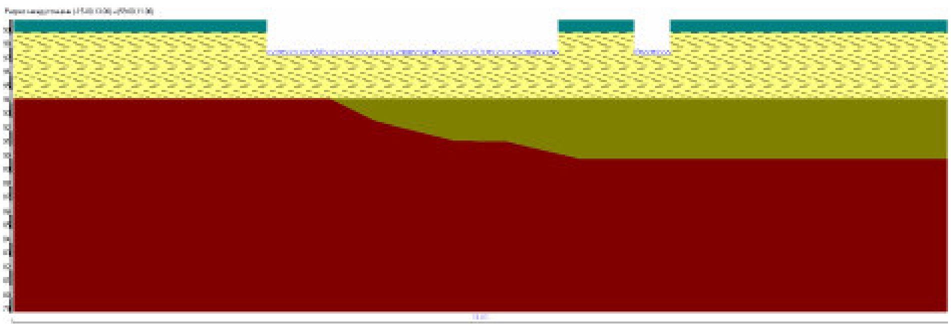

Figure 4 – Model of ground base with wedging out layers

Analysis of the calculations results showed:

– for a frameless building, the rigidity of the skeleton and the compliance of the base, substantial leveling of the displacements in the slab are cosidered;

– for a frame building such influence is very insignificant;

– taking into account the rigidity of overlying structures and the compliance of the base leads to a lower concentration of forces in the slabs;

– taking into account the heterogeneity of the bedding of the base has little effect on the required amount of reinforcement, but leads to some qualitative inconsistency of the reinforcement.

– in the case of homogeneous basis, the calculation of overlapping slabs can be performed without consideration the rigidity of the overlying structures and the compliance of the base with the assurance of reliable operation.

To analyze the economic aspects of the results joint foundations, bases and overlying structures work, considering basic depth and plan heterogeneity and simplified modeling, the working reinforcement of the slabs in all the variants considered was compared.

Calculation of the reinforcement is carried out in the program «Lira–Arm». The result of the program is colour diagrams of reinforcement with scale where such features are indicated: from above – the pitch of the rods of a certain diameter, from below – the area in cm^2, which should be located on 1 m of the length of the plate. Diagrams are obtained for two directions of reinforcement of the lower and upper zone of the plate.

For example, in Fig. 5–7 the coloured diagrams of the reinforcement of frameless building slab lower zone are shown. It can be seen from the figures that in the calculation of a plate without consideration the influence of the skeleton rigidity and the compliance of the base, excessive reinforcement takes place.

It has to be considered that heterogeneity of the base bedding has little effect on the required amount of reinforcement, but leads to some qualitative inconsistency of the reinforcement.

The reinforcement is accepted by mesh in upper and lower zone, with additional mesh in the stress concentration zones, as well as with additional mounting frames and transverse reinforcement.

After processing the results of calculations in the «Lira–Arm» program for the ultimate limit state, the slab reinforcement for all simulation schemes is presented in Table 2.

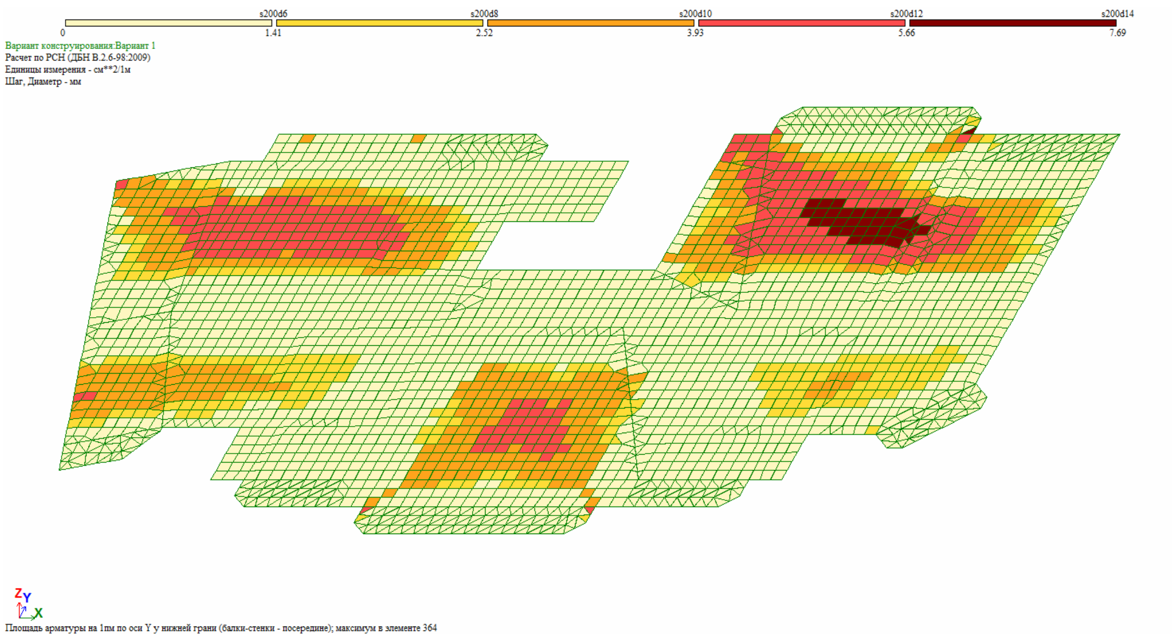

Figure 5 – The diagram of the slabs lower zone reinforcement for a frameless building, determined without consideration base compliance and frame rigidity

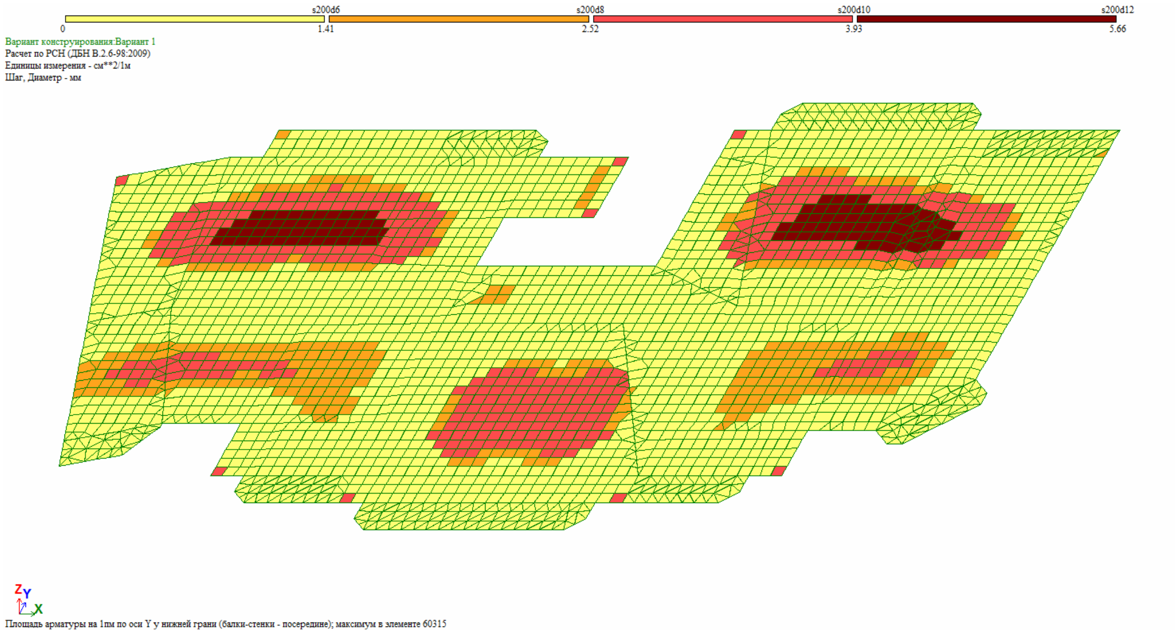

Figure 6 – The diagram of the slabs lower zone reinforcement for the frameless building, defined considering the homogeneous base compliance and the frame rigidity

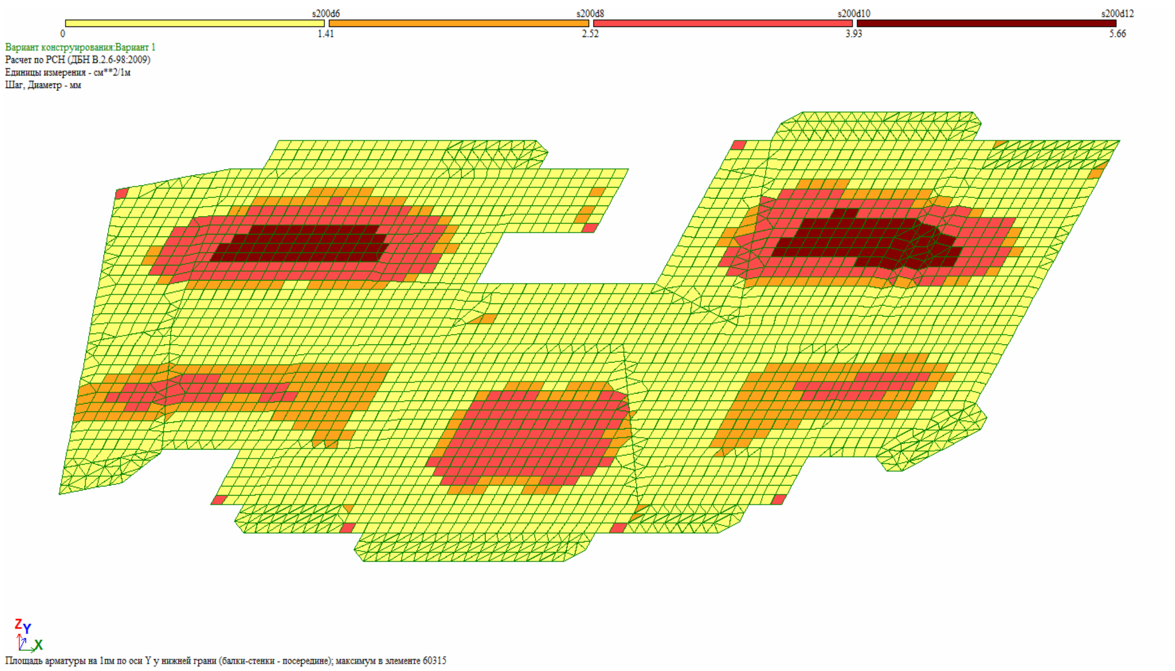

Figure 7 – The diagram of slabs lower zone reinforcement for a frameless building, defined considering the heterogeneous base compliance and the frame stiffness

The outlays based on the obtained results were compiled in the program «AVK 5.3» for each variant, graphs of the amount of reinforcement, cost and labor intensity depending on the modeling method were plotted (Fig. 8 – 9).

As can be seen for the frameless building, for the case of full simulation all the indicators are much better than only from simplified slab modeling.

For a frame building with full simulation, all the indicators are also better than only from simplified slab modeling , but the difference is smaller than in the previous case.

Table 2 – Results of slab versions reinforcement

Direction of rods location	Main reinforcement		Additional reinforcement		Total weight, t
	Along the lower edge	Along the upper edge	Along the lower edge	Along the upper edge	
A building with a full frame, simulation without considering the base compliance and frame rigidity					
Along the Y axis	Ø10 A400C	Ø14 A400C	Ø10 A400C Ø16 A400C	Ø12 A400C Ø32 A400C	13.28
Along the X axis	Ø10 A400C	Ø14 A400C	Ø8 A400C Ø18 A400C Ø22 A400C	Ø16 A400C Ø32 A400C	
The building with a full frame, a simulation considering the homogeneous base compliance and the frame rigidity					
Along the Y axis	Ø12 A400C	Ø14 A400C	Ø12 A400C Ø18 A400C	Ø16 A400C Ø32 A400C	12.93
Along the X axis	Ø12 A400C	Ø14 A400C	Ø10 A400C Ø18 A400C	Ø12 A400C Ø22 A400C Ø32 A400C	
Frameless building, a simulation without considering the base compliance and frame rigidity					
Along the Y axis	Ø8 A400C	Ø10 A400C	Ø10 A400C Ø12 A400C	Ø14 A400C Ø20 A400C	7.09
Along the X axis	Ø8 A400C	Ø10 A400C	Ø10 A400C	Ø10 A400C Ø18 A400C	
Frameless building, a a simulation considering the base compliance and frame rigidity					
Along the Y axis	Ø8 A400C	Ø8 A400C	Ø10 A400C	Ø10 A400C Ø14 A400C	6.20
Along the X axis	Ø8 A400C	Ø8 A400C	Ø10 A400C	Ø10 A400C Ø12 A400C	
Frameless building, a simulation considering the heterogeneous base compliance and the frame					
Along the Y axis	Ø8 A400C	Ø8 A400C	Ø10A400C	Ø10 A400C Ø14 A400C	5.62
Along the X axis	Ø8 A400C	Ø8 A400C	Ø6 A400C	Ø10 A400C Ø12 A400C	

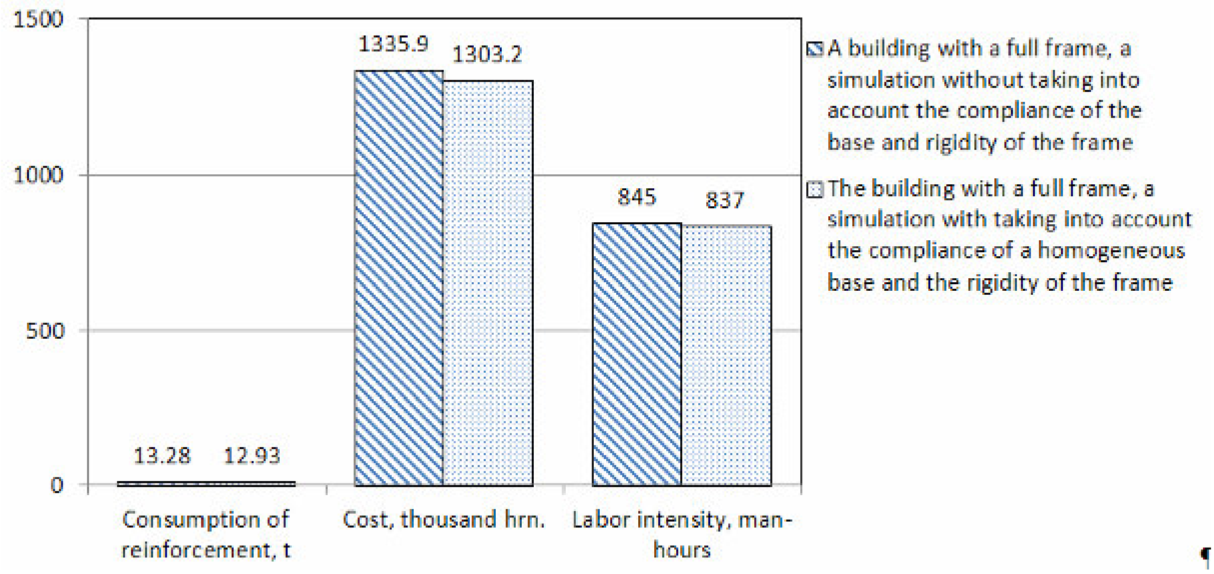

Figure 8 – A building with a full frame

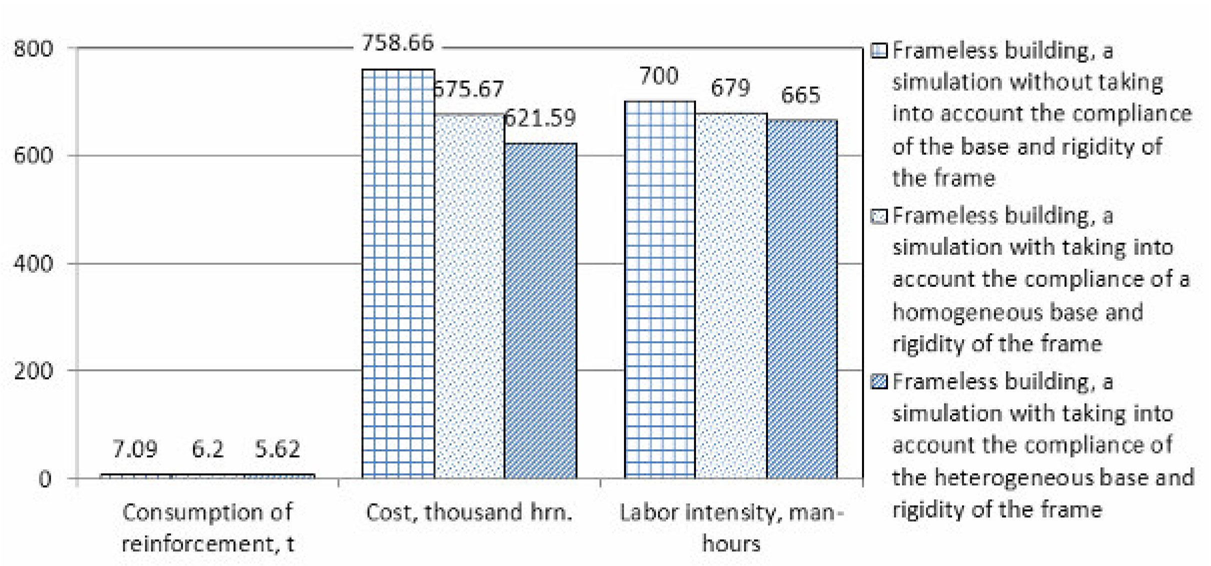

Figure 9 – Frameless building

Conclusions. In overlying structures it has to be considered rigity and the compliance of the base affects on the forces distribution and slabs reinforcement nature.

For frameless buildings, ignoring the rigidity of overlying structures and the base compliance leads not only to a little excrescent in slab reinforcement, but also to qualitative inconsistency of reinforcement, in particular, on heterogeneous basis.

For framed buildings, the simplified slab modeling does not make significant changes in the design solution with full simulation comparison.

In the case of homogeneous basis, the calculation of overlapping slabs can be performed without consideration the rigidity of overlying structures and the compliance of the base with the assurance of reliable operation.

The application of spatial building modeling on compliant basis compared with the simplified slab simulation allows obtaining reinforcement savings in the slab up to 2.5% for frame and up to 14% for frameless buildings.

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Received 01.12.2016

UDC 624.014.042:519.2

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PIPELINE RELIABILITY LEVEL FOR THE DIFFERENT COLLAPSIBLE STRATA

Probabilistic analysis of the loessial collapsible soil deformation modulus in the water saturated for two areas of pipeline laying in the Poltava and Kherson region were performed. Respective distributing laws and statistics were obtained. With help of Ansys finite element simulation was performed in the form of the Monte-Carlo method. Pipeline deformation in the loessial collapsible strata local soaking area was modeled; internal operating pressure and temperature difference were considered. Pipeline failure probability by longitudinal stresses parameter in the fine sand soil was obtained, random functions stochastic apparatus was used. Pipeline failure probability by longitudinal stresses parameter with similar geometric parameters and internal operating pressure simulation results were compared with the base composed by fine sand and loessial collapsible strata 7 and 13 meters.

***Keywords**: buried main pipeline, loessial collapsible strata, longitudinal stresses, reliability level.*

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ІМОВІРНІСТЬ БЕЗВІДМОВНОЇ РОБОТИ ТРУБОПРОВОДУ ДЛЯ РІЗНОЇ ПОТУЖНОСТІ ПРОСАДОЧНОЇ ТОВЩІ

Виконано імовірнісний аналіз модулю деформації лесового просадочного ґрунту у водонасиченому стані для двох ділянок прокладання трубопроводу Полтавської області та Херсонщини. Отримано відповідні закони розподілу та статистики. За допомогою методу Монте-Карло, у формі методу скінченних елементів, проведено імовірнісне моделювання трубопроводу на ділянці локального замочування масиву лесового ґрунту із врахуванням внутрішнього робочого тиску і температурного перепаду. За допомогою імовірнісного апарату випадкових функцій отримано значення надійності трубопроводу прокладеного у мілкому однорідному піску. Порівняно результати моделювання трубопроводу з однаковими геометричними параметрами та внутрішнім робочим тиском для основи складеної мілким піском та просадочною товщою різної потужності 6,2 та 13 м. для трубопроводу Отримано значення імовірності безвідмовної роботи трубопроводу за параметром поздовжніх напружень.

***Ключові слова:** підземний магістральний трубопровід, лесова просадочна товща, поздовжні напруження, імовірність безвідмовної роботи.*

Introduction. Main pipeline linear part MPLP soil basis differential settlements lead to additional longitudinal stresses in the pipeline walls, destruction of anti-corrosion coating and reduce pipeline durability [1 – 6]. In addition, MPLP large deflection may cause formation of gas hydrates in places of «local hollows» that confirms necessity of the different settlements regulation.

Large values of the MPLP differential settlements are typical for pipeline laying in non-standard soil conditions. Non-standard soil conditions are such conditions when there are pipeline layer designed in areas with the following characteristic features [1, 3, 7]: swamp or flooded areas, areas with underground cavities of various nature (mining and mine construction zones, areas with karst cavities, etc.), thawing permafrost areas, landslide territories, seismic zones.

For Ukraine loessial collapsible soils are very common problem, because such soil occupy 65 – 70% of the territory. Such problem is especially urgent for the southern region, where loessial layer reaches 45…50 m, and the value of the soil collapse from its own weight may occur 1…2 m [8 – 10].

Analysis of recent sources of research and publications. MPLP differential deformations in swelling clays investigation show that pipeline, with dimensions 1220 × 22 mm, has longitudinal stress $\sigma_{l} =$ 165 MPa, for soil movements 310 mm on the wave longitude about 50 m [2]. In area of thawing permafrost on the wavelength of 38 m soil local deformations is 158 mm. In those conditions pipeline, with dimensions 1220 × 32 mm, has longitudinal stress about $\sigma_{l} =$ 100 MPa [1]. It should be noted that both of the researches were made basing on the hypothesis of equivalence between soil differential settlements and pipeline deformations [1, 2].

Deterministic calculations for the similar soil conditions typical in Ukraine – loessial collapsible soil showed quite close results [11]. It should be mentioned, that for soil it is used linear model with soil deformational modulus in the natural and water saturated state. For example, in the loessial collapsible 9 m strata settlements which are determined by the engineering technique, S_{slg} = 266 mm pipeline, with external diameter 1000 mm, have follow deformations and longitudinal stresses. Results are presented for different type of contact between soil and pipeline. Bonded contact was chosen as most relevant.

Table 1 – Soil settlements, pipeline deformations and longitudinal stresses

Soaked area length, L, m	Pipeline wall thickness, *t*, mm	Measured Value	FEM Simulation		
			Bonded	Frictional	Contact is absent
50-65 m (water spreading from top to down)	10	S_{slg}, мм	285	285	290
		S_{sl}^{pipe}, мм	284	421	422
		$\sigma_{dif}^{max(min)}$, МПа	+169 -195	+142 -147	+155 -143
	15	S_{slg}, мм	285	285	285
		S_{sl}^{pipe}, мм	285	411	417
		$\sigma_{dif}^{max(min)}$, МПа	+138 -166	+126 -133	+137 -135
	20	S_{slg}, мм	292	285	285
		S_{sl}^{pipe}, мм	288	296	411
		$\sigma_{dif}^{max(min)}$, МПа	+129 -145	+121 -121	+128 -127

Highlighting of unsolved parts of the problem. Pipeline strength in a nonstandard soil conditions is investigated. The next step in improving calculation methodic is to determine pipeline reliability level [4, 12] on longitudinal stresses parameter σ_l. There are four main input random variables RV, which define pipeline reliability by longitudinal stresses parameter: pipeline steel yield strength $\tilde{\sigma}_y$, internal operating pressure value $\tilde{P}$, temperature difference $\Delta\tilde{t}$, uneven settlement value, which almost fully determinate value of the bending stresses $\tilde{\sigma}_{dif}$. Stochastic parameters of the first three factors are investigated. According to geotechnical problems of Ukraine, loessial collapsible soil deformational modulus in the water saturated state and its stochastic parameters are the most interesting parameter for us. Soil local collapsible settlements from its own weight $S_{s\ell g}$ and respective bending stresses σ_{dif} could be obtained with help of Ansys probabilistic simulation.

Purpose of the work is to determine relevant distribution and statistics for the loessial collapsible soil deformational modulus in the water saturated state. Obtain pipeline longitudinal stresses and respective failure probability.

Collapsible soil stochastic characteristics estimation. Engineering data of the geotechnical researches in the Kherson city – Object #1, and Poltava region – Object #2 (fig. 1) are taken as statistical material. Should be noted, that area of the Object #2 is right on the pipeline route, samples were taken during the pipeline isolation repair.

Figure 1 – Loess soil sampling during the pipeline isolation repair works

Statistical data of the Object #1 were taken from 10 bore pits, with 13 m depth. Soil samplings were conducted over 1 meter [9] in rings, with further uniaxial compression test. During the research four geological elements were allocated: Strata-1 – vegetation layer (Loamy black soil, loose, solid); Strata-2 – loess loam, brownish-yellow, carbonate, light silty, in natural state solid in the saturated state fluid, macroporous, collapsible; Strata-3 – sandy loess loam, yellow, carbonate, silty, in natural state solid in the saturated state fluid, macroporous, collapsible; Strata-4 – loess loam, yellow-brown, brown, carbonate, heavy silty, in natural state solid in the saturated state fluid. Strata-5 – noncolapsible loam. During the geotechnical investigations groundwater level wasn't found.

General sampling of the Object #1 physical and mechanical soil properties was 128 elements: where Strata-2 – 52, Strata-3 – 48, Strata-4 – 28. Soil layers thickness were assigned as following: Strata-1 – 0.6 m, Strata-2 – 4.6 m, Strata-3 – 4.9 m, Strata-4 – 2.9 m, total 13 m (fig. 2. a). Underlying Strata-5 – 2 m. Physical and mechanical properties of the array are shown in the table. 2.

Object #2 statistical was taken from the bore pit from the 2,3 m depth. Three soil layers were singled: Strata-1 – vegetation layer (Loamy black soil, loose, solid); Strata-2 – sandy loam, light-brown, silty, in natural state solid in the saturated state fluid, macroporous collapsible; Strata-3 – loam grey heavy silty solid noncollapsible. General sampling of the Object #2 mechanical soil properties was 24 elements. During the geotechnical investigations groundwater level wasn't found. Soil layers thickness were assigned as follow: Strata-1 – 1.1 m, Strata-2 – 6.2 m, Strata-3 – 2 m, total 9 m (fig. 2. b). Physical and mechanical properties of the array are shown in the table. 3.

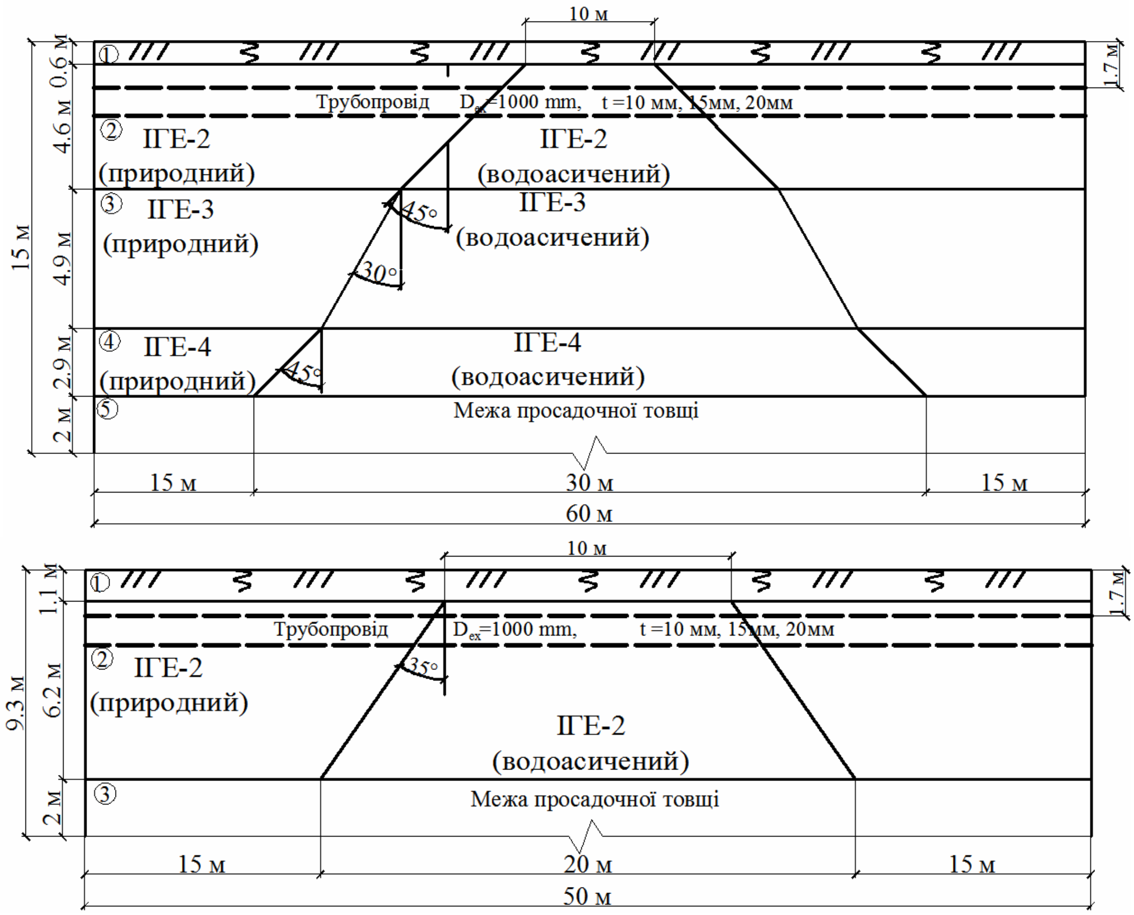


Figure 2 – Design model of the system «MPLP – loessial soil» local soaking: a - soil conditions Object #1; b - Object #1

Table 2 – Physical and mechanical properties of Object #1 loessial basis

Soil properties		Numerical values				
		Strata 1	Strata 2	Strata 3	Strata 4	Strata 5
Strata thickness, h, m		0,6	4,6	4,9	2,9	2,0
Soil density, ρ, kg/m^3		1500	1568	1568	1627	1860
Saturated soil density, ρ_{sat}, kg/m^3		1840	1885	1797	1848	-
Void ratio, *e*		-	0,91	0,94	0,86	-
Relative collapsibility, ε_{sl}, %, for pressure, P, MPa	0,05	-	1,1	1,0	1,0	-
	0,10	-	2,2	2,0	1,9	-
	0,20	-	4,8	4,5	4,3	-
	0,25	-	6,0	5,4	5,1	-
Deformation modulus, E_s, MPa (respective pressure range)	natural conditon	6	12,4	13,2	14	15
	water saturated state		4,7	3,3	2,72	-
Poisson ratio soil, *μ*	natural conditon	0,31	0,33	0,33	0,32	0,31
	water saturated state	-	0,35	0,35	0,34	-

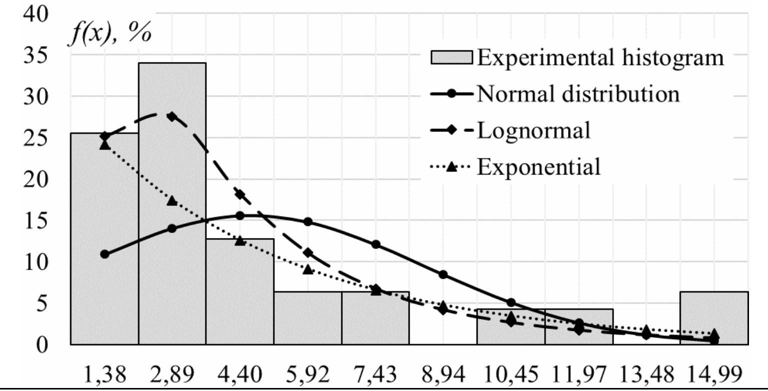
Table 3 - Physical and mechanical properties of Object #2 loessial basis

Soil properties		Numerical values		
		Strata 1	Strata 2	Strata 3
Strata thickness, h, m		1,1	6,2	2,0
Soil density, ρ, kg/m^3		1500	1495	1860
Saturated soil density, ρ_{sat}, kg/m^3		1840	1840	-
Void ratio, *e*		-	1,08	-
Relative collapsibility, ε_{sl}, %, for pressure, P, MPa	0,05	-	1,3	-
	0,10	-	3,0	-
	0,15	-	4,2	-
	0,20	-	5,6	-
Deformation modulus, E_s, MPa	natural conditon	6	12	14
	water saturated state		2,8	
Poisson ratio soil, *μ*	natural conditon	0,31	0,33	0,31
	water saturated state	-	0,35	-

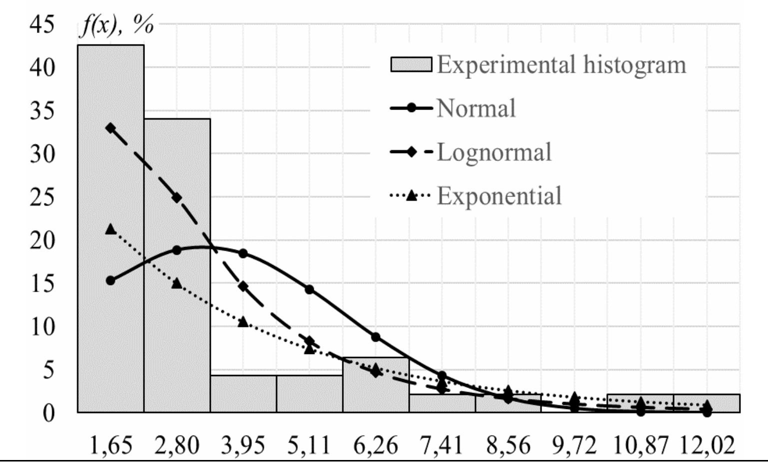
Each soil strata was tested for the uniaxial compression with different pressure intervals for Object #1 (σ = 0,05…0,1; 0,1…0,2; 0,2…0,25 MPa) and for the Object #2 (σ = 0,05…0,1; 0,1…0,15; 0,15…0,2 MPa). It is necessary because soil deformation modulus is variable for each pressure interval [13] and to get most correct results during FEM it is set its value based on actual pressure range (fig. 2 a, b). In the scope of the current paper there only graphs are included used for the following probabilistic simulation (fig. 3).

Figure 3 – Water saturated soil deformation modulus E_{SAT}, MPa

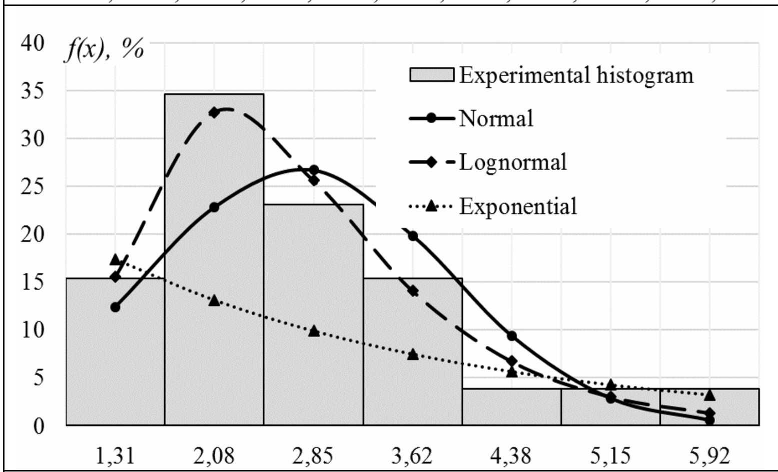
Object #1 Strata-2 for the pressure range 0,05 - 0,1 MPa



Object #1 Strata-3 for the pressure range 0,1 - 0,2 MPa



Object #1 Strata-4 for the pressure range 0,2 - 0,25 MPa



Object #2 Strata-2 for the pressure range 0,1 - 0,2 MPa

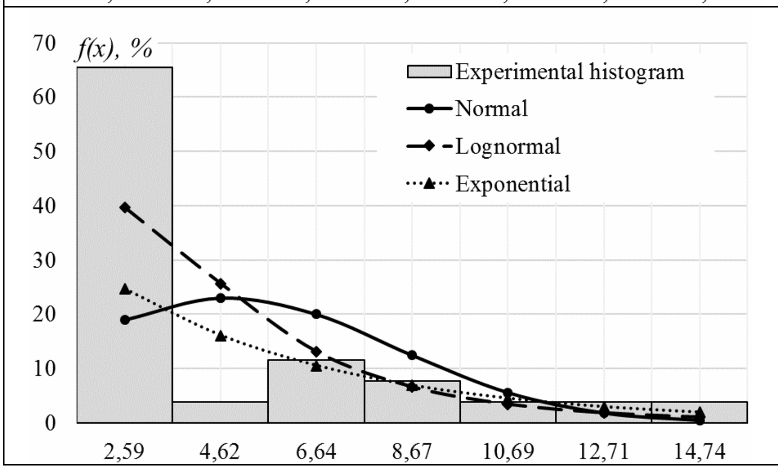


Table 4 – Normal, lognormal, exponential distributions, statistics for water saturated loessial collapsible soil in water saturated state E_{sat}

Stochastic parameters	*M1*	*M2*	*M3*	*M4*	$\overline{X}$	$\widehat{X}$	μ_3	μ_4	$\hat{X}$	ν, %	A_x	E_x	*Distr.*	**Pearson criteria** $\chi^2_{досл.}$ $(\chi^2_{табл.})$
Object #1 Strata-2 – loess loam, brownish-yellow, carbonate, light silty, in natural state solid in the saturated state fluid, macroporous, collapsible														
E_{sat}, MPa, σ = 0,05…0,1 MPa	1,17	7,89	49,29	349,8	4,66 (1,28)	14,93 (0,52)	85,86	349,8	3,9 (0,72)	0,83	1,49	1,18	Norm.	31,4(14,1)
													Log.	7,1(14,1)
													Exp.	11,6(15,5)
Object #1 Strata-3 – sandy loess loam, yellow, carbonate, silty, in natural state solid in the saturated state fluid, macroporous, collapsible														
E_{sat}, MPa, σ = 0,1…0,2 MPa	1,40	6,29	39,58	288,4	3,27 (0,96)	5,75 (0,43)	28,48	228,0	2,40 (0,66)	0,73	2,07	3,89	Norm.	53,2(14,1)
													Log.	9,9(14,1)
													Exp.	25,3(15,5)
Object #1 Strata-4 – loess loam, yellow-brown, brown, carbonate, heavy silty, in natural state solid in the saturated state fluid														
Deformation modulus E_{sat}, MPa, pressure range σ = 0,2…0,25 MPa	0,84	2,92	9,61	39,85	2,72 (0,92)	1,29 (0,16)	1,53	6,34	1,13 (0,4)	0,42	1,03	0,76	Norm.	8,4(14,1)
													Log.	1,8(14,1)
													Exp.	16,5(15,5)
Object #2 Strata-2 – sandy loam, light-brown, silty, in natural state solid in the saturated state fluid, macroporous collapsible;														
Deformation modulus E_{sat}, MPa, pressure range σ = 0,1…0,15 MPa	1,58	5,5	24,8	129,0	2,79 (0,9)	2,19 (0,25)	4,19	19,21	1,48 (0,50)	0,53	1,28	0,97	Norm.	7,7(9,5)
													Log.	1,5(9,5)
													Exp.	9,1(11,1)

Note: M1 – M4 – moments of 1-4 orders; $\overline{X}$ – mathematical expectation; $\widehat{X}$ – dispersion; μ3 –central point of the third order; μ4 – central point of the fourth order; σ – standard; v – variation ratio; A – asymmetry; E – kurtosis.

In brackets values of key statistics for the lognormal distribution are shown.

It was learned experience of the previous soil stochastic researches [13], therefore normal, lognormal and exponential distributions to approximate experimental data are used. Typical histograms and distributions curves for the water saturated loessial collapsible soil in water saturated state E_{sat}, are presented on the fig. 3, respective statistics in the table 4.

Pearson criterion χ^2 were chosen as criterion to estimate current distribution implementation possibility. Statistical analysis of the table 4. show, that lognormal distribution is the most relevant to approximate experimental data of the loessial collapsible soil deformational modulus in the water saturated state E_{sat}. as lognormal distribution is relevant for all mentioned sampling, and respective Pearson criterion value is the smallest comparatively with other distributions table 4.

Above mentioned data confirm previous conclusions which prove that lognormal distribution is the most relevant for soil deformational modulus approximation, but it also essentially new because collapsible soils instead of homogeneous sand embankments are investigated [13].

Probabilistic simulation. Current normative documents in the MPLP design and construction make big focus on the hoop stresses ant operating pressure, in the same time longitudinal stresses from the soil differential settlements are almost ignored [14].

It is reasonable with help of FEM and direct mathematical simulation to compare failure probability of the similar pipeline in the different soil conditions. There is observed pipeline in the homogenous fine sands, and loessial collapsible strata with thickness 13 m Kherson city area (fig. 2 a) – Object #1 and 7 m Poltava region – Object #2 (fig. 2 b). Geometrical parameters of the MPLP, load deterministic and stochastic values for all variants is in the table 5.

For the first variant soil strata without special properties are investigated, therefore it is reasonable to implement direct mathematical simulation. Pipeline as beam on the elastic base fits best. The further step is implementation of the pipeline differential equation stochastic solution in the form of the random functions (RF).

For correct simulation parameters of the correlation function $k_x(\xi)$of the soil conditions variability along the length of the pipeline (1, 2) and respective spectrum density $S_r(\omega)$of this functions should be considered. It should be noted that, those parameters for correct simulations were included in the table 5.

Table 5 – Input parameters to determine pipeline failure probability

Parameter		Designation	Value
External diameter		D_{ex}, m	1,020
Pipeline wall thickness		t, m	0,0096
MO pipeline steel strength		$\overline{\sigma_y}$, kN/cm^2	49
Standard of the pipeline steel strength		$\hat{\sigma}_y$, kN/cm^2	4,9
The length of section		L, m	6036
MO of the soil backfill		$\overline{q}_1$, kN/m	32,1
Pipe and isolation weight		$\sum \overline{q}_{2-4}$, kN/m	3,17
MO of the linear coefficient of elastic foundations		$\overline{c}_{yo}$, kN/m^2	1347
Correlation function parameters of the soil conditions variability along the pipeline length	$k_x(\xi)=e^{-\alpha\xi}$	α, m^{-1}	0,046
	$k_x(\xi)=e^{-\alpha\lvert\xi\rvert} cos\,\beta\xi$	α, m^{-1} θ, m^{-1}	0,023 0,911
Temperature difference MO		$\overline{\Delta t}$, ^{0}C	23,4
Temperature difference standard		$\Delta\hat{t}$, ^{0}C	1,74
Operating pressure MO [12]		$\overline{p}$, kN/cm^2	0,456
Operating pressure standard [12]		$\hat{p}$, kN/cm^2	0,0314
Heterogeneity coefficient		β	1

$$k_x(\xi)=e^{-\alpha\xi} \;=>\; S_r(\omega)=\frac{\alpha\beta_0^2\overline{q}^2}{\pi}\left[\frac{1}{\omega^2+\alpha^2}\right], \tag{2}$$

$$k_x(\xi)=e^{-\alpha|\xi|}\cos\beta\xi \;=>\; S_r(\omega)=\frac{\alpha\beta_0^2\overline{q}^2}{\pi}\left[\frac{1}{(\omega-\theta)^2+\alpha^2}+\frac{1}{(\omega+\theta)^2+\alpha^2}\right], \tag{3}$$

where β_0 – heterogeneity coefficient that characterizes the variation of the total load on the pipeline;

α, θ – correlation function parameters;

ξ – shift between cross sections of the process, 1 m.

The relationship among soil condition heterogeneity RF and pipeline curvature RF [2]

$$S_{\chi}(\omega)=\frac{S_{r}(\omega)\omega^{4}}{(EI\cdot\omega^{4}+\bar{c}_{yo})^{2}}\,,\tag{4}$$

where $\bar{c}_{y0}$ – MO of the linear coefficient of elastic foundations.

Standard of the bending moment $\hat{M}$ RF $\tilde{M}(x)$ calculating with help of pipe bending stiffness the bending EI and pipeline curvature $\bar{\chi}^{2}$

$$\hat{M}=EI\cdot\left[\int_{0}^{\infty}S_{\chi}(\omega)d\omega\right]^{1/2}\,.\tag{5}$$

Pipeline reliability function is follow

$$P(L)=1-Q(L)=P[\sup_{0\le x\le L}\left|\sum\sigma_{позд}\right|<R_{y}]\,,\tag{6}$$

where $\sup\limits_{0\le x\le L}\left|\sum\sigma_{позд}\right|$ – an upper limit of the function $\left|\sum\sigma_{позд}\right|$ at the interval $0\le x\le L$, a $Q(L)$ – pipeline failure probability on the interval L.

Pipeline bearing capacity has normal distribution therefore whole function will have such distribution and failure probability will have form (6) [2, 5]

$$Q(L)=\exp[0{,}5(\gamma_{0}^{2}-\beta^{2})]\,,\tag{7}$$

where γ_{0} – characteristic maximum of RF $\sum\tilde{\sigma}_{позд}(x)$; β – safety characteristic.

According to [4]

$$\gamma_{0}=\sqrt{2\,ln\left[\omega_{\chi}L\Big/\pi\cdot\beta_{\chi}\right]}\,,\tag{8}$$

where $\omega_{\omega\chi}$ – effective frequency of a RF. It characterize MPLP axis curvature and total longitudinal stresses variability; $\beta_{\omega\chi}$ – RF broad badness ratio $\sum\tilde{\sigma}_{позд}(x)$ [4].

$$\omega_{\omega\chi}=\left[\int_{0}^{\infty}S_{\chi}(\omega)\omega^{2}d\omega\Big/\int_{0}^{\infty}S_{\chi}(\omega)d\omega\right]^{1/2}\,,\tag{9}$$

$$\beta_{\omega\chi}=\left[\int_{0}^{\infty}S_{\chi}(\omega)\omega^{4}d\omega\int_{0}^{\infty}S_{\chi}(\omega)d\omega\right]^{1/2}\Big/\int_{0}^{\infty}S_{\chi}(\omega)\omega^{2}d\omega.\tag{10}$$

Longitudinal stresses $\tilde{Y}$ RF MO $\bar{Y}$ and safety characteristic β

$$\bar{Y}=\bar{\sigma}_{y}-\frac{\bar{N}_{\Delta t}-\bar{N}_{p}}{F}\,,\quad \beta=\frac{\bar{\sigma}_{y}-\dfrac{\bar{N}_{\Delta t}-\bar{N}_{p}}{F}}{\sqrt{\hat{\sigma}_{y}^{2}+\dfrac{1}{F^{2}}\hat{N}_{\Delta t}^{2}+\dfrac{1}{F^{2}}\hat{N}_{p}^{2}+\dfrac{1}{W^{2}}\cdot\hat{M}^{2}}}\tag{11}$$

According to [15] the most responsible objects, which failure leads to the biggest consequences class CC3, it must have failure probability lover then $1\cdot10^{-6}$. Corrected values of the correlation functions (1) and (2) allow to obtain following results from table 6, which shows that bending stresses doesn’t exceed 15 MPa and MPLP failure probability is much higher than needed values. These data show that in the homogeneous soils without special properties differential settlements are unimportant.

Table 6 – Modelling results of the MPLP longitudinal stresses failure probability

Parameter	Designation	CF 1	CF 2
Bending moment Standard	$\hat{M}$, kNm	58,6	44,6
Longitudinal (tensile) strength MO	$\overline{N}$, kN	3970	3970
Longitudinal (tensile) strength Standard	$\hat{N}$, kN	929	929
Safety characteristic	β	7,07	7,13
RF effective frequency	$\omega_{\omega\chi}$, m^{-1}	0,21	0,707
RF broad badness ratio	$\beta_{\omega\chi}$	1,708	1,244
Characteristic maximum of RF	γ_0	3,305	2,49
MPLP failure probability	$Q(t)$	**$3{,}3\cdot10^{-9}$**	**$1{,}0\cdot10^{-8}$**

Similar pipeline in the soil conditions fig. 2. a and 2 b is considered. There are collapsible strata 13 m and 7 m respectively. Deterministic characteristics of the soil strata are presented in the table 2 and 3. Stochastic parameters of the soil basis are shown in the table 4. Stochastic parameters of the operating pressure and temperature difference are in the table 5. In general, design scheme can be shown as in the fig. 4. Soil deformations and respective longitudinal stresses for loads MO values are shown on fig. 4 b, c.

With help of Ansys [16] probabilistic modelling in the Monte-Carlo simulation (fig. 5) form distributions and statistics for the soil deformations and pipeline stresses could be obtained. Modeling is performed for MPLP with wall thickness 10 mm for both soil collapsible strata (fig. 6), and for the wall thickness for which pipeline has normative failure probability – $1\cdot10^{-6}$, table 7.

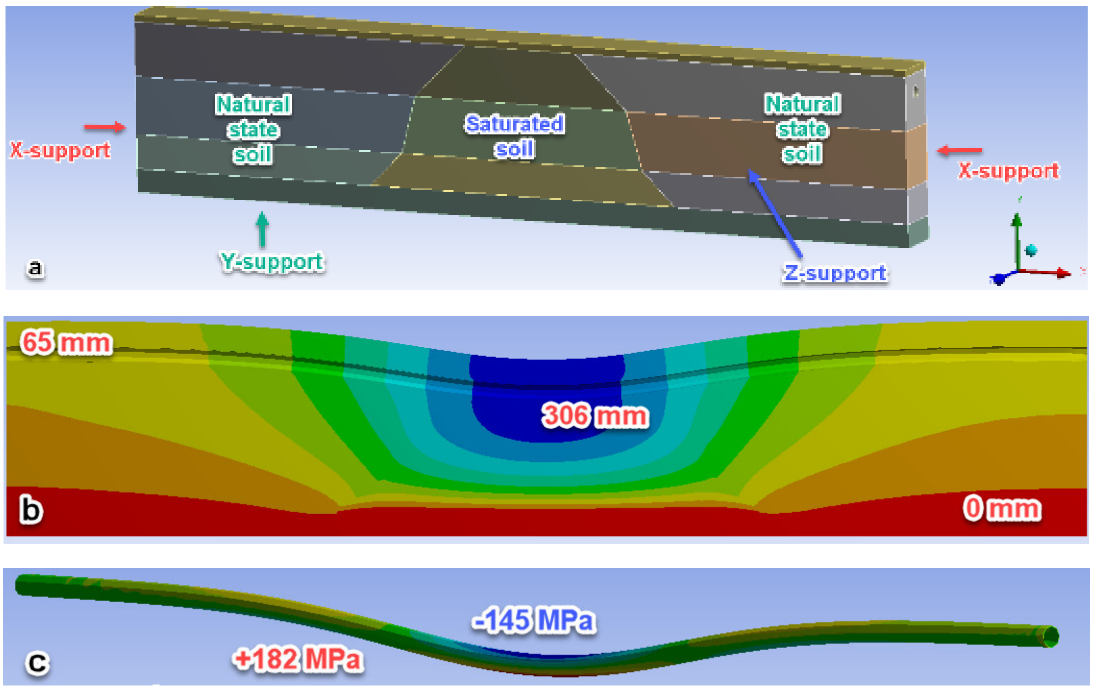


Figure 5 – FEM simulation with loads and influences MO values for the Object #1 soil conditions MPLP wall thickness t = 38 mm

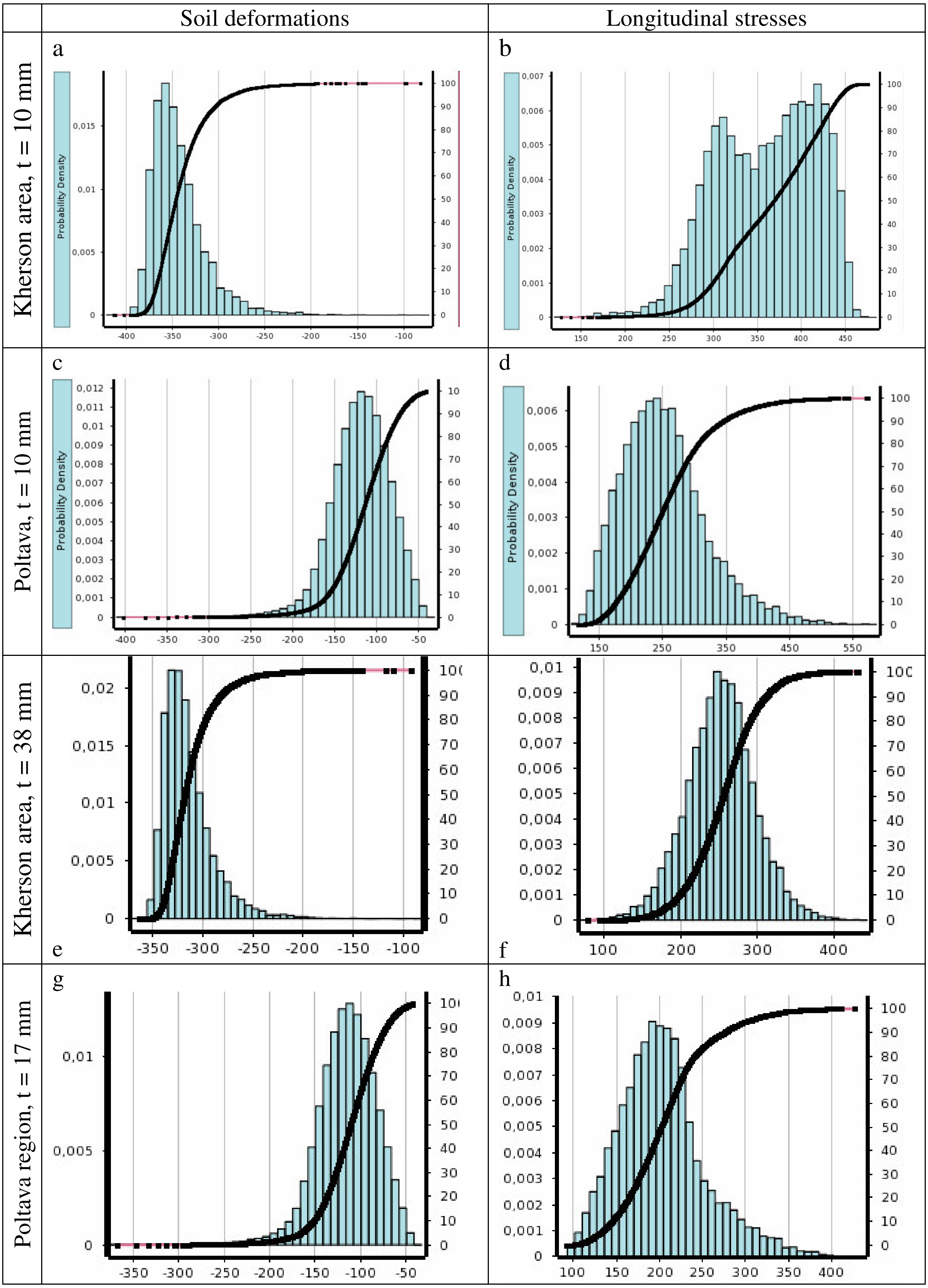


Figure 6 – FEM simulation results for different soil condition and MPLP different wall thickness

Table 7 – MPLP FEM simulation results

Parameter			Designation	Value	Distribution
Kherson Region, **Estimated** wall thickness ***t = 10 mm***	Deformation in the middle of the soil array	MO	$\overline{S}_{mid}$, mm	340	Normal
		Standard	$\hat{S}_{mid}$, mm	29,0	
	Deformation at the bound of the soil array	MO	$\overline{S}_{bound}$, mm	67	Normal
		Standard	$\hat{S}_{bound}$, mm	1,6=>0	
	Pipeline total longitudinal stresses, σ_l	MO	$\overline{\sigma}_y$, MPa	363,9	Normal
		Standard	$\hat{\sigma}_y$, MPa	56,9	
	Safety characteristic		β	2,39	
	MPLP failure probability		$Q(\beta)$	$8{,}2\cdot10^{-3}$	
Poltava Region, **Estimated** wall thickness ***t = 10 mm***	Deformation in the middle of the soil array	MO	$\overline{S}_{mid}$, mm	115,9	Normal
		Standard	$\hat{S}_{mid}$, mm	35,0	
	Deformation at the bound of the soil array	MO	$\overline{S}_{bound}$, mm	39	Normal
		Standard	$\hat{S}_{bound}$, mm	1,1=>0	
	Pipeline total longitudinal stresses, σ_l	MO	$\overline{\sigma}_y$, MPa	258,2	Normal
		Standard	$\hat{\sigma}_y$, MPa	67,1	
	Safety characteristic		β	3,70	
	MPLP failure probability		$Q(\beta)$	$4{,}4\cdot10^{-4}$	
	Designation		Value	Distribution	
Kherson Region, **Estimated** wall thickness ***t = 30 mm***	Deformation in the middle of the soil array	MO	$\overline{S}_{mid}$, mm	311,0	Normal
		Standard	$\hat{S}_{mid}$, mm	24,8	
	Deformation at the bound of the soil array	MO	$\overline{S}_{bound}$, mm	59	Normal
		Standard	$\hat{S}_{bound}$, mm	1,1=>0	
	Pipeline total longitudinal stresses, σ_l	MO	$\overline{\sigma}_y$, MPa	2	Normal
		Standard	$\hat{\sigma}_y$, MPa	24,6	
	Safety characteristic		β	4,73	
	MPLP failure probability		$Q(\beta)$	$1{,}0\cdot10^{-6}$	
Poltava Region, **Estimated** wall thickness ***t = 17 mm***	Deformation in the middle of the soil array	MO	$\overline{S}_{mid}$, mm	110,0	Normal
		Standard	$\hat{S}_{mid}$, mm	33,2	
	Deformation at the bound of the soil array	MO	$\overline{S}_{bound}$, mm	39	Normal
		Standard	$\hat{S}_{bound}$, mm	1,1=>0	
	Pipeline total longitudinal stresses, σ_l	MO	$\overline{\sigma}_y$, MPa	207,7	Normal
		Standard	$\hat{\sigma}_y$, MPa	50,8	
	Safety characteristic		β	4,87	
	MPLP failure probability		$Q(\beta)$	$5{,}5\cdot10^{-7}$	

Conclusions. Lognormal distribution fits best for the probabilistic representation of the loessial collapsible soil deformation modulus in the water saturated state. The smallest values instead normal and exponential laws were confirmed by Pearson criterion. Pipeline with geometrical parameters $1020 \times 9{,}6$ (10) mm, similar internal pressure $\overline{p} = 4{,}56 MPa$, and temperature difference $\overline{\Delta t} = 23{,}4^{o} C$, has follow longitudinal stresses parameter failure probability for the different soil conditions: homogenous fine sand – $3{,}3\cdot10^{-9}$ – $1{,}0\cdot10^{-8}$ (according to the correlation function), soil collapsible strata is about 6,2 m – $4{,}4\cdot10^{-4}$, soil collapsible strata is about 13 m – $8{,}2\cdot10^{-3}$. So, for soil basis without special properties it was obtained very low failure probability, instead for the non-standard (loessial) soil conditions having unacceptable failure probability value. Also high variation ratio for the soil deformations in the array middle and pipeline stresses could be observed, which is explained by very high soil deformation modulus variation ration and lognormal distribution. The latest make impact on the character of the output functions distribution.

Mentioned design model with FEM simulation allows designing pipelines with pre-calculated reliability level. For considered soil conditions normative failure probability reached for the wall thickness 30 and 17 mm for Kherson and Poltava region.

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Received 02.02.2017

UDC 69:001.89;624

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INJECTION PROPAGATION MODEL IN SANDY SOIL

This articles focuses on the process of injection solution in the sand as physical phenomena. During theoretical studies of the propagation of the fluid phase in a porous medium, actual is the definition of the parameters of the jet, necessary for layer material discontinuity in the contact zone of the jet to the surface. It can be achieved through the injection process modeling in a dispersion medium by constructing geometrical similarity of injection jet. It is found that the propagation of the fluid phase in the solid body pores suggests that as they reach the point at the distance from the injector to the «end» of the jet in pores pre-saturated by water, gradual change in injection rate depending on the initial rate of efflux and viscosity occurs..

***Keywords:** soil injection, dispersion medium, physical modeling, solution jet, dispersion medium.*

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МОДЕЛЬ ПОШИРЕННЯ ІН’ЄКЦІЙНОГО РОЗЧИНУ В ПІЩАНИХ ҐРУНТАХ

Розглянуто процеси ін’єкції розчину в пісок з позиції фізичних явищ. Зясовано, що в теоретичних дослідженнях процесу поширення рідкої фази в пористому середовищі актуальним є визначення параметрів струменя, необхідних для порушення цілісності шару матеріалу в зоні контакту струменя з поверхнею. Виявлено, що цього можна досягти за допомогою моделювання процесу ін’єкції в дисперсне середовище шляхом побудови геометричної подоби струменя ін’єкції. Установлено, що розподіл рідкої фази в порах твердого тіла передбачає, що в міру досягнення крайньої точки відстані від ін’єктора до «кінця» струменя в попередньо насичених водою порах відбувається поетапна зміна швидкості розповсюдження ін’єкційного розчину залежно від його початкової швидкості витікання та в’язкості.

***Ключові слова**: ін’єкція ґрунтів, дисперсне середовище, фізичне моделювання, струмінь розчину, дисперсійне середовище.*

Introduction. During the study of the interaction of the dispersion medium with the dispersed phase it is rational to create a model on the level of structural inhomogeneities. It was considered that in a moving jet of dispersion medium, which is a multi-phase non-Newtonian fluid, soil particles can not accumulate or transfer mechanical energy from the rotor flux [1].

Analysis of recent research and publications of sources. In hydro or aeromechanical interaction the kinetic energy of the selected soil particles can be transformed into potential, although the effectiveness of such an accumulation of potential energy is much smaller than in the case of motion of Newtonian fluids in disperse systems [2 – 4]. For a qualitative description of such processes one of the effective ways at an early stage of their research is their modeling [5 – 6]. Physical modeling of injection into the soil it is the fundamental definition of their parameters under the model characteristics found in its study [7 – 8]. A feature of the physical modeling is that the characterizing does not require mathematical description of the processes, but an idea about the mechanism (physical nature) of the phenomena, in order to properly calculate the parameters of the main subject according to tests of its model. Since the physical modeling of the physical nature of the phenomena that occur in natural product and the model is the same, according to the results of experiments on the models, the nature and effects of the quantitative relationship between the values for field conditions can be evaluated [9].

Isolation of previously unsolved aspects of the problem. The task of theoretical studies of the fluid phase propagation in the porous medium includes determining of a minimum dynamic pressure of the jet necessary to discontinuities in the material layer of the jet in the contact zone with the surface.

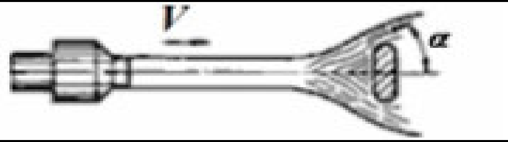
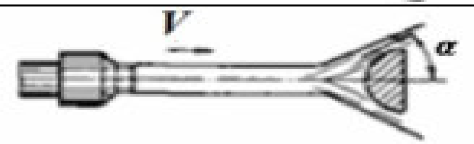
Formulation of the problem. To determine the fluid jet parameters required for dynamic fracture of the layer of porous material, it is necessary to study the interaction of the jet with the surface of the solid phase particles. In order to assess this interaction it is necessary to know the characteristics of spreading of free axially symmetric jets.

Main material and results. The model of the propagation of the jet injection in the soil was taken as an analysis object. Under the injection jet it is considered the jet that goes from the injector into the thickness of the material, with a shape that is characterized by a length L and propagation diameter D_0. The model of conical jet allows to analyze the mechanisms of its propagation depending on the pressure. The form of propagating jet in the form of a cone assumes that, depending on the width of its propagation, in its volume, the fluid phase is in the free state, in the form of polyabsorption layers and a monoabsorption layer in close proximity to the apex of the cone. At the fluid discharge from the injector in heterogeneous environment, by virtue of obstacles (Table 1) [10], there is a pressure on certain areas. Based on this assumption, we conditionally allocated four zones that characterize the gradual spread of the fluid phase to a solid medium at $\rho = \text{const}$, Fig.1.

Depending on the accepted model of fluid phase injection into the solid, one or another mechanism of interaction is proposed. When presentation of soil as a capillary-porous systems the main focuse is made on processes and phenomena that occur in the pores and cavities of various sizes in the propagation of fluid in them.

The main factors influencing the spread of fluid in the system include: discharge pressure effect on the modeling material particles; a hydraulic pressure of the fluid and its migration to the weak places; capillary effects associated with changes in pressure, depending on the concentration of the fluid; osmotic phenomena associated with the occurrence of concentration gradients of pore fluid; crystallization pressure that occurs during chemical reactions of hydration.

Table 1 – Examples of jet interaction with the stationary surfaces [10]

Interacton scheme	Pressure by interaction
	$P = \frac{\gamma}{g} QV$
	$P = \frac{\gamma}{g} QV(1 - cos\, a)$
	$P = \frac{\gamma}{g} QV(1 - cos\, a)$

where V – exhaust velocity of jet compressed section; Q – flow of fluid; a – angle of jet deflection; γ – speed factor.

In the event of adoption of a fictitious soil model as a two-component system, consisting of a matrix in which the distributed switching (sand), the main attention is devoted to the differences in the absolute values of grain parting coefficients of matrix and impurities of the material.

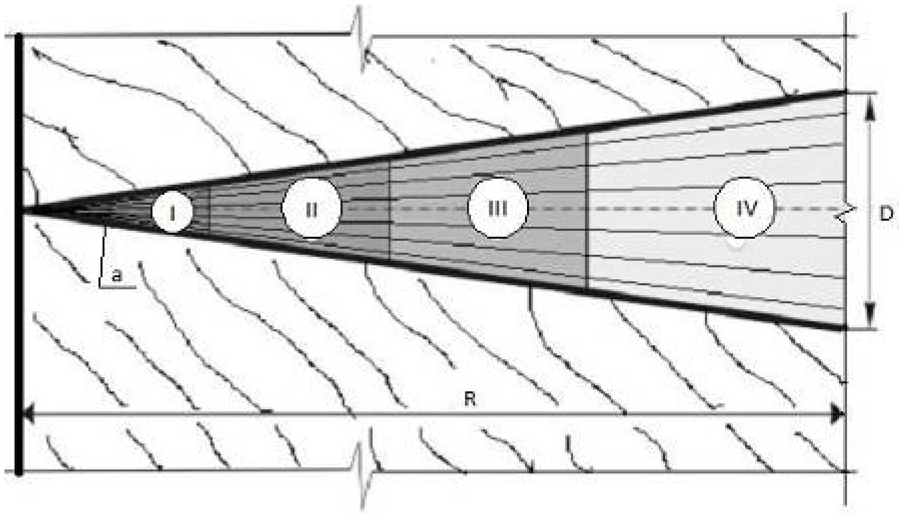


Figure 1 – Model of conical jet into a dispersion medium in the context:
R – the length of the jet; (D) – the diameter of the jet propagation; a – propagation angle;
I, II, III, IV – zone, characterizing gradual propagation of the fluid phase in a solid medium

Analysis of the structure of moistened (system moisture is about 5-6% by weight) quartz sand (as a false model) sandy soil showed capillary interaction of small particles of sand large. «Adherent» particles form aggregates-globules, which are in turn building the «arched» structure. This penetration of the fluid under pressure in its pore spaces may lead to a parting of the grains and the violation of their capillary interaction.

Due to the fact that the adhesion strength between particles varies due to the difference in their size, it is logical to assume that the first fluid jet will increasingly lead to a breakdown in communication and greater parting directly at the head of the injector, i.e. where the speed of the outflow is maximum. It may be due to the fact that the adhesion strength of the particles is much lower than jet response force.

For structures composed of discrete units, interacting through the internal interface, the diffusion mass transfer coefficients may vary by orders of magnitude. Therefore, in such cases, it is appropriate to speak not about the local («crevice») mechanism of mass transfer, but the front as wide enough area formed under the effect of gradients jet velocity. Such assumptions allow to extend the idea of distributing a fluid phase jet at different pressures in the discharge of its bulk of the solid. Due to the fact that first of all at samples injecting into thickness the solid particles come into operation, producing surface effects, when selecting the model we will assume that the jet extends deep into the compressed material in the form of a cone. Model of false soil with a single jet in the form of a cone, the base of which is ellipse, shown in Fig. 2a.

In general terms cone surface of the second order is based on ellipse; in a suitable Cartesian system of reference *(x-axis* and *y-axes* is parallel to ellipse axis, the top of the cone coincides with the origin, center of the ellipse lies on the axis *Oz)* its volume equation has the form:

$$V_{cm} = \frac{1}{3}\pi Rr \cdot H = \frac{1}{3}\pi Ox \cdot Oy \cdot Oz = \frac{1}{3}\pi O_1A \cdot O_1C \cdot OO_1 \ .$$

In the most general case, when the cone is supported by an arbitrary flat surface, it can be shown that the equation of the lateral surface of a cone (with the vertex at the origin) is given by the equation, where the function is homogeneous, i.e. satisfying the conditionfor any real number α.

a b

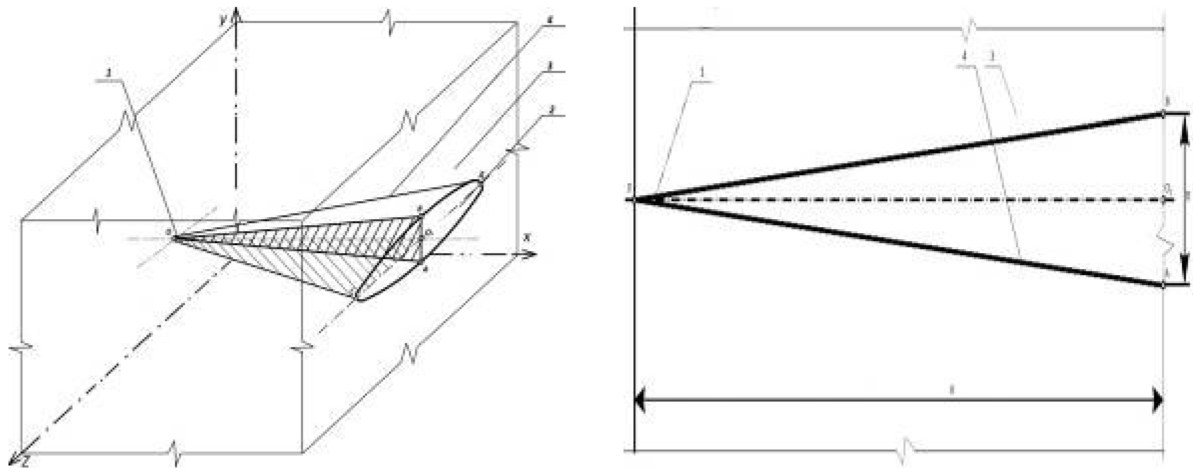


Figure 2 – Model of injection jet propagation in the false soil:
a – cone-shaped jet model; b – jet model fragment in cross section
1 – (pole) the mouth of the jet; 2 – jet dissipation area;
3 – model ground; 4 – external jet bound

As adopted in the terminology and characteristics of the jet in the hydrodynamics it includes present exterior free jet boundaries, which are characterized by their surface area; jet front, defined by its length or length; jet pole, estimated by radius of the pole. At the first stage we will analyze planar (two-dimensional) model of a jet, which is a sectional view of a bulk jet passing through its axis of symmetry (AOB, fig. 2.b). Thus, as the jet model dimensional wedge jet is adopted, which is characterized by the spread diameter (D), a length (R), exterior boundaries (OA, OB) and the radius of the pole, Fig. 2. b.

If assume that the diameter of the jet propagation (D) is influenced by the initial velocity of its outflow. The wedge shape of the jet suggests that along its length the speed of propagation varies. Propagation of the fluid in a solid body over time may not match the rate of movement depending on the structure of the matrix material. It is due to the fact that with increasing of particle size of the solid material as the distance from the injector in the volume of the material the propagation velocity decreases. Moreover, it is interesting to note that such jet propagation is observed in the air as well, Fig. 3 [1]. The adopted jet model and the jet propagation speed change scheme depending on the distance from the pole is shown on Fig. 4.

In general, the following areas for the propagation velocity of the jet can be identified

– OE area, where in the jet has a velocity substantially equal to the nozzle outlet;

– EG area, wherein the jet slows down the speed of propagation due to the frictional forces on the surface of the particles;

– GO_1 portion$_1$ wherein the jet practically lost initial kinetic energy, water is in monoabsorption state.

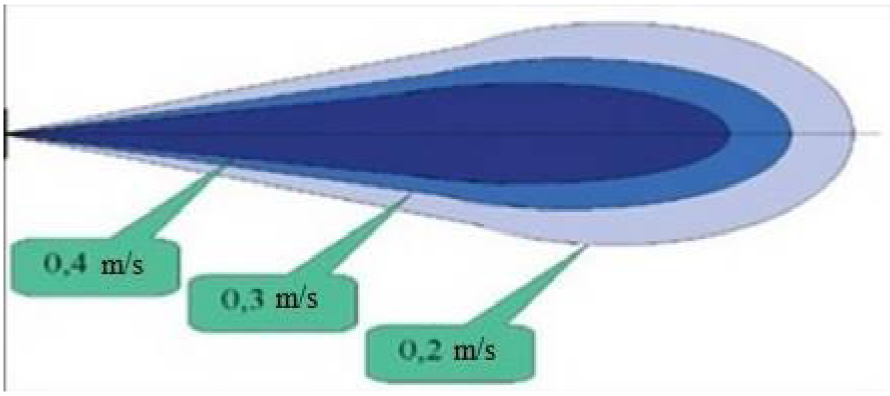


Figure 3 – Model of jet propagation speed in a homogeneous medium

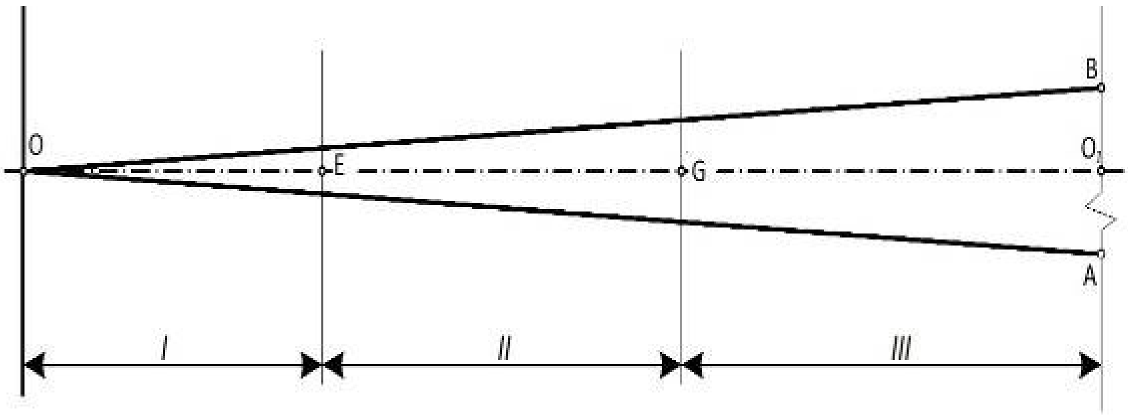


Figure 4 – Propagation of the propagation velocity in the material on the jet length in the jet model:

I – area of propagation without changing the speed of initial outflow of the jet;
II – area of of the jet propagation with the speed slowdown due to the friction forces occurring on the surface of the particles to a greater extent;
III – area of the jet propagation in which it significantly slows its speed, expanding in diameter at the same time

Propagation of the fluid phase in the pores of a solid body suggests that as they reach the point at the distance from the injector to the «end» of the jet in pores pre- saturated by water a gradual change in injection rate occurs depending on the expiration of the initial rate and viscosity.

Conclusions. The research which were carried out allows us to take for analysis the following jet propagation models in the model soil:

1. Model jet propagation in sand, arranged on a principle «fluid in the solid body» includes jet injections at different levels of structural inhomogeneities due to its

propagation. The analysis allowed simulate jet which, depending on the distance from the injector changes the propagation form and speed. Such jets includes jet, the propagation of which took place at a pressure = const.

2. As a jet model a two-dimensional wedge-shaped jet with fixed parameters is adopted. The wedge shape of the jet is taken on the assumption that along its length, due to changes in the distance from the injector, its speed reduces due to increasing friction with the soil particles. Changes of communication between the soil particles occur between the opposite shores. It involves changing of the radius of the propagation by length of the jet.

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Received 29.11.2016

UDC 624.131+624.15

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COMPARATIVE ANALYSIS OF DESIGN SETTLEMENT FOUNDATIONS METHODS ACCORDING TO DATA OF CONE PENETRATION TEST ON NATIONAL AND EUROPEAN STANDARDS

It is given a comparative analysis of the methods of computation of foundations settlement according to Cone Penetration Test with the current normative documents of the Republic of Belarus and the Eurocode 7 «Geotechnical design» (part 1, 2). Three methods of computation of foundation settlement for the sediment limit state SLS are considered in accordance to European standards and two methods of computation of foundation settlement are considered in accordance to National standards. The similarities and the differences in construction of the conditional pile foundation are identified according to National and European standards. The similarities and the differences in the methods of foundation settlement computation in accordance to European and National standards are revealed and summarized. The difference of the calculated values of foundation settlement is defined in percentage.

***Keywords:** Cone Penetration Test, foundation settlement, elastic modulus, deformation modulus, equivalent raft foundation, nominal foundation.*

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СРАВНИТЕЛЬНЫЙ АНАЛИЗ МЕТОДОВ РАСЧЕТА ОСАДКИ ФУНДАМЕНТА ПО ДАННЫМ СТАТИЧЕСКОГО ЗОНДИРОВАНИЯ ПО НАЦИОНАЛЬНЫМ И ЕВРОПЕЙСКИМ НОРМАМ

Приведен сравнительный анализ методов расчета осадки фундаментов по данным статического зондирования согласно действующим нормативным документам Республики Беларусь и EUROCODE 7 «Geotechnical design» (part 1, 2). Рассмотрены согласно европейским нормам три метода расчета осадок фундамента для предельного состояния SLS и два метода расчета осадок по национальным нормам. Определены сходства и различия в построении условного свайного фундамента по национальным и европейским нормам. Выявлены и обобщены сходства и различия методов расчета осадок фундамента по европейским и национальным нормам. Определена разница полученных расчетных значений осадок фундамента в процентом отношении.

***Ключевые слова:** статическое зондирование, осадка фундамента, модуль упругости, модуль деформации, эквивалентный плитный фундамент, условный фундамент.*

Introduction. Currently in the Republic of Belarus it is a process of mass introduction of European materials, technologies and equipment. It became necessary to deal with the process of harmonization of National regulatory documents with Eurocodes in order to be able to enjoy these achievements. From July 1st, 2015 the EU design standards (Technical Code of Practice EN) became mandatory when design of all building structures, including bases and foundations.

The comparative analysis of the methods of foundations computation settlement according to European standards and standards of the Republic of Belarus will have undoubted interest for the experts in the field of geotechnical engineering.

Eurocode 7 [4, 5] consists of two parts that cover specific technical aspects. National Annexes to Eurocodes stipulate additional requirements to particular parameters of construction that can be higher but not lower than European requirements. Every country determines such requirements independently and until now there is no developed unified approach in geotechnical engineering.

National regulatory documents [1 – 3] as well as Eurocode 7 [4, 5] prescribe design of various objects according to two groups of limit states (according to Ultimate limit state design and Serviceability limit state design) and they have the uniform terminology and designations, so in general there are no key distinctions between Nation standards of the Republic of Belarus and European tendencies.

To evaluate the limit states of the second group in [1, 2, 3] it is clearly indicated what kind of calculations should be done:

– by deformation of the base buildings due to the external loads and the own weight of soil;
– by training and crack opening in the construction of foundations.

The fields of application of the methods of computation of foundations settlement are clearly stipulated in [1, 3], as well as the calculation formulas are adjusted, the rules on definition of deformation modulus are introduced. These methods combine the techniques of evaluation of joint work of base and superficial structure, where the rigidity of upper foundation structures is taken into account approximately by means of the correction coefficients that depend on type of a structure according to the rigidity. Such methods are developed in standards concerning the design of bases and structures and they are the most frequently used in practice because of their simplicity. The methods of design settlement, given in [1, 3] are the following: the method of layer by layer summing up, the method of equivalent layer, the method of linearly deformable layer of the finite thickness.

And in [4, 5] it is said about the design criterions you should follow when the computations and the mandatory procedure of verification using the partial factors is established as well. The value partial factors are advisory in nature; they can be modified in National Annex. The measures for prevention of the beginning of the limit state, as it is stated in [4], are largely focused on consideration of stabilized and instabilized states. There are two key conditions in [4] that should be studied: the ultimate limit state (ULS) at failure and the serviceability limit state (SLS) when operating loading where the deformation must not exceed the maximum permissible. Even National Annexes to determine ULS strongly differ for these simple cases in spite of the fact they are based on similar principles of destruction, as well as the computations based on these principles also differ. When checking the soil base according to the serviceability limit state the partial factors are set equal to 1.

In Eurocode 7 [4, 5] there is no unified approach to determine the foundation settlement. General requirements and recommendations to determine the limit states and the limit values of displacement foundation are presented. Annex F describes the simple analytical computational methods of settlement [4]. There are four methods of settlement computation in Eurocode part 2 [5] (although they are not presented in Eurocode 7 part 1 [4]), based on the results of field test using the semi-empirical computation models

(Annexes B2, C2 and D4 [5]). Use of this particular method is usually stipulated in National Annex to Eurocode.

When calculation of the settlement pile groups both in European and National standards they use the principle of linear deformation. The foundation calculation using the piles built-in ground coat and its bases according to the deformations should be performed, as a rule, in terms of the nominal raft foundation in compliance with the requirements [1, 2, 3]. The definition of the limits of nominal foundation is performed in [2] and it is well-known to national geotechnical engineers. In accordance with [4,12] the basic methods of calculation of the pile foundations are based on assumption that the group of piles behaves as a fundamental unit with a certain degree of flexibility that depends on the rigidity of pile cap's connection to piles. Taking into consideration this fact it is possible to use the well-known principles of soil engineering to determine the bearing resistance and the settlement of piled foundation. In the present case the equivalent foundation slab is used. The piles arrangement in pile cap is almost identical according to European and National standards. The distance between the centers of driven pile shouldn't be less than three diameters of pile.

Analysis of recent achievements and publications. The examples of calculation of settlement of shallow foundation are given in lectures [6, 7 ,8]. The example of computation of settlement of shallow foundation taking into account the soil consolidation on multiarrayed base is given in study [10]. The example of calculation of settlement of pile foundation is presented in study [13]. The authors [9, 11] explain and comment the articles of Eurocode 7 that contain new approaches to designing; they give the examples of computation of settlement foundations according to European standards. The author [12] gives the examples of calculation of foundation settlement according to data of Cone Penetration Test.

Accentuation of precedently unsolved aspects of general problem. Despite the increased interest of well-known scientists to chosen range of problems, the "harmonization" of foundation settlement design according to data of Cone Penetration Test following National and European standards does not lose relevance. These issues stay unsolved in full and this requires their further development and elaboration.

National standards of the Republic of Belarus and Eurocode 7 have a number of similar provisions concerning the limit state design. However, the designing results are different. In our opinion the direct use of European standards without taking into account national peculiarities of design and foundation computation in the Republic of Belarus is impossible.

Purpose of study – comparison of the results of foundation settlement calculation according to data of Cone Penetration Test following National and European standards of designing.

Main part. There are more than 20 methods of foundation settlement calculation. As the limit calculation value for effect (settlement) – S_d in European standards [4,5] they accept the sum of three constituents of settlement:

$S_d = S_e + S_c + S_s$.

where S_e –is the immediate settlement, that appears immediately after construction.

In European geotechnical engineers' opinion the monetary settlement S_e is dominant for gravel and medium sand. There are several methods of specification and definition of such settlement:

– solving problems of the theory of elasticity (Annex F [4]),

– formula of Janbu et al (1956) with amendments of Christian and Carrier (1978) [13] – is used to determine settlement when undrained condition,

– Schmertmann's method – is used when calculation of settlement in sand (Annex D.3 [5]) or Schmertmann's formula [12, c. 68]).

– S_c – is the settlement caused by consolidation that is dominant for clay soil.

The consolidation settlement S_c is usually calculated using precondition of dimensional compression. It is admissible to determine the deformation parameter of ground coat according to the empirical dependences of Terzaghi's one dimensional consolidation theory [9 – 12], taking into account the average degree of consolidation U_m the corresponding time factor T_v.

– S_s – is the settlement caused by creep (secondary settlement), that is dominant for ground coat with significant rheological properties.

Secondary settlement continues for a long time.

Stage 1. Design settlement shallow foundation.

Let's consider the simplest case of mutual interaction of shallow foundation with one layer supporting ground. We'll restrict ourselves by problem of determining ultimate stabilized settlement foundation due to the load action that is transmitted to ground coat through the bottom of foundation.

To calculate foundation settlement the results of Cone Penetration Test on the territory of Vitebsk region of the Republic of Belarus were taken into consideration.

Ground coat – is medium sand, with medium strength. The design value of ground coat specifications:

$q_s = 6{,}53$ MPa, $E = 26{,}85$ MPa, $\gamma_{II} = 18{,}8$ kN/m^3, $c_{II} = 0{,}001$ MPa, $\varphi_{II} = 35{,}5^{\circ}$.

Shallow foundation, foundation depth is of 1,5 m, correlation of length and width of foundation is equal to 1.

Vertical load on foundation $N = 1500$kN (set conditionally).

1.1. Calculation of shallow foundations settlement according to National standards of the Republic of Belarus.

The final foundation settlement S_c with the use of design scheme in terms of linearly-deformable halfspace with conditional constraint of compressible thickness layer is determined by the method of layer by layer summing up using the following equation:

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

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

$\sigma_{zp,i}$ – is the average value of vertical stress beneath load in i^{-th} elementary layer of soil equal to semi-sum of subjecting to stress on upper and lower limits of i-th elementary layer, kPa;

h_i and E_i – are consequently depth and deformation modulus of i-th elementary layer of ground coat. The depth of h_i layer should not exceed 0,4 of foundation width;

n – is a number of layers into which compressible soil column is divided.

The computation is performed for the varying bottom width of a foundation. The calculation results are recorded in Table 1.

Table 1 – Calculation results of foundation settlement according to formula (1)

Dimensions of foundation base	1×1	2×2	3×3	4×4
Value of settlement according to method of layer by layer summing up, m	0,045	0,019	0,01	0,006

The final settlement impaction (maximal ultimate value – for slender foundations and average – for rigid foundations) using the equivalent layer method is determined with the use of the theory of filtration consolidation assuming that base of foundation is a linearly-deformable body, according to the formula:

$$S = h_э \cdot m_v \cdot p_0 , \quad (2)$$

where m_v – is the coefficient of volume change of ground coat of homogeneous foundation,

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

$h_э$ – is the power of equivalent layer, m, is determined using the formula:

$$h_э = A_\omega \cdot b , \quad (3)$$

where A_ω – is the coefficient of equivalent layer taken according to the type of ground coat and form of foundation base, (Table 5.14 [3]);

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

In the study we find out the coefficient of volume change of ground coat of homogeneous foundation using the following formula:

$$m_v = \frac{\beta}{E} , \quad (4)$$

where E – is the total soil deformation modulus,

$$\beta = \frac{1 - 2\nu^2}{1 - \nu} , \quad (5)$$

ν – is the coefficient of lateral dilatation of ground coat accepted according to Table 5.14 [3].

We make the computation for various bottom widths of a foundation. The calculation results are recorded in Table 2.

Table 2 – Calculation results of foundation settlement according to formula (2)

Dimensions of foundation base	1×1	2×2	3×3	4×4
Value of settlement according to method of equivalent layer, m	0,063	0,029	0,018	0,011

1.2. Calculation of shallow foundations settlement according to European standards.

As it was stated above, the foundation settlements according to European standards can be determined using the different formulas. In this case the characteristics of ground coat are obtained with the help of Cone Penetration Test and it is said in [5] that «it should be used the semiempirical methods of computation as well as the mathematical method of design».

If the adapted method of recoverability specification (with account of necessary improvements) is used for the computation of shallow foundation settlements according to the results of Cone Penetration Test, then by the load resistance it is possible to determine the drained (long-term) Young's of elasticity E_m (Annex F [4]). There is some uncertainty when determination of value of E_m according to data of Cone Penetration Test. Eurocode 7 Annex D [5] offers to determine with the help of two methods: using tabular data and using the coefficient $\alpha = 2…4$ [12].

For the computation we use the calculation formula of elastic settlement Annex F [4].

$$S = \frac{P \cdot b \cdot f}{E_m} , \quad (6)$$

where P – is the bearing pressure (linearly distributed) on the base of the foundation;

E_m – is the design value of the modulus of elasticity;

b – is the foundation's width;

f – is a settlement coefficient:

$$f = (1-\nu^2)\cdot I \quad , \tag{7}$$

where ν – is the Poisson's ratio for sands;

I = 1,12 (ratio of length and foundation width equal to 1 and determination of extreme draft under the center of the foundation).

$E_{m1} = \alpha q_c$ = 4 × 6,53 MPa = 26,12 MPa.

Young's modulus when Cone Penetration Test can be also determined using the formula [12, c. 70]:

$$E_{m2} = \alpha_E \cdot (q_t - \sigma_{v0}), \tag{8}$$

$$\alpha_E = 0.015 \cdot [10 \cdot (0.55 \cdot I_c + 1.68)] \tag{9}$$

where I_c – is a coefficient acceptable according to the Table in [12, p. 27],

$q_t = q_c$ for sands,

σ_{v0} – is a total vertical pressure at the considered depth.

E_{m1} = 16,56 MPa.

As it was already mentioned above, the settlement calculated by the formula (6), is dominant for river and medium gravels. Therefore, in this case we can write the following $S_d = S_e$.

We make the computation for various bottom widths of a foundation. The calculation results are recorded in Table 3.

Table 3 – Calculation results of foundation settlement according to formula (6)

Dimensions of foundation base	1×1	2×2	3×3	4×4
Value according to adjusted elasticity method, m with E_{m1}	0,059	0,028	0,017	0,010
Value of settlement according to adjusted elasticity method, m with E_{m2} of equivalent layer , m	0,093	0,044	0,026	0,017

The second method of determination of foundation settlement according to data of Cone Penetration Test is the calculation using the Schmertmann's method [5, 12]. Foundation settlement s due to load q is determined using the following formula:

$$s = C_1 \cdot C_2 \cdot (q - \sigma'_{v0}) \cdot \int_0^{z1} \frac{I_z}{C_3 \cdot E'} dz, \tag{10}$$

where $C_1 = 1 - 0,5 \cdot [\sigma'_{v0} / (q - \sigma'_{v0})]$,

$C_2 = 1.2 + 0.2 \lg t$,

C_3 – is the correction for shape of footing (for square footings C_3 =1,25),

σ'_{v0} – is a primary effective vertical stress at the level of the footing,

t – is the time, year,

I_z – is an influencing factor of subjecting to stress,

E' – is Young's of elasticity, in this case E' = 2,5 q_c.

The calculation of settlement using the formula (10) we will mention as an instance for the foundation with dimension 1×1 m when the ground conditions mentioned above.

We will replace the sample profile of Cone Penetration Test by the diagram (Figure 1), where the vertical lines combine the areas having the average CPT resistance for a sublayer q_c.

$$\Delta q = q - \sigma'_{v0} = \frac{1500}{1\times1} - 18{,}8\times1{,}5 = 1471{,}8 \text{ kPa},$$

$$\sigma'_{vp} = 18{,}8\times1{,}5 + 18{,}5\frac{1}{2} = 37{,}6 \text{ kPa},$$

$$I_z = 0{,}5 + 0{,}1\sqrt{\frac{\Delta q}{\sigma'_{vp}}} = 0{,}5 + 0{,}1\sqrt{\frac{1474{,}8}{37{,}6}} = 1{,}13 .$$

Knowing the value I_z we construct a graph of distribution of the impact factor of vertical intensity for square foundation having the following dimension 1×1 m (Figure 1).

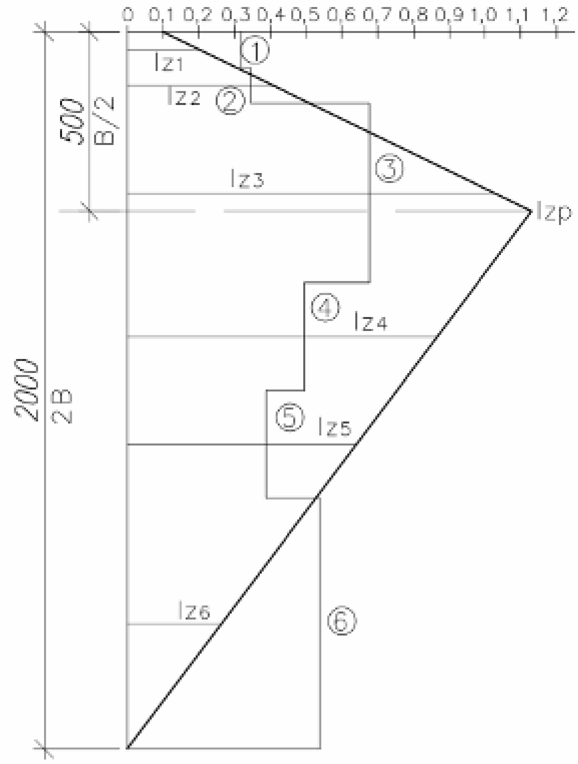

Figure 1 – Graph to determine impact factor of subjecting to stress

The calculated data are summarized in Table 4.

Table 4 – Calculated data to determine settlement using the Schmertmann's method

Number of layer	Resistance for sublayer q_c, MPa	Impact factor I_{zi}	Young's of elasticity in i-th layere E', MPa	Layer thickness, m
1	3,17	0,2	7,9	0,1
2	3,45	0,41	8,62	0,1
3	6,77	1,03	16,9	0,5
4	4,95	0,87	12,37	0,3
5	3,87	0,64	9,67	0,3
6	5,39	0,26	13,47	0,7

where $C_1 = 1 - 0{,}5 \cdot [28{,}2 / 1471{,}8] = 0{,}99$, $C_2 = 1$ for the end of construction.

$s_1 = 0{,}99 \cdot 1 \cdot 1471{,}8 \cdot \dfrac{0{,}0852}{1{,}25} = 0{,}099$ m.

The further computations are made for various bottom widths of a foundation. The calculated data are summarized in Table 5.

Table 5 – Calculation results of foundation settlement according to formula (10)

Dimensions of foundation base	1×1	2×2	3×3	4×4
Value of settlement according to the Schmertmann's method, m	0,099	0,033	0,016	0,007

Stage 2. Settlement computation of piled foundations.

The piled foundation composed of 4 displacement piles C8-40. The pile's interfacing with the foundation piling is non-yielding. The depth of pile setting in the foundation piling is of 0,5 m. The laying depth of the foundation piling is of 1,5 m.

The vertical load on the piled foundation N = 3000 kN (given conditionally).

For this calculation we assume that the settlement is homogeneous. The computation is carried out on the assumption of that there is only one soil layer in the subfoundation - medium coarse sand, with average strength.

2.1. Settlement computation of piled foundation according to National standards.

The foundation computation of the piles built-in ground coat and its primary structure according to the deformations should be performed, as a rule, in terms of the conventional foundation slab according to the requirements [3]. The determination of limits of the conventional foundation is given in [2] and it is well-known to national geotechnical engineers.

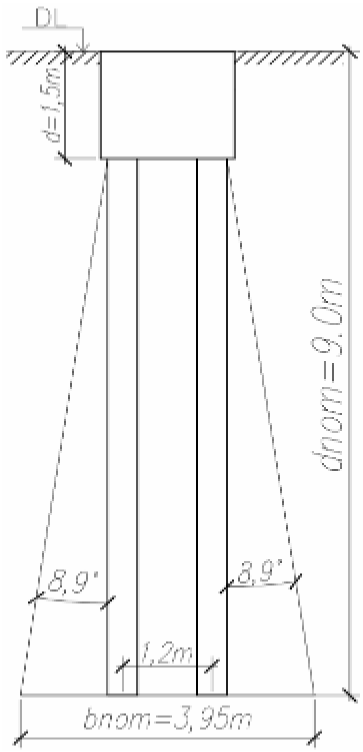

Figure 2 – Graphical conduit of conventional foundation according to National standards

We will determine the settlement of the conventional foundation using the formulas (1 – 2), mentioned above. The calculation results are recorded in Table 6.

Table 6 – Calculation results of settlement of conventional foundation

Method of computation of settlement according to National standards	Value of settlement, m
Method of layer by layer summing up	0,021
Method of equivalent layer	0,037

2.2. Settlement computation of piled foundation according to European standards.

There is practically no information concerning the determination of settlement of the piled foundations in Eurocode 7 [4, 5].

The first step in the settlement analysis is to determine the vertical stress distribution below the base of the equivalent raft or block foundation, built under the principle 1:4 (Figure 3), while using the graph [13, p 254]. An approximate method that is often used in the calculations lies in the supposition that the effective level of transmission is at the depth of 2 D/3 below the top of the pile. European geotechnical engineers consider the application of the method of equivalent foundation slab to calculate the settlement of the piled foundation sufficiently reliable. This method is widely used to determine the preliminary settlements or to check the settlements obtained by computer calculation. When checking the limit state of SLS piles group [4] it is recommended to use the partial coefficient 1,0 for actions and properties of the ground coats unless otherwise specified.

The second step is to determine the limit of the compressible thickness where the vertical stress resulting from the net pressure at the foundation level amounts only to 20% of the initial intergranular stress caused by the weight of the ground coat.

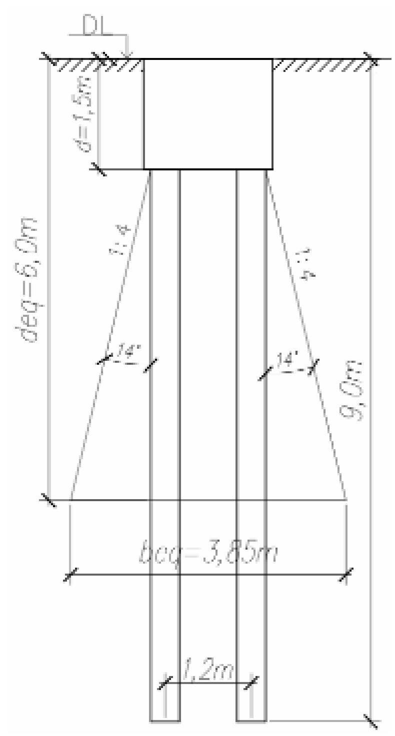

Figure 3 – Graphical construction of equivalent raft foundation according to European standards

The third step is to calculate the settlement of the foundation. As it is shown in Annex F [4], it is possible to perform the settlement computation using the «stress-strain method». The second method of the settlement calculation is the adjusted elasticity method. The graphical representation of this method that is adapted to the equivalent conditional foundation is shown in Figure 4.

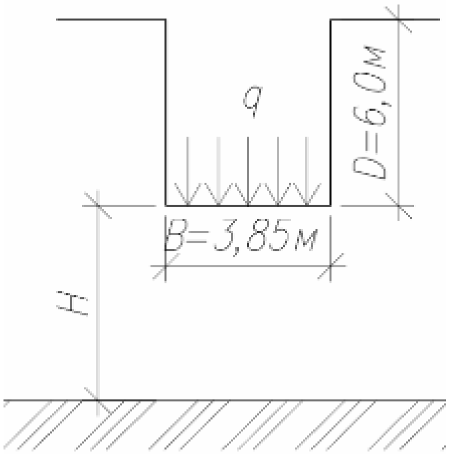

Figure 4 – Equivalent foundation slab for settlement computation

The average settlement of the piled foundation can be determined using the following formula:

$$s_e = \frac{\mu_1 \cdot \mu_2 \cdot q_n \cdot B}{E_u}, \qquad (11)$$

where μ_1, μ_2 – are the influence factors, that are determined using the graphs of Christian and Carrier. These coefficients can be determined by the graphs [13, p 260].

The coefficient μ_0 depends on the ratio $D/B = 6/3.85 = 1.56$, correspondingly $\mu_0 = 0.91$.

The coefficient μ_1 depends on the ratio $H/B = 6/3.85 = 1.56$, $L/B = 3.85/3.85 = 1.0$, correspondingly $\mu_1 = 0.55$.

q_n – is the pressure at the level of foot portion of the equivalent foundation slab.

B – is the width of bottom of the equivalent foundation slab (Figure 3).

E_u – is the elastic modulus.

In this study the soil characteristics were obtained with the help of Cone Penetration Test. We will take the Young's modulus from the section 1.2 of the study. E_{m1}=26,12MPa, E_{m2}=16,56MPa.

The formula (11) is designed to calculate the average settlement of slender piled foundations that's why for the use of the non-yielding pile foundations when the computation the correcting coefficient 0,8 is introduced.

The average settlement of the piled foundation is equal to:

when E_{m1} $s_e = 0{,}8 \cdot \frac{0{,}91 \cdot 0{,}55 \cdot 202{,}43 \cdot 3{,}85}{26120} = 0{,}012\textit{м}$

when E_{m2} $s_e = 0{,}8 \frac{0{,}91 \cdot 0{,}55 \cdot 202{,}43 \cdot 3{,}85}{16560} = 0{,}019\textit{м}$

The results of the executed researches.

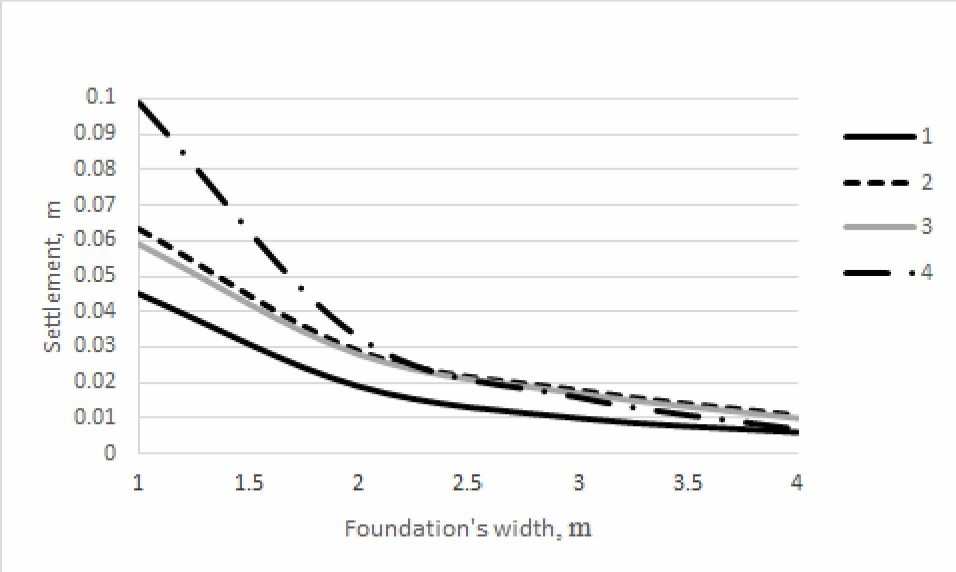

Figure 5 – Graph of settlements of shallow foundation subfoundation.
1 – method of layer by layer summing up,
2 – method of equivalent layer,
3 – adjusted method of elasticity Annex F [4] (when E = 26,12 MPa),
4 – Schmertmann's method.

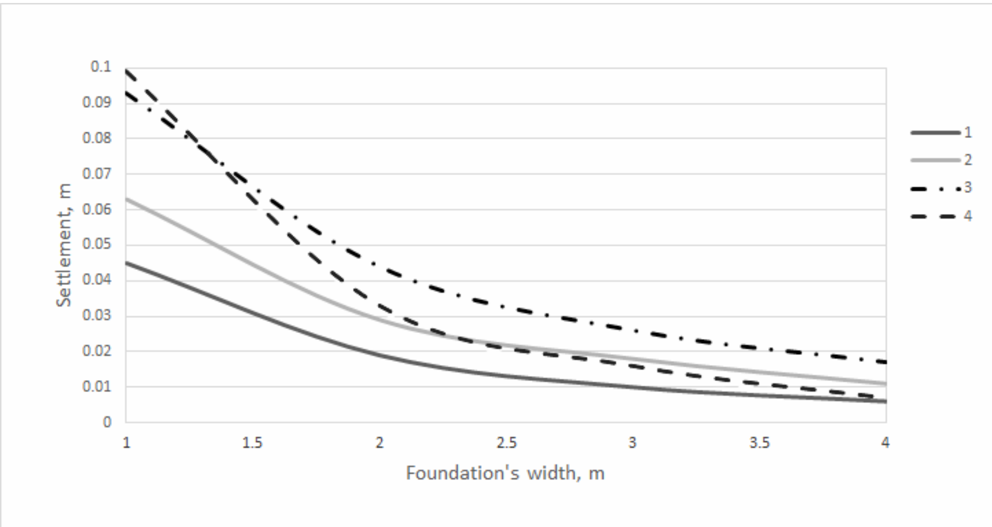

Figure 6 – Graph of settlements of shallow foundation subfoundation
1 – method of layer by layer summing up ,
2 – method of equivalent layer,
3 – adjusted method of elasticity Annex F [4] (when E = 16,56 MPa),
4 – Schmertmann's method.

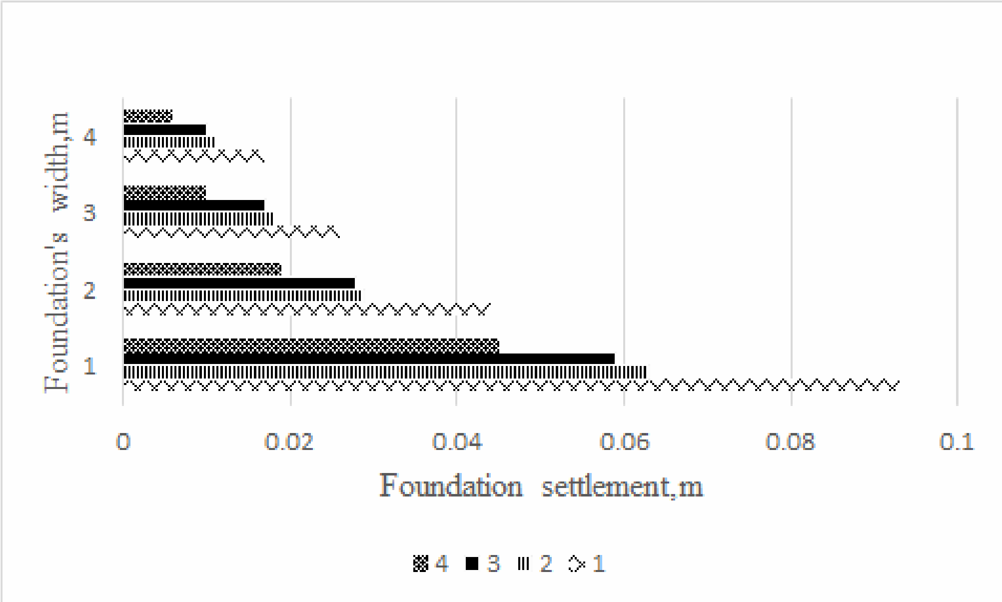

Figure 7 – Quantitative ratio of values of slabby subsurface foundation settlements
1 – adjusted method of elasticity Annex F [4] (when E=16,56MPa),
2 – method of equivalent layer,
3 – adjusted method of elasticity Annex F [4] (when E=26,12MPa),
4 – method of layer by layer summing up.

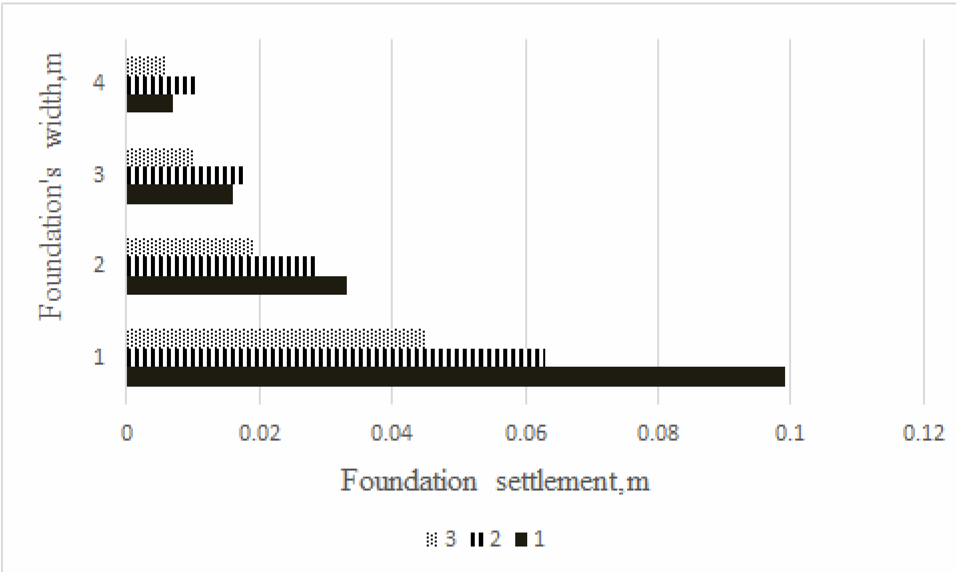

Figure 8 – Quantitative ratio of values of slabby subsurface foundation settlements:
1 – Schmertmann's method, 2 – method of equivalent layer,
3 – method of layer by layer summing up

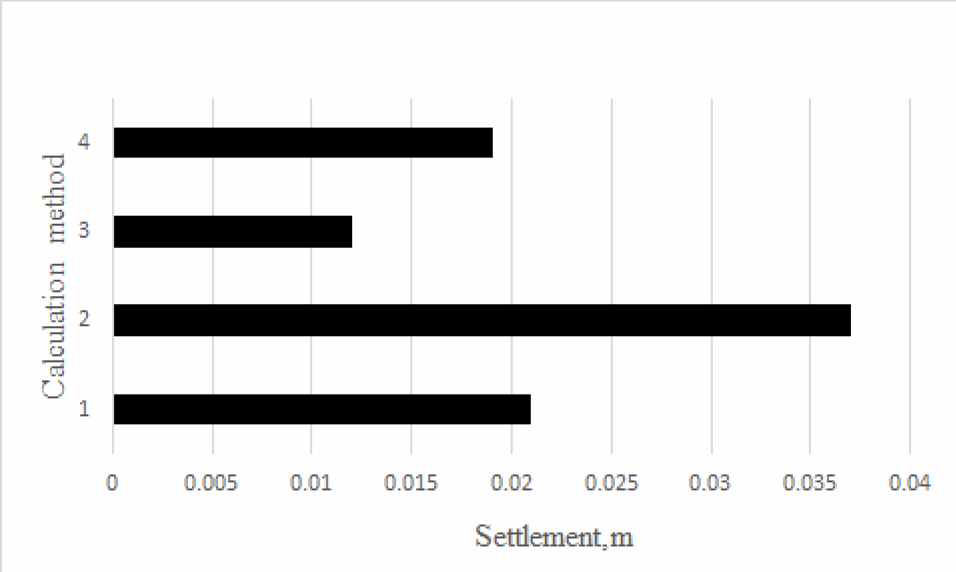

Figure 9 – Quantitative ratio of values of piled foundation settlements:
1 – method of layer by layer summing up,
2 – method of equivalent layer,
3 – adjusted method of elasticity Formula 11 (when E = 26,12 MPa),
4 – adjusted method of elasticity Formula 11 (when E = 16,56 MPa).

Conclusions:

1. The computation of the settlement of shallow foundations in sandy ground according to European standards as well as according to National standards is based on the same principles of the stress distribution in soil body. If there is the value of loads that correspond to impaction phase, so the values of the settlement using the method of equivalent layer, the Schmertmann's method and the adjusted method of elasticity (when E = 26.12 MPa) Annex F [4] are practically the same.

2. The difference between the settlement values obtained by the computation using the method of layer by layer summing up and the settlement values obtained with the use of the Schmertmann's method and the adjusted method of elasticity (when E=16,56 MPa) Annex F [4] amounts to 35-40%. This difference is due to the various approaches to the definition of E (modulus of deformation).

3. The length and the width of the conditional foundation according to National standards depend on type of the ground coat (drained angle of internal friction), situated along the length of the pile. According to European standards the inclination of profile planes to determine the equivalent foundation slab is always the same and it is equal to 1:4 (14^0). However, the dimensions of the conditional foundation base are obtained practically the same, due to the difference in determining the depth of the conditional foundation.

4. The value of settlement of the piled foundation in sandy ground when computation by the method based on the solutions of the theory of elasticity with the use of the coefficients that are determined by the graphs of Christian, J.T. and Carrier, W.D. Formula (11) when E=26,12 MPa is 42% less than the value of settlement with the use of the method of layer by layer summing up and almost 2 times less than the value of settlement with the use of the method of equivalent layer. The value of settlement of the piled foundation under the formula (11) when E=16,56 MPa is 10% less than the value of settlement under the method of layer by layer summing up and 49% less than the value of settlement under the method of equivalent layer.

5. The maximum limiting draft of settlement of house footings and buildings according to Eurocode 7 is accepted at the rate of 5,0 cm. Some well-known European geotechnical engineers [6, 11] suppose that the maximum limiting draft of settlement of foundations of framed buildings and buildings is at the rate of 7,5 cm (sand) and 13,5 cm (clay). According to National standards the maximum limiting draft of settlement of foundations of framed buildings and buildings is at the rate of 8 cm (for reinforced concrete structural framing) and 12 cm (for steel framework) and it does not depend on type of the ground coat under the foundation base.

6. The values of the Young's modulus when the computation according to European standards were determined either with the use of tables or with a help of the coefficient of correlation that don't take into account the ground conditions of the Republic of Belarus. It makes sense to develop the regional correlation dependences to determine the Young's modulus taking into consideration the soil resistivity under the penetrometer cone. Probably, then there will be no such divergence of the values of settlement according to National and European standards.

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Received 30.11.2016

UDC 69.059.7:725.4 (477.53)

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ANALYSIS OF THE INDUSTRIAL OBJECTS RENOVATION EXPERIENCE

Examples of foreign and domestic experience of renovation the industrial objects are considered. It is shown that in the European Union countries for the last 30 years, according to statistic research, the new construction and renovation index has increased. For the European practice, the expantion of renovation process from typical neutral objects to monuments of industrial architecture is a common thing. It has been noted that domestic practice is usually characterized by the composition of trading and office rooms in the environment of adaptation the typical neutral objects of industrial estate. Analysis of the examples showed that the renovation utilizes structural and planning, as well as aesthetic potential of the industrial buildings. Analysis results allow identifycation the options and directions for converting the inoperable industrial objects into civil buildings. This experience might be utilized for development the rational architecture-planning and structural solutions for reconstruction.

***Keywords:** renovation, inoperable building, reconstruction, converting.*

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АНАЛІЗ ДОСВІДУ РЕНОВАЦІЇ ПРОМИСЛОВИХ ОБ'ЄКТІВ

Розглянуто приклади закордонного і вітчизняного досвіду реновації промислових об'єктів. З'ясовано, що у розвинутих країнах Європейського Союзу за останні 30 років, згідно зі статистичними даними досліджень, зростає відсоткове співвідношення між новим будівництвом і реновацією; для європейської практики властиве поширення процесу реновації не лише на типові нейтральні об'єкти, але й на пам'ятки промислової архітектури. Відмічено, що вітчизняна практика реновації зазвичай характеризується формуванням торговельних і офісних об'єктів в умовах пристосування типових нейтральних об'єктів промислової нерухомості. Аналізом прикладів засвідчено, що при реновації використовується конструктивно-планувальний та естетичний потенціал промислових будівель. Доведено, що результати проведеного аналізу дають змогу визначити можливості й напрями перепрофілювання не- функціонуючих промислових об'єктів під будівлі громадського призначення, а отриманий досвід можна використовувати при розробленні раціональних архітектурно-планувальних та конструктивних рішень, які застосовуються при реконструкції.

***Ключові слова:** реновація, нефункціонуюча будівля, реконструкція, перепрофілювання.*

Introduction. Nowadays a significant number of large industrial enterprises completely or partially suspended production for economic reasons or to be passed beyond the city through sanitary harmfulness. Often located in the central areas of the city of factories are idle creating an environmental and aesthetic imbalance in a time when there is a deficiency of the city for the development of small and medium business sphere. The problem of overdue thanks to the renovation of industrial facilities under the buildings for public use. There is already international and domestic experience in the renovation of industrial objects and territories.

Analysis of recent studies and publications sources indicates that in the developed countries of the European Union for the last 30 years, according to statistics researchings [1], the percentage ratio between new construction and renovation. In General, starting from 2010 in Western Europe investment in renovations existing facilities on average make about 550 billion. euros per year, exceeding almost 1.5 times the volume of new construction. For the European practices with the distribution process of the renovation not only typical neutral objects but also in the interest of industrial architecture. Forming on their basis of public and residential facilities possible for the internal redevelopment of monuments of industrial architecture and while maintaining the facades. The domestic practice of renovation is usually characterized by the formation of a shopping and office facilities in terms of adaptations common neutral industrial estate. But the analysis of [2 – 6] indicates that the formation of retail and office centers based on the renovation of the existing industrial buildings in these works employ a practically independent.

Parts of general problem unsolved before. One of the directions of the perfection of the structure of modern cities is changing the functional purpose of industrial objects, partially or not at all. The creation of public buildings based on the renovation of industrial facilities provides an opportunity to effectively use existing industrial estate and at the same time to solve nazrìlì problems of development towns. In addition to the economic efficiency of the use of existing industrial buildings, renovation is a means of preserving historical kanvi bridge, solve the aesthetic and ethical problems of the existence of old factories. It should be noted that the percentage of industrial zones in the structure of modern cities is from 10 to 50% [5].

Problem formulation. The main aim of the research is the analysis of the foreign and domestic experience renovating industrial objects [7 – 9] that enable you to identify opportunities and diversifying them under a building for public use.

The basic material and results. In practice, most developed countries observed the formation of community facilities based on the preservation of objects of the industrial heritage, the multifunctional use of buildings and areas of active development of the transport network to form the frame of the city on the updated areas, landscaping and uncovering the natural capacity of the territories, creating a Visual connection with the surrounding environment. Analysis of international and domestic experience renovating industrial objects discovered spreading process of renovation at different structural levels of the industrial zones:

–industrial district;
– industrial site;
– industrial site;
– a group of industrial buildings;
– a separate industrial building.

From this perspective, it is advisable to consider the most famous examples of renovation. So one of the ways to use a full dismantling of existing and construction of new functions of the complex from scratch. An example of such a decision is the heart of the town renovation, Gelìzen (Germany) at the factory on manufacture of furnaces, designed by architect m. Koval′s′ki (fig. 1) [7].

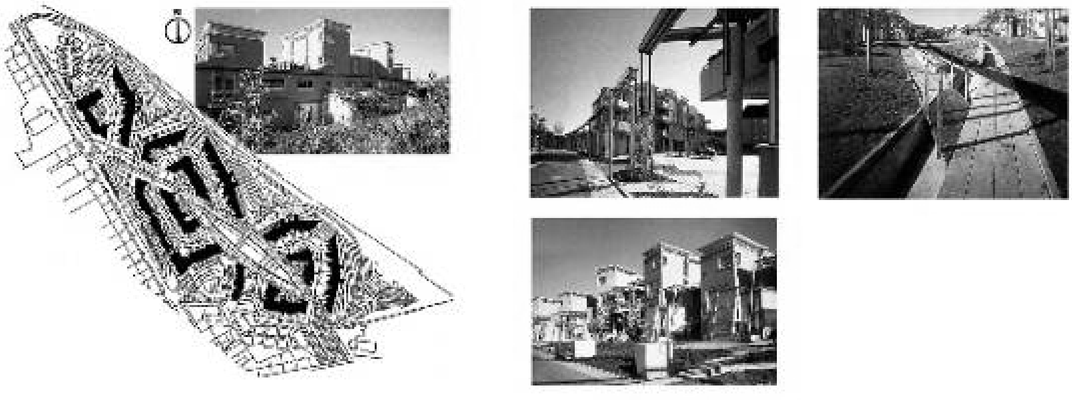

Figure 1 – Renovation of the quarter Gelìzen Heart (Germany) at the factory on manufacture of furnaces, designed by architect m. Koval′s′ki

However, with this approach substantially increases expenses for demolition objects, clearing and preparing for new construction, etc. Therefore, it is advisable to consider the examples of the various options for the conversion of industrial areas and objects change their functions, as well as to analyze the experience of different countries and architectural workshops in this direction. So, a typical example of such admission is a Center of Arts and media at Karsruê (Zentrum für Kunst und Medientechnologie Karlsruhe), Germany (fig. 2) [7]. Accommodation in 1997 and in buildings of industrial enterprise «IKWA-Karlsruê-Augsburg» modern Civic Center was one of the examples of the radical view of the role of the industrial object in the renewal of the urban landscape. An existing industrial complex of the object represented by the space-planning solutions a wide, height in three floors of the building blocks of the factory, symmetrically arranged around ten domestic branches. The building is made of concrete frame filling brick masonry on the fasadam. An abandoned in 70-ies of the last century, and later busy artists, the building was given the status of monuments of industrial architecture. Competition for the renovation, maintenance, and expansion of the plant won the architectural Studio ASP SCHWEIGER ASSOZIIERTE. Architects aptly preserved the building of 1918 and used new high-tech items.

Figure 2 – Center for arts and media in Karsruê, Germany (renovation of the industrial enterprise «IKWA-Karlsruê-Augsburg»)

For example, in order to eliminate the negative effects of noise and vibration on building a sound Studio was rendered by its Chairperson's factory in the form of a large glass cube in front of the façade. The courtyard was blocked lights that changed the internal environment, these architects have achieved the ideal of modern functional space. On the roof of the solar generators that supply the tramway path adjacent territories.

Particular attention during the renovation architects to transform the area around the building, with the aim of creating a natural complex, using the contrast between high technology and return to nature (at the exit of the building). The problem with the parking was solved thanks to a device recessed parkìnga along the building. The whole area above the parking lot is a lawn with modules made from steel sheets, keeping thus the industrial past of the object.

The next object that deserves attention is the renovation of the complex-holders in Vienna, Austria (fig. 3) [7]. Gasholders was built in 1896 – 1899. The building (62 m inner diameter, height is 72 meters) tanked for gas, but in 1970, they ceased to function and all technical equipment was dismantled. The remaining brick shell and 90000 m^3 inner space, protected by the State as a monument of architecture.

Reconstruction of the object consisted of 4-architectural workshops, each designed one of the four buildings in the complex. All the architects come to solve the problem differently.

In the Nouvel inner part consists of 9 segments that are located on a circle with a slight retreat from the existing walls. When renovating design the 14-floor accommodation. In the middle is the mall that is covering the dome, which has a relationship with all 4 me gasholders.

If all the architects formed only internal volumes, then Wolfe Priks proposed Supplement 3 – new forms, and one of them is a sticker. In the middle is a cylindrical volume with offices out – lomana flat shape of the screen, with Office space, and on the 1st floor is a multifunctional hall for public events, shops, and entertainment.

In the project Vedorn Architects in the Middle of gas holders' space is divided into 8 sectors, each of which height is divided into functional areas: housing, offices, trade, parking (top-down). Yard above the garage is blocked, a large glass dome, forming a public recreation area.

Wilhelm Holzbauer approached the design of the 4-th gasholders another image. In his project, there is no common interior space. From the middle of the full height rises cylindrical volume of the residential building. From his three blades extending cases that divided this whole inner volume 3 yard.

Building up renovation were the culmination of the industrial zone: absolutely closed, self-sufficient structure, the highest over the warehouses and wastelands. After renovation, they were the highlight of the entire area, but now it is not submitted by «skeletons», and attractive luxury offices, apartments, and shops.

Another interesting facility renovation in terms of originality of the interaction of historical building and the new building is the tall Centre Melbourne, Australia (fig. 4) [7]. Melbourne residents believe their city «the most technically equipped» in the southern hemisphere, so this is often called «Colosseum consumers». The author of this project is to architect Kìšo Norìakì Kurokawa.

Construction of the complex, located in the historic center of the city, was carried out in 1986 – 1991. It consists of a high-rise office building, shopping center, futuristic shape, as well as other institutions of cultural and entertainment destination. 55-story skyscraper hovering over a nearby trading center; in the processing of its facades used different materials: aluminum, stone, mirrored and tinted glass. In the construction definitely felt Japanese motifs. Part of the Mall became a huge 20-story glass Cone in the middle of which is a monument of Australian history – built in 1894, the brick Tower is the only preserved building of the former factory of lead pipes that once stood on this spot. In this case, the existing Tower not played special importance from the architectural point of view.

However, it is important a high dominant of the city of Melbourne, which is accustomed to city residents. It is part of the past, urban history, which Kurokawa carefully preserved, protecting the glass Cone, making it part of the Interior of the new shopping center.

Another, quite a good example of a renovation, the Russian Architects – Museum of water in the territory of the enterprise «Vodokanal» outlet (fig. 5) [7] (the renovation of the water tower), Saint-Petersburg, Russia. Reconstruction of the building of the water tower is the first in St. Petersburg experiences a revival of the old industrial buildings that have lost their former purpose. This project is an experiment in mixing styles of the XIX and XXI century. The main objective was to restore, clearing from later «layers» and adapting to the new features of interior spaces. Preservation of the integrity of the Interior – beautiful rooms with arched floors. The red-brick octahedron water tower, projected by Mertz and Šuberskìm in 1860 – 1863, associated with water only functionally: monolithic volume denies any fluidity. During the reconstruction failed to solve not only the question of content is placed in the Tower of the Museum «World Water», but also shaped. The requirements for the conservation of historic interiors of the towers caused the takeaway elevator and stairs in a building extension. It has become the main focus of reconstruction. In its forms, and the story can be read in the image.

As a negative example, or unfinished or failed, you can bring the Moscow Project Gallery Yakuta (fig. 6) [7]. An attempt to make art gallery in the building of gas holder's gas factory of Armagh, on the manners of the Austrian project has not been implemented. Create a glamorous Club and Gallery reflected only on the form of the building, and then failed. Monotone overlapping beams and slabs of internal space is not a good example of solving difficult problems. Gallery – this is just the first attempt to convert the factory territory in the Business Park, the planned installation of offices, trading.

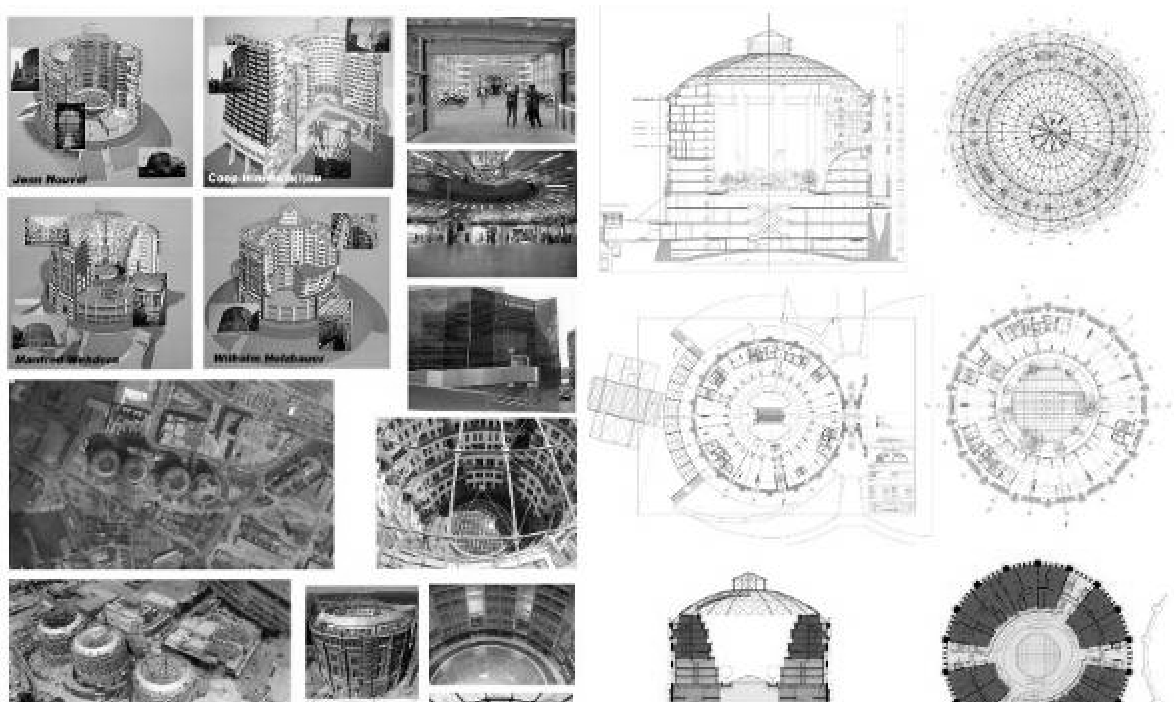

Figure 3 – Project of renovation of the complex-holders in Vienna, Austria

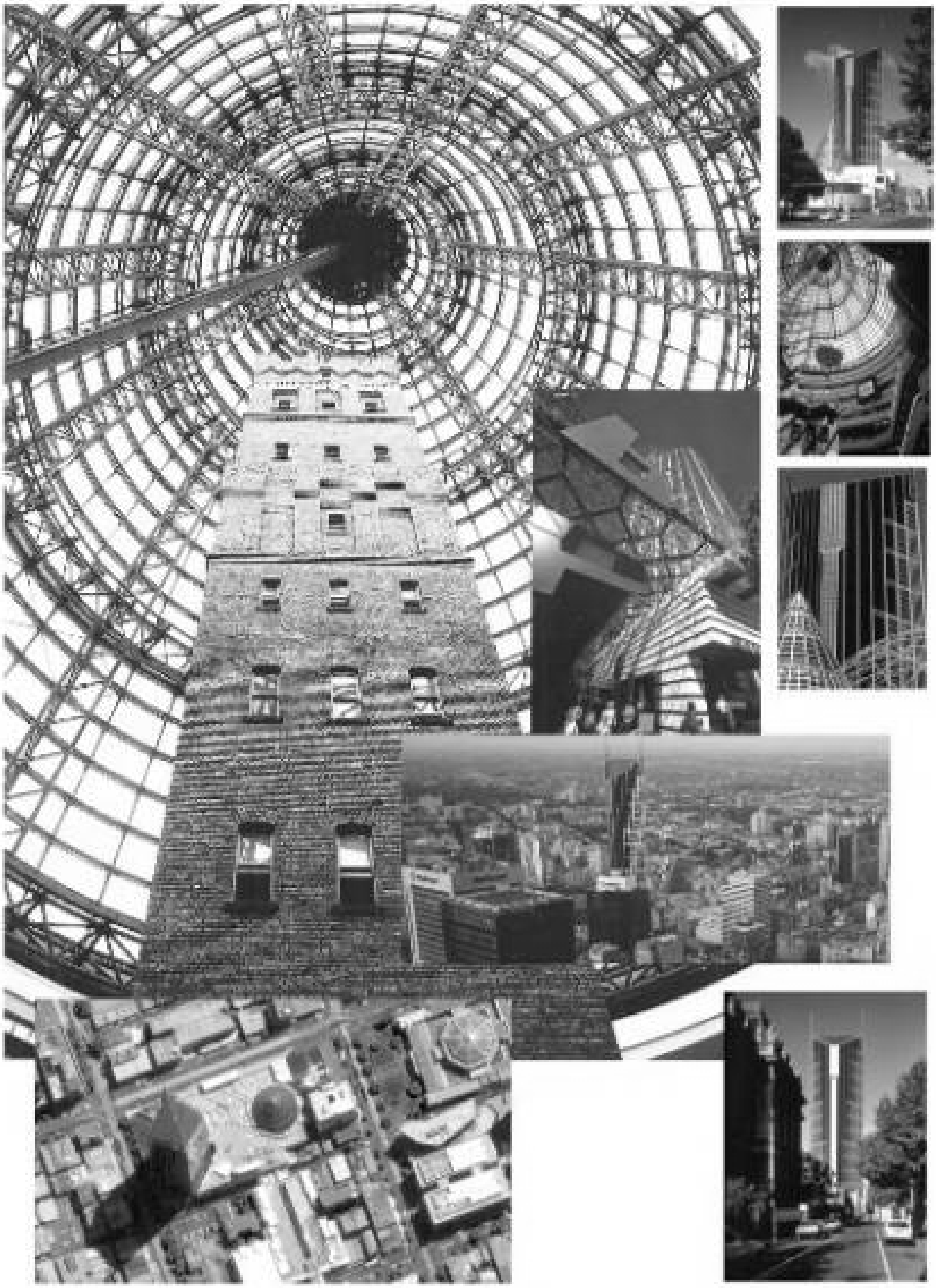

Figure 4 – The tall Centre Melbourne (Australia)
architect Kìšo Norìakì Kurokawa

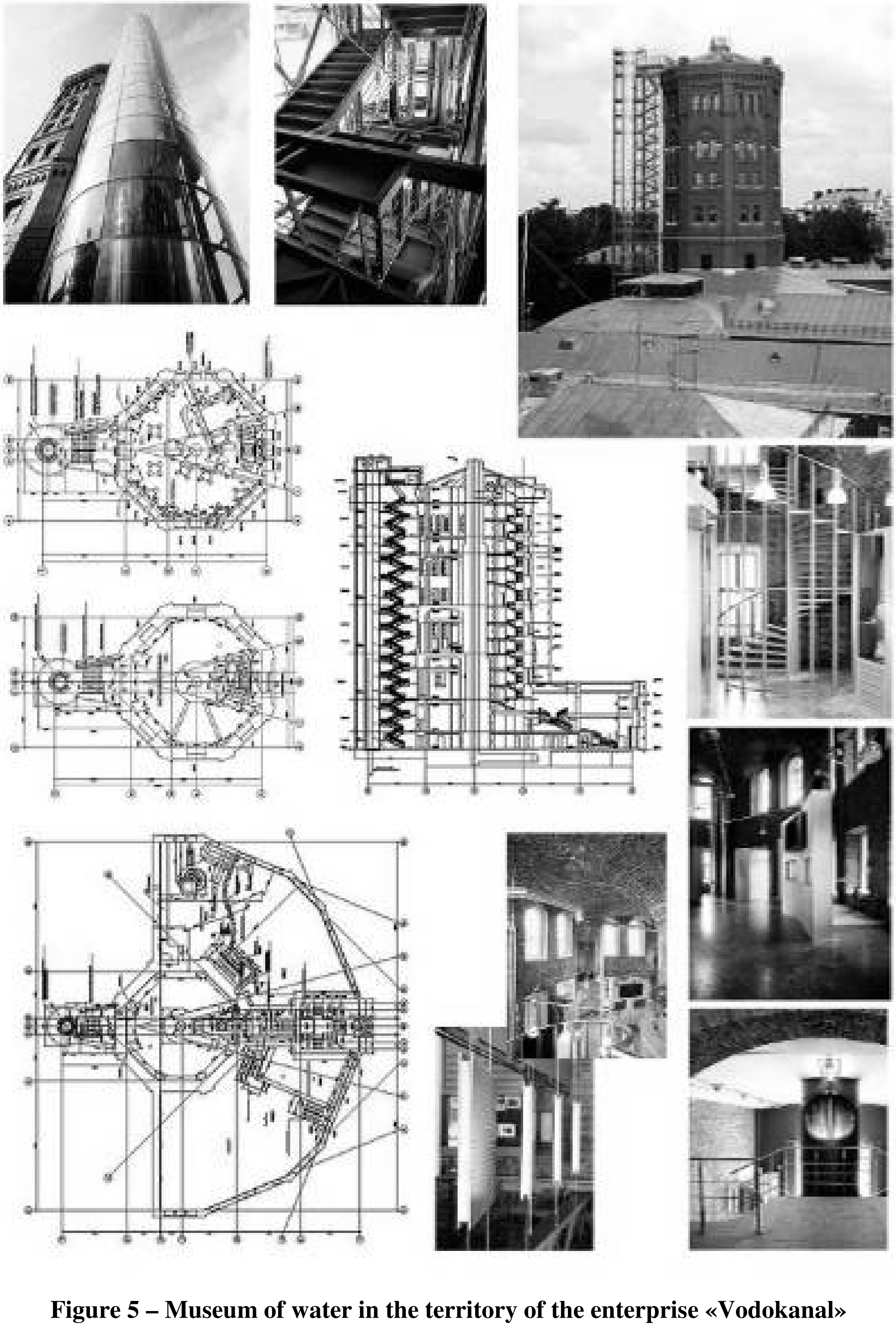

Figure 5 – Museum of water in the territory of the enterprise «Vodokanal» outlet (renovation of the water tower), Saint-Petersburg, Russia

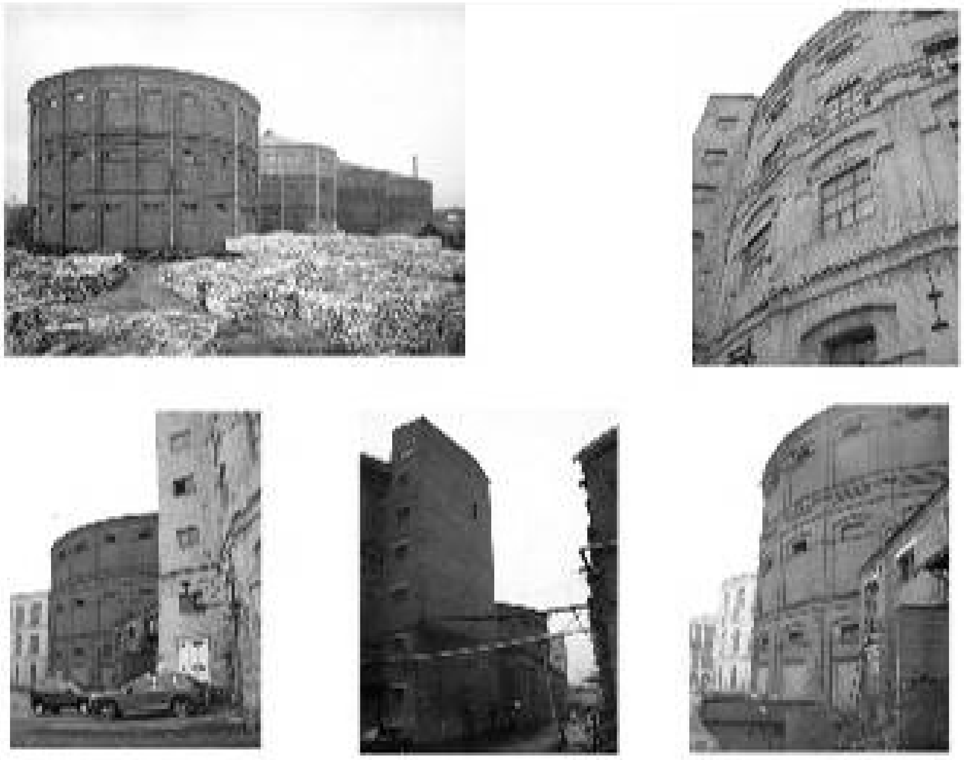

Figure 6 – Yakuta Gallery, Moscow, Russia

In conclusion the analysis of the existing experience of renovating, you should select the best, in our opinion, projects, industrial renovation (fig. 7) [2, 8], among which:

– The project of renovation of the coal power station, Battersea, London (United Kingdom), Figure 7, a . The project involves creating a theme park dedicated to the history of the English industry. The main target of this complex will be the unusual roller coaster going through all levels of a building, including, and inside;

– Museum of glass The Corning Museum of Glass (renovation of the factory on manufacture of glass), Kornìng, New York (United States of America), Figure 7, b;

– Office complex (renovation of the caramel factory), m. Lajnat (Italy), Figure 7, c;

– Art Gallery «Great Hall of pictures» (renovation of railway workshops), Provence (France), Figure 7, d;

– Multifunctional complex of Xintiandi Factory (plant renovation), Guangzhou (China), Figure 7, e, which combines a shopping center, as well as Office and hotel space. Thus according to the adopted concept of most industrial equipment it left as soon;

– Entertainment complex Tropical Islands Resort (the renovation of aviation hangars), Germany, Figure 7, f – a giant water park, located in the aviation hangar during World War II;

– Parking in the building of the Michigan theater, Detroit (United States of America), Figure 7, g – the only in the world of parking in the style of the Italian Renaissance. The irony is that the theater with a capacity of 40000 spectators, that gradually became unprofitable, was built at the beginning of the 20th century on the place of the first Henry Ford automobile plant shop;

– Rock climbing (renovation of silage towers of sugar factory), Montreal (Canada), Figure 7, h, k where deciding to keep the Interior of the marked building, the walls were decorated in the form of the rocks white, symbolizing the sugar stocks, which was once filled with silage.

  

a *b* *c*

  

d *e* *f*

  

g *h* *k*

Figure 7 – Forein examples of industrial renovation prijects:
a *– The project of renovation of coal-fired power plant Battersea, London (United Kingdom);* ***b*** *– Museum of glass (renovation of the factory for the production of glass), Kornìng, New York (United States of America);* ***c*** *– Office complex (renovation of the caramel factory), Lajnat (Italy);* ***d*** *– Picture Gallery Great Hall of pictures (renovation of railmay workshops), Provence (France);* ***e*** *– Multifunction complex Xintiandi Factory (the renovation of the plant), Guangzhou (China);* ***f*** *– Entertainment complex Tropical Islands Resort (the renovation of aviation hangars), Germany;* ***g*** *– Parking in the building of the Michigan theater, Detroit (United States of America);* ***h, k*** *– Rock climbing (renovation of silage towers of sugar factory), Montreal (Canada), the facade and interior of the building*

For the domestic practice characterized by the spreading process of renovation on a typical neutral industrial objects. Basically, you create objects that perform the trademark function. Despite the fact that in Ukraine there are a large number of nonfunctioning industrial areas, factories, factories, warehouses building cases when the objects of the real estate are given a second life, still, do not have a mass character.

Examples of the domestic refurbishment of industrial objects is represented in Figure 8:

– Shopping Center «Macros » erected on the basis of unfinished industrial plants, Kiev, the total area of 16000 m^2, the shopping area of 8300 m^2, (fig. 8, a). The functional structure of the Center: supermarkets, specialized shops, fast food restaurant, cafe, children's room, parking;

– Shopping mall «Gorodok», Kyiv was established on the basis of renovation of plant factory «Welding» (fig. 8, b), the total area of the Centre of 9000 m^2, shopping area of 7000 m^2. The functional structure of the Center: supermarkets of electronics, perfumes, and jewelry, a fast food restaurant. The line-up also includes specialty stores, beauty salon, children's room, Tour Desk, parking for 250 seats. Handling facilities are located on the back side of the building in two levels and make up a 28.9% of the trading area;

– Shopping complex «Promenada», Kyiv, Architects L. Merkulova, V. Zaplatnìkov, (fig. 8, c), based on the renovation of the building of the plant «Promkabel». While pereprofìlûvannì successfully uses the pro Gono VU structure of industrial building – top lìhtarne lighting in the direction of the runs. One by one are perfumery supermarket, venture catering, household appliances (first level) and food supermarket (at the end of the ground level with a convenient way of parking);

– Shopping and entertainment center «Megamax» Kyiv, Architect Y. Antoniuk, (fig. 8, d), the total area of 12700 m^2, shopping area of 6500 m^2, built on the basis of the refurbishment of the factory workshops «Promsvyaz», contains: supermarket goods and necessities (ground floor); grocery store of household appliances; cinema-mul′tìpleks; extra service in the form of a café, specialty shops, slot machines (on the second floor). Wide mesh supports (12 x 18 m) on the second floor has the possibility to place rooms for viewing. Food supermarket has a direct relationship with the parking. Serving the entire premises are grouped and arranged on two levels along the backside of the building façade;

– Shopping and entertainment center «Megamax» Kyiv, on the Uritskogo street, based on the refurbishment of the factory workshops «Promsvyaz», (fig. 8, e), the total area of 42000 m^2, trade area 31000 m^2. The functional structure of the Center: supermarkets, specialized shops, fast food restaurant, cafes, bars, cinema, parking;

– Renovation of part of the territory of turbo mechanical plant, located along the red line construction Zinkivska Street, Poltava. Well-developed infrastructure, access roads and parking lots to accommodate, as well as the proximity to the railway station, led to the choice of a new purpose for an industrial facility that is subject to reconstruction. It should be noted that when this was applied a few common techniques:

1) complete disassembly of the main building of the plant with a summary in its place the trade and entertainment complex «Kyiv», the architect of Dudnec R. P., (fig. 8, f);

2) change the administrative case of factory under the building for public use with the placement of commercial and Office premises, (fig. 8, g);

3) renovation of mechanical casing factory when building for public use with the placement of trading premises food market and commuter bus (fig. 8, h).

Renovation of buildings of the turbo mechanical plant is a shining example of a partial refurbishment of what is happening in the composition of the existing enterprises in the derivation of individual shops that do not meet the requirements of modern technology, providing them with public function;

– Renovation of the territory of Poltava plant «Prodmash» on the Covpaca street, 26 with the creation on its basis of trade and entertainment complex «Equator» (fig. 8, k) and the «City.com» (fig. 8, l). It should be noted that this territory is today a number of not functioning buildings that make up the prospect of the development of this urban area.

The domestic experience of mastering not functioning industrial certifies such negative phenomenon as a chaotic renovation – self-development of the territory on the basis of its distribution among private owners, characterized by multi functionality using schaterred or rebuilt territory, buildings, lack of forecasting the development of individual elements of the system [10, 11].

Figure 8 – Examples of domestic projects, industrial renovation:
a *– Shopping Center «Macros» (erected on the basis of unfinished industrial plants), Kyiv;*
b *– Shopping mall «Gorodok» (renovation plant factory «Welding»), Kyiv.;*
c *– Shopping complex «Promenada» (renovation of the plant «Promkabel»), Kyiv;*
d, e *– Shopping and entertainment centre «Megamax»*
(renovation factory workshops «Promsvyaz», Kyiv;
f *– Trade and entertainment complex «Kyiv», Poltava;*
g *– Trade-Office complex, Poltava;*
h *– Commercial premises, food market and commuter bus station*
(turbo mechanical renovation of the plant), Poltava;
k *– Trade and entertainment complex «Equator»), Poltava;*
l *– Shopping mall «City com» (renovation of the plant «Prodmash»), Poltava*

Conclusions. Reconstruction of not-functioning industrial buildings is one of the current options for improving the image of the city, which effectiveness is tested in many countries. The leading European experience analysis shows attractiveness for industrial objects reconstructed thanks to the determination of the existing industrial image of the object, subject to renovation. For the European practice is spreading the renovation not only typical neutral objects but also in the interest of industrial architecture. Among the domestic practice of building is characterized, as a rule, the formation of trade and office facilities in terms of adapting industrial estate. Despite the fact that Ukraine has a large number of nonfunctioning industrial zones (factories, plants, warehouses), cases of renovation when the objects of the real estate are given a second life, did not have the mass character. Analysis of the examples showed that the renovation is used constructively-planning and aesthetic potential of the industrial buildings. The results of the analysis, you can identify opportunities and directions for converting not functioning industrial facilities under the buildings for public use. You can use the experience gained in the development of rational architecture planning and design solutions that are used for reconstruction.

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Received 21.11.2016

UDC 697.12/.14

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LATENT HEAT ENERGY STORAGE DEVICE AS A PART OF THE VENTILATION SYSTEM OF INDIVIDUAL HOUSE

Energy efficient ventilation systems allow to minimize the heat energy consumption for heating supply air, which is very relevant in the context of rising energy prices. It is dealt with the variant of the installation seasonal phase-change heat storage device in the system of ventilation with recuperation of heat energy. Preliminary engineering calculations were perfomed and the quantity of heat storage material (water) necessary for the operation of the ventilation system was determined. The diagrams of change the temperature of the air at the output from the accumulator and distribution of water and ice during the heating period were given. In the article the calculation of seasonal heat storage device on the basis of the water in the supply air ventilation systems are considered , which allows to minimize costs of thermal energy for heating the outside air before it enters to the premises of the house. It is proved that in modern economy conditions development the energy-efficient ventilation system is a promising direction of research and implementations.
***Keywords:** heat storage device, heat recovery, individual house.*

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ТЕПЛОАКУМУЛЯТОР ФАЗОВОГО ПЕРЕХОДУ У СКЛАДІ СИСТЕМИ ВЕНТИЛЯЦІЇ ІНДИВІДУАЛЬНОГО БУДИНКУ

Визначено, що енергоефективні системи вентиляції дозволяють звести до мінімуму споживання теплової енергії для нагріву припливного повітря, що є досить актуальним в умовах зростання цін на енергоресурси. Досліджено варіант установлення сезонного теплового акумулятора в систему припливної вентиляції з рекуперацією теплової енергії. Наведено графіки кількості теплоти, необхідної для роботи системи вентиляції, діаграми зміни температури повітря на виході з акумулятора й розподілу води та льоду протягом опалювального періоду. Розглянуто розрахунок сезонного теплового акумулятора фазового переходу на основі води у складі припливної вентиляційної системи з рекуперацією теплоти, що дозволяє звести до мінімуму витрати теплової енергії для нагріву зовнішнього повітря перед подачею у приміщення житлового будинку. Доведено, що в умовах сучасної економіки розроблення енергоефективної вентиляційної системи є перспективним напрямком досліджень.
***Ключові слова:** тепловий акумулятор, рекуперація тепла, житловий будинок.*

Introduction. The rise of fuel and energy price is the most important stimulus of their economy, but any transition to energy-saving technologies will require additional investment and material costs. The rising cost of fuel and energy production contributes to inflationary processes, affects to the size and structure of non-productive consumption. The total impact of the rise in price of energy on the macroeconomic indicators is difficult to quantify [1].

In modern construction, especially in design of the building envelope with big thermal resistance, installation of the modern plastic windows, the main share of the heat load of the buildingis is accounted for the ventilation system. That is why in the current economic conditions with the rising cost of energy development of energy-efficient ventilation system is a promising direction of research and implementations.

In well heat insulated houses, ventilation systems use about 50 – 60 percent of heat. The use of the various designs of the air-to-air heat recovery unit allows to reduce ventilation load by 25 – 30 percents, but with the increase their coefficient of efficiency acute a problem of condensation and icing in this type of heat exchangers.

In the face of rising prices on different types of fuel and the government's commitment to energy independence there is need to develop new technologies which use alternative energy sources. In parallel with these questions arises the problem of efficient use and storage received energy, especially from the air.

Analysis of the latest sources of research and publications. The main task, structure and classification of a variety type of heat accumulators are considered[2, 3]. When using the latent heat of some substances the process of accumulation characterized by a high density of energy stored, small changes in temperature and stable temperature at the outlet of the heat accumulator. Such constructions of the phase change heat storage device are shown (PCHSD) (see Fig. 2) [4, 5].

The selection of not resolved before part of the general problem. There is a significant number of scientific papers devoted to the use of phase-change heat storage device on the basis of paraffin or hydrates, which reduce heat consumption by hot water systems and heating[6, 7, 8], but it should considerd very substantial part of the heat load of the building is the ventilation system. The article describes the calculation of the seasonal type heat storage device, which installed as part of the supply ventilation system with energy recuperation. This engineering solution allows to minimize costs of thermal energy for heating the outside air before it enters to the premises of the house.

Statement of the problem. The main aim of this work is preliminary monthly calculation of the seasonal type phase change heat storage device which isinstalled as part of ventilation system of the individual house, made on the basis of mathematical model of heat balance of water, air and ice.

Main material and results. Heat accumulators can perform the following tasks: compensation of the thermal energy consumption peaks; alignment (optimization) schedules of the thermal energy production by storing excess energy; accumulation of thermal energy, which will be used during the trip (lack of) energy supply.

Any heat storage device consists of: heat-accumulating material (HAM); thermal insulated storage tank to store HAM; systems for charging and discharging; auxiliary equipment.

The heat accumulators are distinguished:

1) the nature of HAM: thermochemical heat storage based on the allocation or absorption of heat in reversible chemical and photochemical reactions; heat accumulators which uses the heat capacity of the heat storage material without changing the state of aggregation; heat accumulators basis on phase transition, which use the latent heat of fusion of the substance;

2) the period of charging and discharging: short-term (up to three days); medium-term (up to one month); long-term heat accumulators;

3) the operating temperature: low temperature (100 °C); medium temperature (100 to 400 °C); high temperature heat storage (400 °C).

A wide range of substances exist, providing accumulation temperature from 0 to 1400 °C. It should be noted that the wide use of phase change heat storage device is constrained primarily by considerations of cost-create installations.

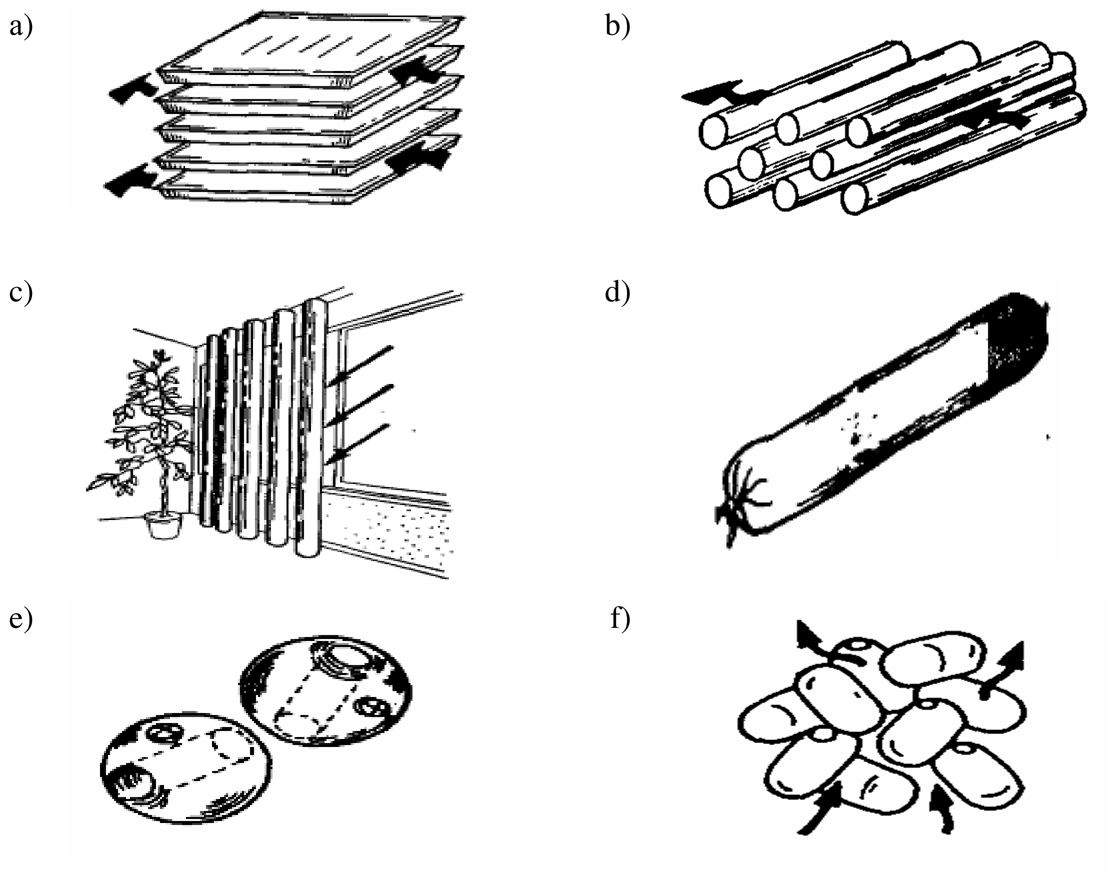

Figure 1 – Main type of the phase change heat storage device:
a – panel; b – pacilic; c – polyethylene pipe which used in passive system of use solar energy; d – pipe; e – balls with through channels for air; f – capsules

When using a thermal effect that arises as a result of the heating and cooling of water or gravel, the quantity of heat produced is small comparated to with the size of the heat accumulator. A disadvantage of this type of heat storage is that it requires significant space and in the process of heat transfer the temperature of the accumulator is reduced. At phase transformations (melting, evaporation and crystallization) allocated the so-called latent heat of phase change, and the quantity of generated heat can be quite significant. If, for example, take water, to obtain 1 kg of water from ice we need 336 kJ/kg and to evaporate 1 kg of water necessarily already 2268 kJ/kg. In the process of evaporation has significantly more heat, but there are significant changes in the volume of the HAM, so the process is advantageously carried out in the heat storage device. Therefore, it is possible to use only the latent heat of melting [4, 9].

Mathematical model to calculate PCHSD will consist of stationary equations of heat balance of air, water and ice under the condition of equality of the final temperature of air and water.

$$\begin{cases} c_{air} \cdot G_{air} \cdot (t_{air}^{in} - t_{air}^{out}) = c_w \cdot G_w \cdot (t_w^{in} - t_w^{out}) \\ c_{air} \cdot G_{air} \cdot (t_{air}^{in} - t_{p.c.}) = r \cdot m_{ice} \end{cases}, \quad (1)$$

where c_{air}, c_w – specific heat respectively for air and water, kJ/kg·K;

G_{air} – mass flow rate of the air supplied to regenerator, kg/h;

t_{air}^{in} – initial air temperature at the inlet to the heat storage device (assumed to be the average monthly temperature t_m, K);

t_{air}^{out} – final temperature of air at the outlet of the thermal battery, K;

$t_{p.c.}$ – phase change temperature of the heat-accumulating material, K;

t_w^{in}, t_w^{out} – respectively the initial and final temperatures of HAM (water), K;

m_w, m_{ice} – mass of water and ice, kg;

r – latent heat of crystallization, for water is 336 kJ/kg.

The use of energy-efficient ventilation system with series connection of the PCHSD and plate heat exchanger is considered on the example of an individual house with total living area of 100 m^2.

The total air flow rate for individual house is:

$$L_{tot} = L_{room} + L_k + L_b, \quad (2)$$

where $L_{room} = F \cdot K$;

F – living room area, m^2;

K – air changes per hour, 1/hour.

L_k, L_b – values of air flow rate, respectively, for living rooms, kitchens and bathrooms, taken in accordance with the norms [10], m^3;

The working time of ventilation system is 720 hours in a month. The design air temperature in the living room is +18°C. The supply air temperature can be reduced on 3°C from given that the room air distribution is carried out through the grille, located under the room ceiling. Total air flow rate for the house is equal L_{tot} = 100 + 90 + 50 = 240 m^3.

For calculations, one should take into account the change of the outdoor temperature during the heating period. For example, you can use the regulatory document [11] to determine the average temperature of the heating period in Ukraine, table 1.

Table 1 – Values of average for the month air temperature for city Poltava

Month	October	November	December	January	February	March	April
t_m,°C	7,7	1,3	-3,4	-5,6	-4,7	0,3	9,0

Modes of operation for heat storage device: in winter – discharging (temperature range from t_m÷0 °C); summer – charging, returns accumulated during the winter cold for the needs of the air conditioning system. Working temperature range for plate heat exchanger – heating air from 0°C to supply air temperature. Climatic data are taken for city Poltava and made a preliminary calculation of the phase change heat storage device with use the mathematical model, the results shown in figures 2, 3 and 4.

From the graph, Fig.2, it is seen that the inclusion of heat storage device as a part of the ventilation system keeps the outlet air temperature is not lower than 0 °C, which prevents icing of the plate heat exchanger.

In the diagram (Fig.3) shows the amount of heat that is released or absorbed by the heat storage material, respectively, in the discharging and charging working modes of the seasonal heat storage device. The following it is given a graph of gradual freezing of the ice in the accumulator during the heating period, his maximum weight is 9318 kg.

From the diagram (Fig. 4), we can see that the main mass of the ice formed during January – February, further it is possible to observe the process of his melting.

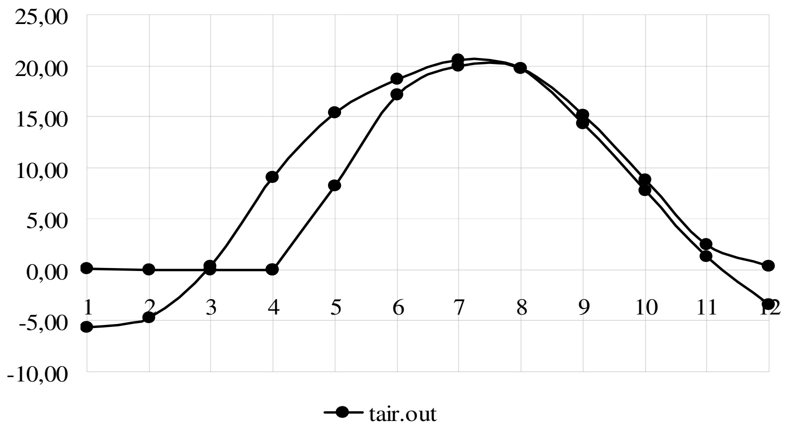

Figure 2 – The graph of air temperature distribution at the output from the heat storage device and the average for the month outdoor air temperature

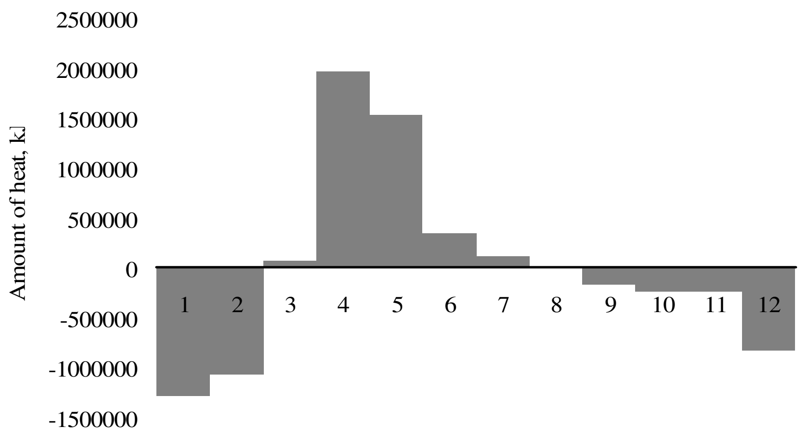

Figure 3 – The chart of monthly distribution of the amount of heat which gives HAM for heat the outside air

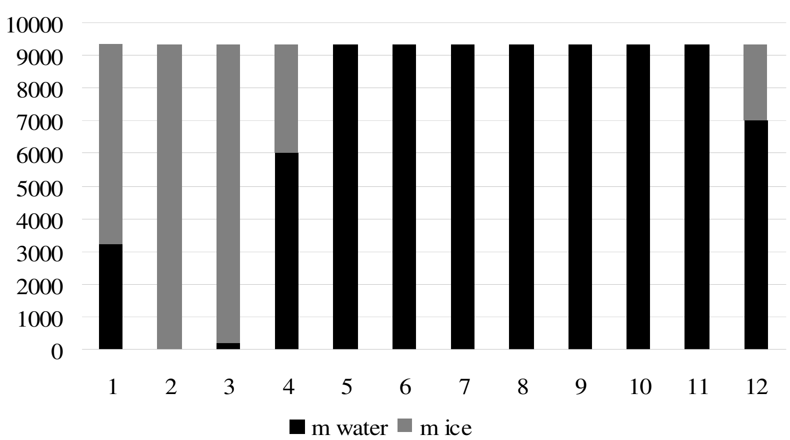

Figure 4 – The graph of monthly distribution of the mass of ice and water in the heat accumulator throughout the heating period

Conclusions. The prospect of switching to supply ventilation system with heat recovery PCHSD is considered that allows to minimize the use of heat energy for ventilation needs of the house and avoid such negative processes as the icing of the plate heat exchanger. On the basis of the equations of thermal balance of the heat storage device, preliminary calculation were performed and graphically the temperature distribution at the exit of the accumulator was depicted, the distribution of the quantity of heat and changes in the amount of water and ice in the heat exchange process was charted.

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Received 24.05.2016

UDC 628.161

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THE EFFICIENCY WATER CLARIFICATION IN MODELS OF VERTICAL TANK

The purpose of this article is to analyze theoretical and experimental data on the structures and analyze the performance of type clarifier, as well as to highlight the shortcomings of the proposed designs. It was found that for the experiments it is necessary to create a physical model of the vertical sediment tank in the laboratory using the following factors: material costs, technological parameters, design parameters. The purpose of this article is to show the effect of the velocity of upward flow of water inside the vertical settler on the efficiency of its lighting on the physical model of the vertical sediment tank. There are mathematical dependences of turbidity of clarified water as well as the efficiency of water clarification from the turbidity of the incoming water for different speed values in the lighting area of the vertical sediment tank. It was established the use of the calculations required area lighting area, vertical settling tanks, water speed, cleansing of water without efficiency loss, which leads to significant savings in capital investments.

***Keywords:** water treatment, vertical sump, the efficiency of water clarification, water turbidity, the speed of the water upward movement.*

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ЕФЕКТИВНІСТЬ ОСВІТЛЕННЯ ВОДИ У МОДЕЛІ ВЕРТИКАЛЬНОГО ВІДСТІЙНИКА

У статті проаналізовано теоретичні та експериментальні дані щодо конструкцій та аналізу роботи відстійників цього типу, а також виділені недоліки запропонованих конструкцій. Встановлено, що для проведення експериментів необхідно створити фізичну модель вертикального відстійника в лабораторії використовуючи наступні фактори: матеріальні затрати, технологічні параметри, конструктивні параметри. Розглянуто вплив швидкості висхідного потоку води всередині вертикального відстійника на ефективність її освітлення на фізичній моделі вертикального відстійника. Отримані математичні залежності каламутності освітленої води, а також ефективності освітлення води від каламутності вхідної води для різних значень швидкості її в зоні освітлення вертикального відстійника. Встановлено, що використання у розрахунках необхідної площі зони освітлення вертикальних відстійників швидкості руху води у ній, при якій відбувається очищення її без втрати ефективності освітлення, призводить до суттєвої економії капітальних вкладень.

***Ключові слова.** водопідготовка, вертикальний відстійник, ефективність освітлення води, каламутність води, швидкість висхідного руху води.*

Introduction. After the Soviet collapse remained water treatment plant in Ukraine worked at 70-80% of its design capacity. Nowadays, tariffs growth for utilities has significantly reduced the consumption of water by population and industrial enterprises. It has led to irrational use of land resources, electricity the like. The construction of a modern municipal water treatment plants requires compliance with the quality requirements of water purification, which are governed by the regulatory documents. On the other hand, the deterioration of water quality from surface sources requires increasing the effectiveness of water treatment to meet drinking needs of the population [1–3]. It would be achieved by retrofitting municipal water treatment plants and modern water purifying installations [4].

The use of existing industrial water purification plants with small capacity vertical tanks for receiving water output of the necessary quality is more economical than the use of sedimentation tanks of other types. Vertical sumps do not require large areas. They can be grouped in blocks and have sufficient indicators of water clarification efficiency.

Analysis of recent research sources and publications. Improving the efficiency of water clarification in vertical settling tanks is possible by reducing the speed of water upward flow in the clarifying zone, but it will lead to the need of increasing their size or quantity. It is considered in details process of water clarification in sedimentation tanks. Settling (sedimentation) of suspended particles consisting of mineral particles, as well as, under some assumptions, and unstable suspensions is described by the linear law of Stokes

$$F_C = 3\pi \cdot \mu \cdot u \cdot d \ , \tag{1}$$

where μ – coefficient of fluid dynamic viscosity;

u – the settling velocity of the particle;

d – particle diameter.

This law is valid for particles of small size deposited with low speed when viscosity forces only affect the resistance to their motion. More generally, the drag force of particle settles is represented by Newton-Rayleigh's law. [5]

$$F_C = \psi \cdot \rho_1 \cdot u^2 \cdot d^2 , \tag{2}$$

where ρ_1 – the density of the liquid;

u – particle sedimentation rate;

d – the equivalent particle diameter;

ψ – the coefficient of particles sedimentation, which depends on the Reynolds number and is determined by the expression

$$\psi = \frac{\rho_2 - \rho_1}{\rho_1} \cdot g \cdot \frac{\pi \cdot d}{6 \cdot u^2} , \tag{3}$$

where ρ_2 – the density of the particle.

In view of this, the sedimentation rate of the particles of certain type can be determined by the formula

$$u = \sqrt{\frac{\rho_2 - \rho_1}{\rho_1} \cdot g \cdot \frac{\pi \cdot d}{6 \cdot \psi}} . \tag{4}$$

Dependence of resistance coefficient on the Reynolds number is set empirically. Zegzhda V. P. obtained experimental data of sedimentation of grains of sand and gravel of different grain size [5].

At small values of Reynolds number in small particle sizes and small values of the speed of their subsidence, the dependence of resistance coefficient on the Reynolds number has direction at an angle of 45°. With the increase in the settling velocity of the particles and their size, linear law can be broken.

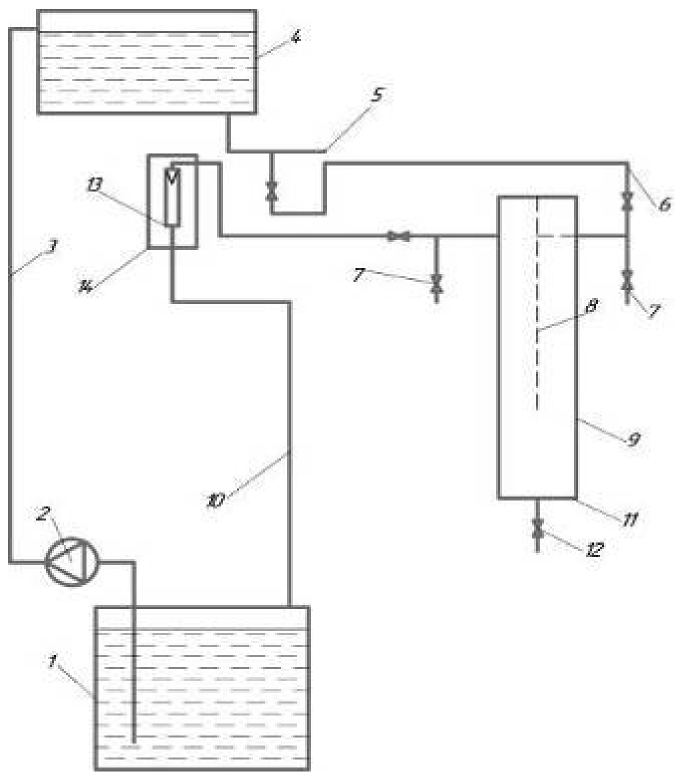
The limit of justice of the linear law is determined by the value of Reynolds number equal to 1.0. Suspended solids contained in water from natural sources have different shape and hydraulic size. The curve of suspended matter sedimentation, obtained experimentally, allows to determine the average value of its hydraulic size [6]. These curves determine the necessary residence time of water in the sump, corresponding to the required value of the illumination efficiency.

Selection of the unsolved parts of problem. Technological simulation of sedimentation of water in the sump [7] is to determine under laboratory conditions the design parameters for settling tank – the sedimentation velocity of suspension and the length of water stay in it that provides given effect of water clarification. This method is based on the similarity of the curves suspended solids precipitation obtained for different values of t cylinder laboratory height where it is settling.

In view of movement lack in laboratory cylinder, the values of energy efficiency obtained from the results of such experiments will be different from the values obtained for ponds in which there is water movement.

According to norms [8], the ratio of tank diameter to its height must be 1.0 or 1.5. Speed of water movement in the area of lighting vertical tank for reagent-free cleaning should not exceed 0,08-0,15 mm/s. Such a wide range of water velocity values can lead to the need for specific zone area lighting septic tanks, which will also depend on the total water consumption. For example, water consumption of 3,000 m^3/day or 125 m^3/hour, required area lighting zones septic tanks, depending on the water speed in it will be in the range of 300 to 560 m^2.

Statement of the problem. The aim of this work was to obtain the dependence on water clarification efficiency from the turbidity of the incoming water at different values of its movement speed in the lighting area. For the experiments it was established the physical model of the vertical sediment tank (Fig. 1), which consists of the camera flaking (Central tube) and lighting zones and accumulation of sediment (case) [9].



1 – tank for clarified water;
2 – circulating pump, clarified water;
3 – duct circulation of clear water;
4 – tank for preparation of working solution with paddle stirrer mechanical type;
5 – «comb» to distribute the supply of working solution among settings;
6 – the pipe feeding the working solution in a sedimentation tank;
7 – ball valve for sampling water;
8 – camera flaking (Central tube);
9 – sump body;
10 – the pipeline of the withdrawal of the clarified water;
11 – area of sediment accumulation;
12 – ball valve for sediment sampling;
13 – rotameter;
14 – panel for mounting flow meter

Figure 1 – Installation diagram model of the vertical sediment tank with flowmeter

General material and results. To simulate the process of water precipitation as research water it has been used cloudy solution obtained after mixing of clean water with sand or loam. Before starting work, each time it was carried out the calibration of the flow meter by the volumetric method. To do this, it was measured the time in which the filled measuring cylinder with certain specific indications of the rotameter costs. According to the results of the calibration it was constructed graphic dependence of the water movement rate in the clarifying zone model of the clarifier of flow rate with rotameter (Fig. 2).

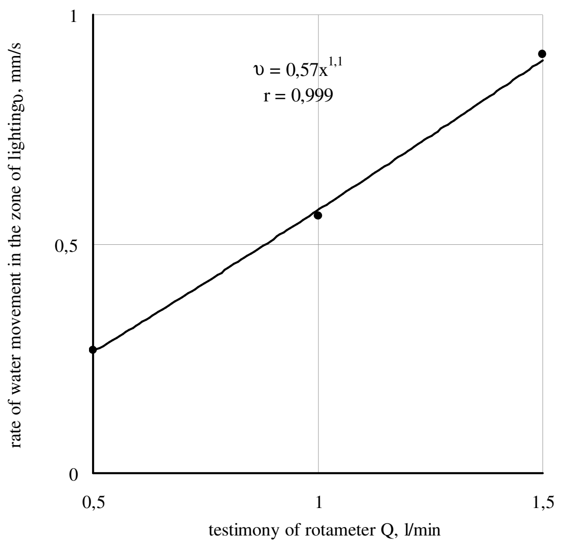


Figure 2 – Tiruvalla curve to determine the value of the rate of water movement in the zone of lighting according to the testimony of rotameter

The turbidity measurement was conducted as follows. The research sample of water was taken in cuvette and set in the single-beam photoelectric colorimeter (RPO-1). The second cell selected distilled water and with the help of the device determined the amount of light transmission through both the cuvette and thiruvallam schedule. It was determined by the test sample turbidity. Similarly, it was determined water sample turbidity at the outlet of the septic tank. The results of the experiments are shown in figures 3 – 5. The magnitude of the motion velocity in the lighting area of the vertical sediment tank 0,169 mm/s corresponds to water flow according to indications of the flow meter 0.5 l/min, and the speed 0,417 mm/s – 1, 5 l/min.

The results of determining the turbidity of clear water, depending on the velocity of its motion in the lighting area of the obtained graphical dependence on water bleaching effect in the physical model of vertical sump from the turbidity of the incoming water are shown (Fig. 6 – 7).

For example, it is calculated the required area of vertical sedimentation tank for reagent-free lighting cloudy water flow m^3/day. It is calculated the required area of illumination zone [8, formula (8)]

$$F_{в.о.} = \beta_{об} \cdot \frac{q}{3{,}6 \cdot v_p \cdot N_p} = 1{,}3 \cdot \frac{3000}{3{,}6 \cdot 24 \cdot 0{,}15 \cdot 3} = 100{,}3 \text{ m}^2, \tag{5}$$

where $\beta_{об}$ – factor considering the use of tank volume, the value of which adopted 1.3 (ratio of the diameter of the tank to its height as 1,0);

v_p – the estimated rate of upward water flow [8, tab. 16] is adopted for the lighting of water for industrial purposes $v_p = 0{,}15$ mm/s;

N_p – number of working septic tanks (tentatively accepted N_p =3 pcs.)

If vertical tanks used for preparation of drinking water, the design speed of the rising water flow in the area of lighting, the reagent-free cleaning must not exceed ν_p = 0,08 mm/s [8, table. 16]. In this case, the required area of vertical clarifiers is

$$F_{в.о.} = \beta_{об} \cdot \frac{q}{3{,}6 \cdot \nu_p \cdot N_p} = 1{,}3 \cdot \frac{3000}{3{,}6 \cdot 24 \cdot 0{,}08 \cdot 3} = 188{,}1 \text{ m}^2, \quad (6)$$

As it is shown in Fig. 7, the efficiency of water clarification in the model of vertical sediment tank with the speed of motion in the lighting area 0,417 mm/s has quite high value from 43 to 86% (average value $E = 72\%$).

The required area lighting vertical zones septic tanks at speed of 0.417 mm/s, without the use of reagents will be important

$$F_{в.о.} = \beta_{об} \cdot \frac{q}{3{,}6 \cdot \nu_p \cdot N_p} = 1{,}3 \cdot \frac{3000}{3{,}6 \cdot 24 \cdot 0{,}417 \cdot 3} = 36{,}1 \text{ m}^2, \quad (7)$$

Thus, when calculating the required area lighting vertical settling tanks, the speed of water movement in the clarifying zone is 0.4 mm/s, in contrast 0,08...0,15 mm/s [8, tab. 16]. Savings of capital investments in the construction of the sumps will be: when cleaning industrial water to 64%, and in the purification of drinking water – up to 81%.

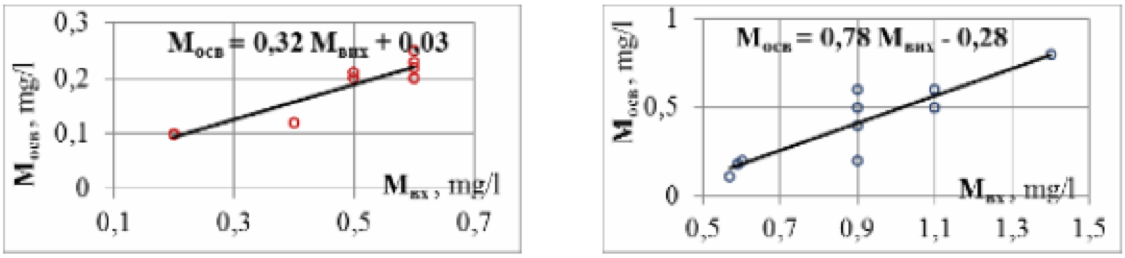


Figure 3 – The dependence on the turbidity of the clarified water $M_{осв}$ from the input $M_{вх}$ for sand

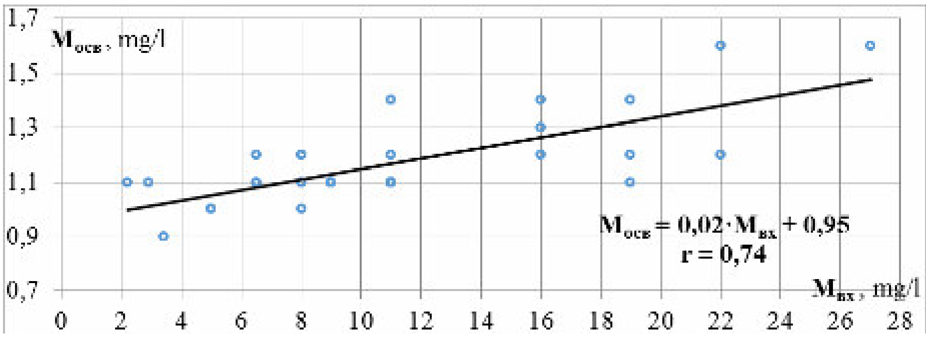


Figure 4 – The dependence of the turbidity of the clarified water $M_{осв}$ from $M_{вх}$ input to loam at a speed 0,169 mm/s

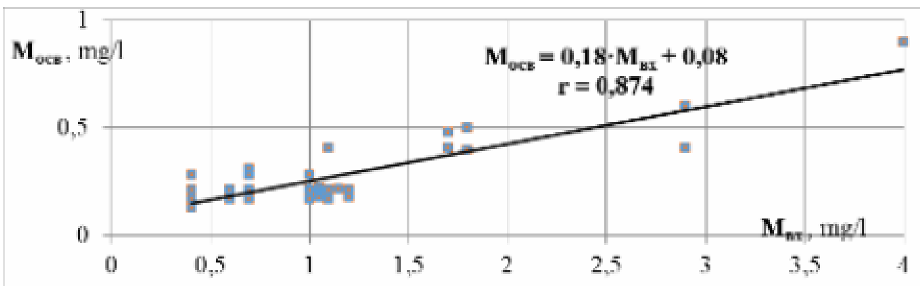


Figure 5– The dependence of the turbidity of the clarified water $M_{осв}$ from $M_{вх}$ input to loam at a rate of 0,417 mm/s

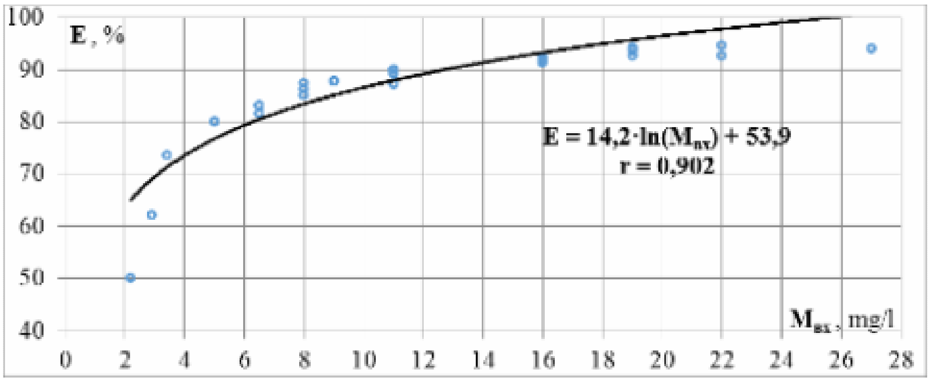


Figure 6 – The dependence on the effect of water clarification in vertical sump model E from the turbidity of incoming water $M_{вх}$ when the velocity of its motion in the lighting area is 0,169 mm/s

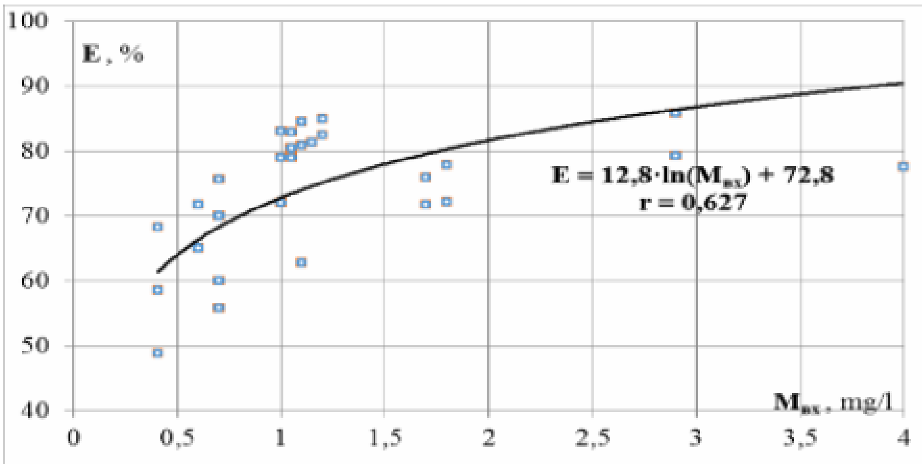


Figure 7 – The dependence on the water clarification effect in a model of vertical sump from the turbidity of the incoming water when the velocity of its motion in the lighting area is 0,417 mm/s

Conclusions.

1. The study of the water clarification process on physical model of the vertical sediment tank showed that water purification without the use of reagents can increase water speed in the clarifying zone to 0.4 mm/s (in contrast to suggested standards [8] 0,08–0,15 mm/s); lighting efficiency remains rather high (average value $E = 72\%$).

2. Water speed increase in the area of lighting to 0.4 mm / s reduces the area of the zone coverage to 64% – for industrial water and up to 81% – for drinking water; the results can be used in the design of new and existing cleaning operation structures with vertical tank.

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Received 13.02.2017

UDC 625.7

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ROADWAY ROUGHNESS RESEARCH AND CAUSES DETERIORATION ANALYSIS

The article deals with modern directions of domestic and foreign smoothness research coverage on the roads. The problem of causes changes establishing in smoothness coverage related to the irregularities in the procedure of road construction layers is highlighted. The research results of the trafficway smoothness and its causes deterioration analysis, performed by operation of roads and airfields laboratory at National Transport University on research road area H-18 around the city Buchach is shown. By the research results the road profile is drawn and the detailed analysis of road topping smoothness changes during road operation is done. Samples at the specific points on the road topping is taken: in one place it is a transverse crack, in another – without noticeable defects. It is established that road profile hollows and transverse cracks caused by black layers uneven thickness along the road.

***Keywords:** longitudinal smoothness, road, road topping, pavement design.*

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ДОСЛІДЖЕННЯ РІВНОСТІ ПРОЇЗНОЇ ЧАСТИНИ Й АНАЛІЗ ПРИЧИН ЇЇ ПОГІРШЕННЯ

Розглянуто сучасні вітчизняні та закордонні напрями дослідження рівності покриття на автомобільних дорогах. Виділено проблему встановлення причин зміни рівності покриття, що пов'язані з порушенням технології влаштування шарів дорожньої конструкції. Наведено результати дослідження рівності проїзної частини й аналізу причин її погіршення, виконаних лабораторією експлуатації автомобільних доріг та аеродромів Національного транспортного університету на дослідній ділянці автомобільної дороги Н-18 навколо м. Бучач. За результатами досліджень побудовано поздовжній профіль і проведено детальний аналіз зміни рівності дорожнього покриття в процесі експлуатації дороги. Узято керни в характерних точках на покритті: в одному місці проходить поперечна тріщина, в іншому – без помітних дефектів. Установлено, що западини поздовжнього профілю і поперечні тріщини обумовлені нерівномірністю товщини чорних шарів вздовж дороги.

***Ключові слова:** поздовжня рівність, автомобільна дорога, покриття, конструкція дорожнього одягу.*

Introduction. Since 2004, the laboratory of operation of roads and airfields at National Transport University (hereinafter – NTU) in research roads areas, which are located in different Ukraine regions, it is conducted regular surveys, consisting parts of what is the smoothness and wheel tracking parameters determination, and also road-mat deformation. The disadvantages of smoothness assessing methods of road topping by viameter or IRI is that they refer to the area altogether and does not allow to highlight points, where is the maximum indications accumulation. In addition, they do not reveal the cause of smoothness deterioration, which is due to its degradation.

Generally this problem can be characterized as smoothness informative indicators lack used in modern regulatory and technical base of Ukraine. Therefore, an essential part of survey research areas are leveling and road profile drawing and also its change analysis during the road operation and revealing the causes of smoothness deterioration.

Review of research and publications recent sources. Road and airfield topping smoothness research can be divided into the following directions:

– smoothness indicators justification [1 – 4];
– smoothness normative values justification [5 – 7];
– methods, techniques and means of smoothness measuring development [8 – 11];
– smoothness impact on the speed and transport mode studying [13];
– smoothness impact on traffic safety and accident occurrence studying [12, 13].

The issue of technological supporting of road topping and smoothness revealing causes in scientific and technical literature it is paid much less attention, except for a few studies [11, 14].

Parts of the common problem that earlier unsolved. When you travel on the highway H-18 Ivano-Frankivsk – Buchach – Ternopil (detour of city Buchach) it is felt periodic blows caused by road topping irregularities presence. The question arose about this phenomenon causes establishing. Professionals polling, who engaged this road section operation, come to nothing. As things go, it was decided to conduct a smoothness research of road topping length of 72,0 m (km 63 + 098 – km 63 + 170), the overall look and scheme of what is shown in Fig. 1, 2.

The purpose of research – longitudinal profile of trafficway research road section H-18 analysis and road topping deterioration causes revealing.

Figure 1 – General view of road section H-18
Ivano-Frankivsk – Buchach – Ternopil (km 63 + 098 – km 63 + 170)

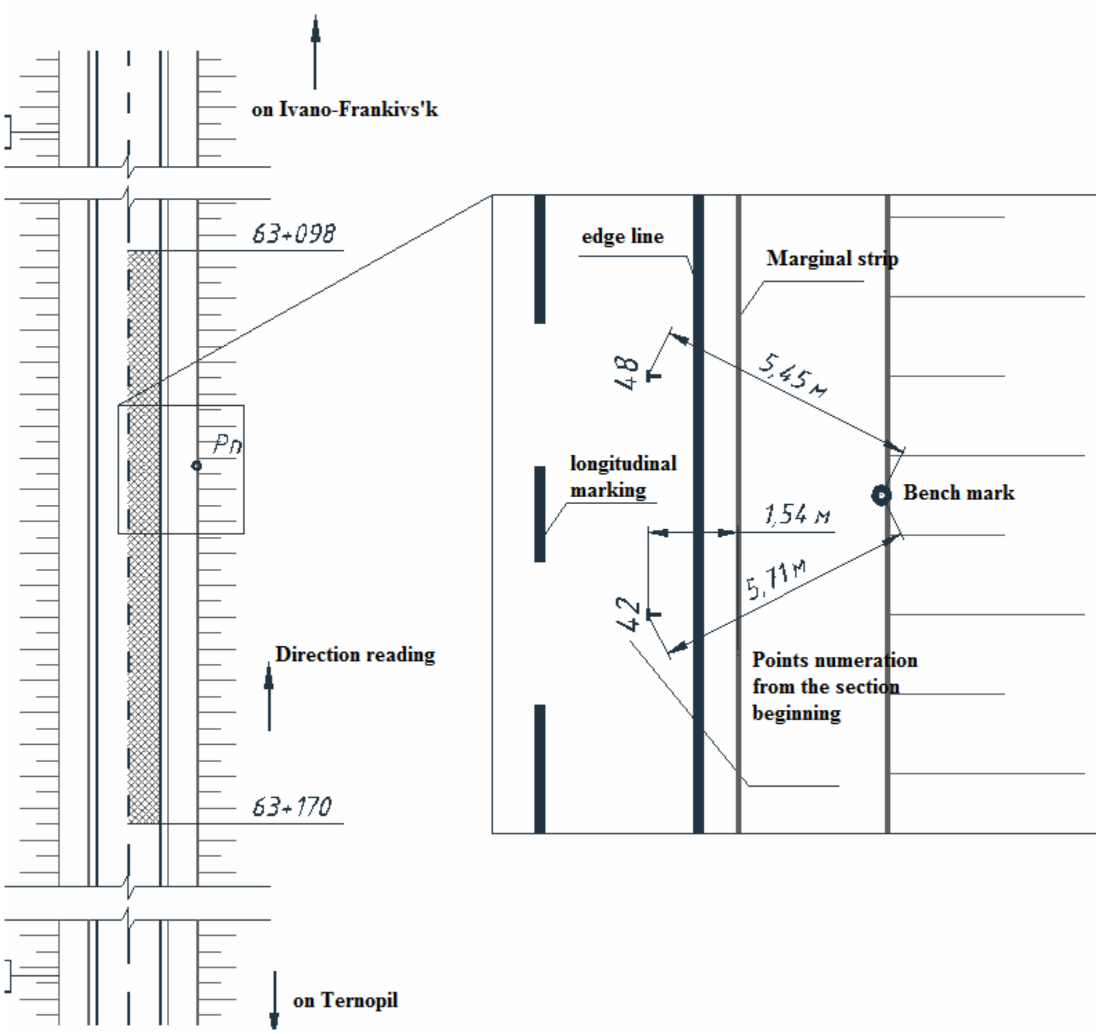

Figure 2 – Scheme of road section H-18 Ivano-Frankivsk – Buchach – Ternopil (km 63+098 – km 63+170)

The main material and research results. The article deals with the problem of smoothness changes by the example of road section H-18 Ivano-Frankivsk – Buchach – Ternopil (km 63+098 – km 63+170), which is connected with irregularities in the procedure of road construction layers.

In April 2014 by means of leveling «Sokkia B20» and three-meter leveling rod RDU «Condor» it is received the longitudinal profile of road topping. Repeated measurements were made on that year August.

Longitudinal profile detailed assessment of the trafficway at the research section included implementation of following stages:

– bench mark fixation (Fig. 2) and its recovery after repeated researches;

– flagging for measurement at a distance of 0,5 – 1,0 m from the marginal strip by using measuring tape and chalk and planimetric position georeferencing to the bench mark;

– route leveling with survey increment of 0,5 m and high-altitude position georeferencing to the bench mark.

As a result of leveling and measuring the gap at RDU it is perform the longitudinal profile section determining.

Longitudinal profile leveling results is shown on Fig. 3, where you can see the characteristic hollows, what, according to the authors, causes to blows. On profile reduction points it is continuous transverse cracks (Fig. 4).

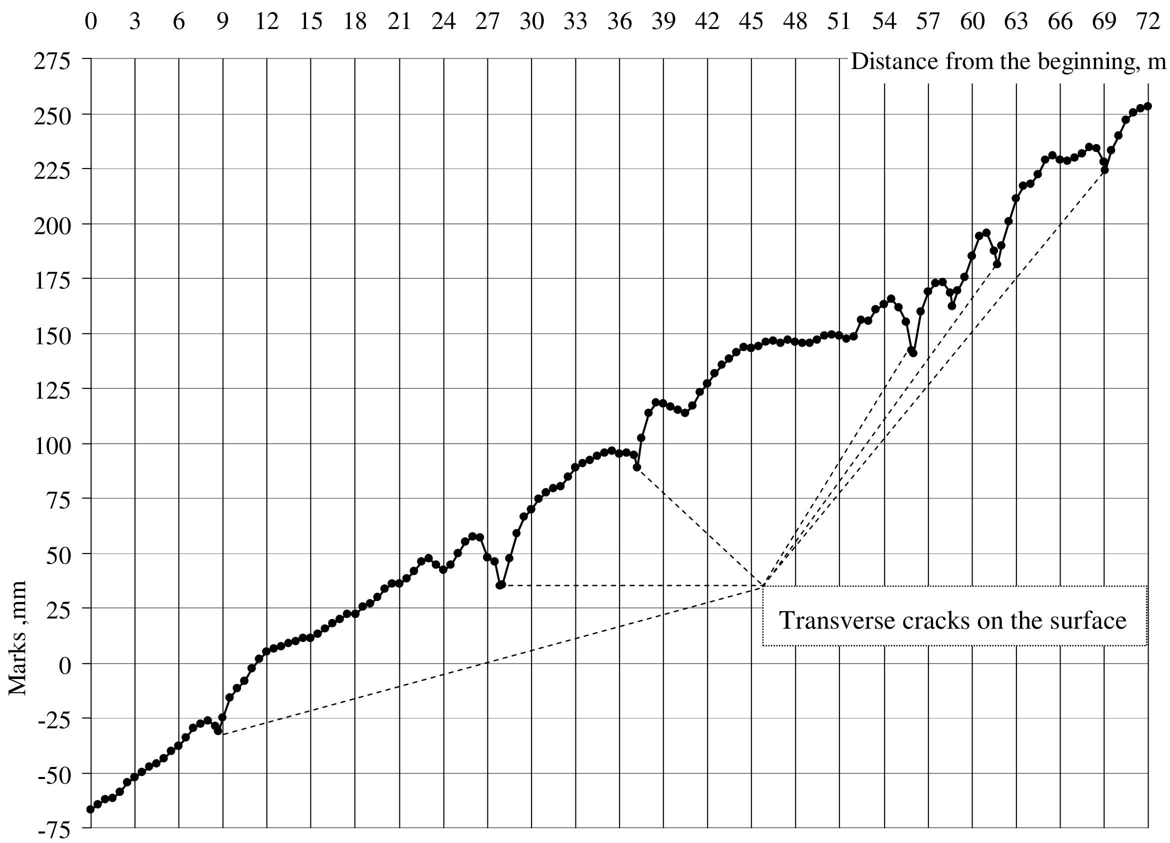

Figure 3 – Longitudinal profile of the right trafficway track

Figure 4 – Continuous transverse temperature crack at a characteristic section

In order to explain the road topping irregularities causes and cracking it is taken the samples in certain places – on crack (km 63+114,10) and in a place with no visible defects (km 63+105,00) (Fig. 5).

Comparison of road-mat cores taken without defects and at place of transverse crack passing through is shown on Fig. 6.

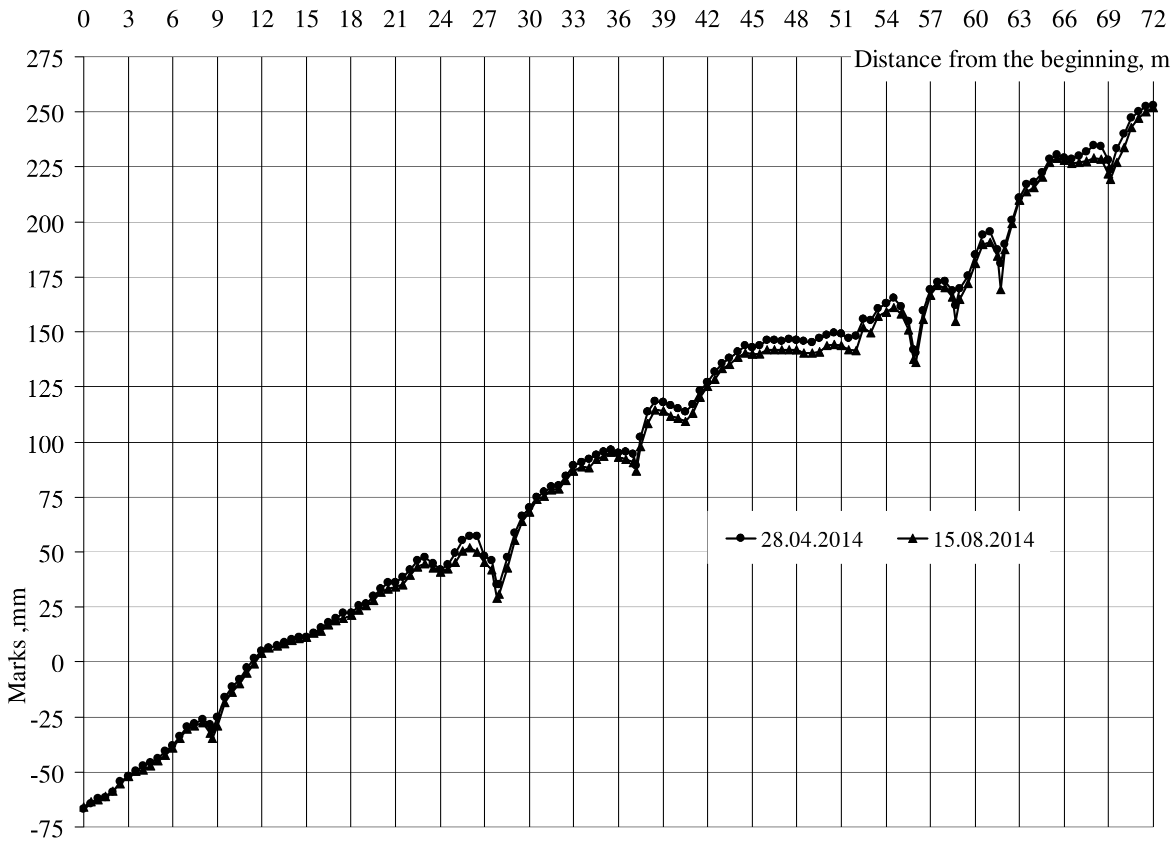

Figure 5 – Location of core taking points in the longitudinal profile section

Figure 6 – Road-mat cores

As shown on Fig. 6, left road-mat core that taken on the road section without defects (km 63+105,00) has the upper layer of fine graded bituminous concrete with thickness of 4,5 cm, lower layer – coarse bituminous concrete with thickness of 7 cm. Right core, taken on the road section in a place of transverse crack passing through (km 63+114,10) has a thickness layers respectively 3 and 4 cm.

Situation's arisen, can be represented schematically on Fig. 7.

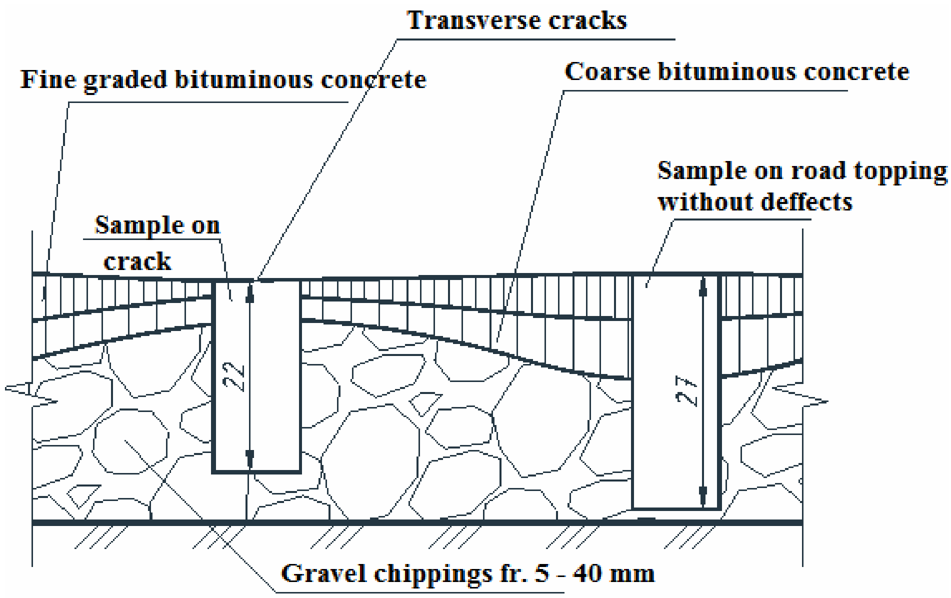

Figure 7 – Schematic representation of longitudinal profile road-mat cores

Longitudinal profile hollows and transverse cracks caused by layers uneven thickness along the road length. In the places of reduced layers thickness it is a weakening of the road-mat profile cross-section, so when the environment temperature is reducing in these places there was a gap that led to the cracks formation.

Conclusions. Trafficway longitudinal profile detailed analysis of the experimental highway section H-18 (km 63+098 – km 63+170) and the samples content, selected in specific places suggests that the regular irregularities cause of the road topping and transverse themperature cracks may be the layers uneven road-mat construction thickness, that is primarily connected with irregularities in the procedure of its structure.

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Received 25.04.2016

UDK 625.7/.8

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THE PROSPECTS MANUFACTURE OF RECYCLED HOT MIX ASPHALT WITH FIBER PLASTIC REINFORCEMENT

The most common ways of milled asphalt re-use, what is formed during road topping repair is considered. The results of experimental studies of preparation technology features and physical and mechanical properties of recycled hot mix asphalt determination based on milled asphalt with plastic fiber addition that obtained from industrial waste is presented. Defined regulations correspondence obtained recycled hot mix asphalt and ways of their use in road construction is determined.

Keywords: *asphalt pavement, hot recycled, milled asphalt, plastic fiber, recycled hot mix asphalt (RHMA), reclaimed asphalt pavement (RAP).*

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ПЕРСПЕКТИВИ ВИГОТОВЛЕННЯ ГАРЯЧИХ РЕЦИКЛЬОВАНИХ АСФАЛЬТОБЕТОННИХ СУМІШЕЙ З АРМУВАННЯМ ПЛАСТИКОВОЮ ФІБРОЮ

Розглянуто найбільш поширені способи повторного застосування фрезованого асфальтобетону, який утворюється під час ремонту дорожнього покриття. Наведено результати експериментальних досліджень щодо особливостей технології приготування та визначення фізико-механічних властивостей гарячих рециклюваних асфальтобетонних сумішей на основі фрезованого асфальтобетону з додаванням пластикової фібри, що отримана з побутових відходів. Визначено відповідність нормативним вимогам отриманих гарячих рециклюваних асфальтобетонних сумішей та шляхи їх застосування в дорожньому будівництві.

Ключові слова*: асфальтобетонний матеріал, гаряче рециклювання, фрезований асфальтобетон, пластикова фібра, гаряча рециклювана асфальтобетонна суміш, відновлений асфальтобетонний матеріал.*

Introduction. Asphalt – the most common material in our country for the road topping upper layers design in the permanent road-mat construction. Due to limited funding almost 90% of public roads for the last thirty years is not repaired, hereupon they do not meet modern requirements both on strength (39,2%) and smoothness (51,1%) [1].

Asphalt pavement recovery usually involves placing over the old pavement design a new asphalt pavement layer. However, such measures give only short-term effect, since a few years on the traffic-bearing surface appear old deformations and destructions. A more effective way to restore road-mat operability is to replace defective and damaged layers of all pavement design using recycling process – old asphalt reusing.

Analysis of recent sources of research and publications. The issue of old asphalt reusing our scientists and process man began to engage from the 40-s of the last century, as evidenced by the work [2 – 4]. However, the absence at that time the relevant technologies and techniques not allowed to bring recycling processes at the practical level use in highway engineering.

The impact for old asphalt reusing has become a global energy crisis of the 70-s, that led to the search of replacement the deficient at that time organic binder for the asphalt mixtures preparation [5 – 6].

Today in highway engineering is spreading the following ways to old asphalt reuse that is usually formed as a result of milling (cutting by pavement profiler) or road-mat layers removing with further crushing and sorting [7 – 10]:

– roadside verge strengthening and subgrade slopes;
– bed course and basecourse installation;
– racked-in chipping;
– cold organic and hydraulic mixtures production;
– hot mix asphalt preparation.

The choice of one or another method of old asphalt reusing depends on the technical, environmental and economic factors [11 – 12].

Identification of general problem parts unsolved before. The most rational way it is possible to consider the old asphalt using at the hot recycling technology – hot mix asphalt preparation with partial or full recycled material content. For example, in the most European countries, upon condition technological requirements observation it is allowed to add up to 10% of milled asphalt to the new hot asphalt mixtures, intended for the upper layers; 30 – 50% – for the road-mat lower layers; till 100% – base course [7 – 10].

Since the modern domestic road practice, compared with Europe and America, the issue of milled asphalt reusing is not become sufficiently spreading, so it was decided to conduct the research of recycled hot mix asphalt preparation reasonability based on milled asphalt with plastic fiber reinforcement, obtained from postconsumer plastics.

Formulation of the problem. The aim of research – to study physical and mechanical hot mix asphalt properties based on milled asphalt with plastic fiber reinforcement.

Basic material and results. During experimental studies in the laboratory conditions based on milled asphalt (see Fig. 1, a) with plastic fibers addition, obtained from postconsumer plastics, size 25×3 mm (see. Fig. 1, b), three research samples series of recycled hot mix asphalt following composition is produced:

– seria A1 – milled asphalt without additives (control range);
– seria A2 – milled asphalt with plastic fibers addition in an amount of 0,75% by the basic material weight;
– seria A3 – milled asphalt with plastic fibers addition in an amount of 1,5% by the basic material weight.

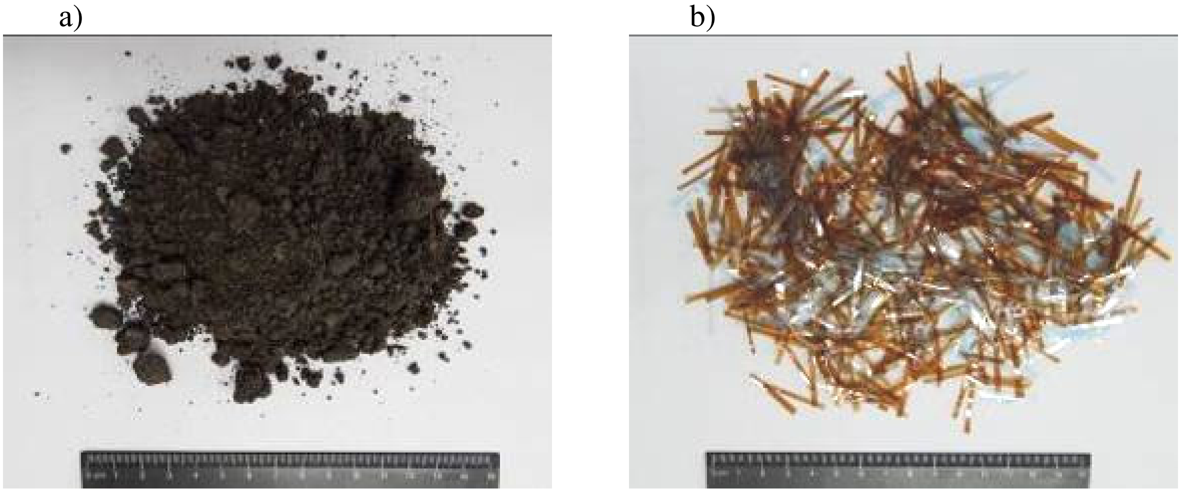

Figure 1 – General view of initial materials:
a – milled asphalt; б – plastic fiber

Milled asphalt grain fineness, determined by sifting through a standard sieve with holes from 40 to 0,071 mm, the most meet regulatory requirements [15] to hot asphalt mixtures type D (Fig. 2).

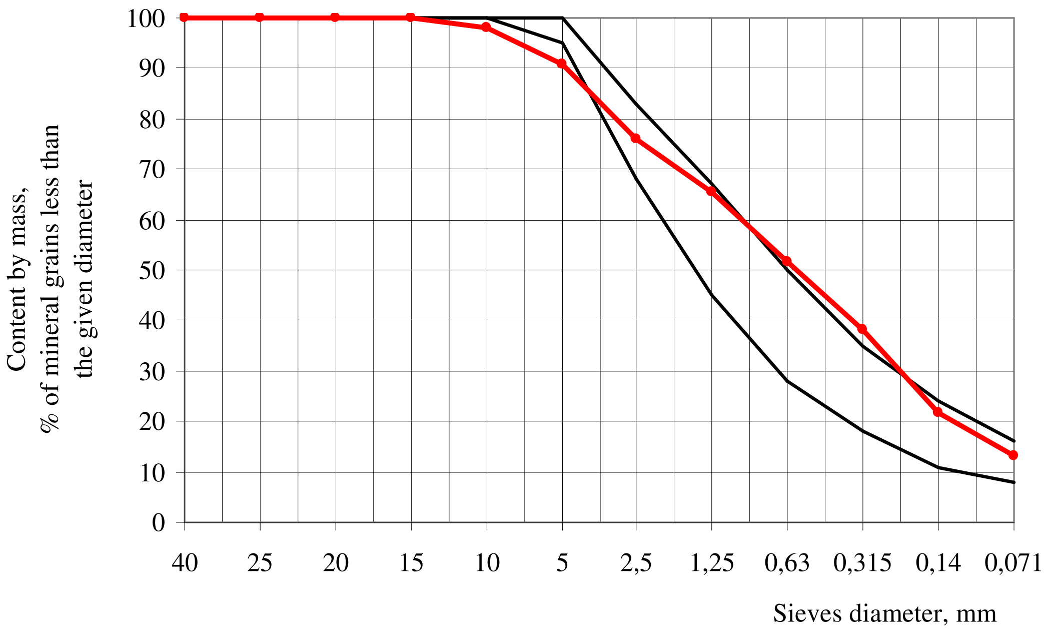

Figure 2 – Comparison of milled asphalt grain fineness with regulatory requirements for hot asphalt mixtures type D

Preparation and testing of recycled hot mix asphalt research samples based on milled asphalt is carried by a standard procedure for the average material density determination, water saturation, swelling and compression strength limit in accordance with requirements [13 – 15]. The results of physical and mechanical properties determination of test samples are shown in Table 1 and Fig. 3 – 6.

Table 1 – Physical and mechanical properties of test samples

Name of properties	Samples seria		
	A1	A2	A3
Average density, g/cm^3	2,30	2,28	2,36
Water saturation, %	3,12	3,11	3,31
Swelling, %	0,34	0,32	0,50
Compression strength limit, MPa – in the dry condition at the temperature: – 20^{o}C – 50^{o}C – in the moisten condition	 5,70 6,13 4,63	 6,09 6,59 4,39	 6,50 5,75 4,32

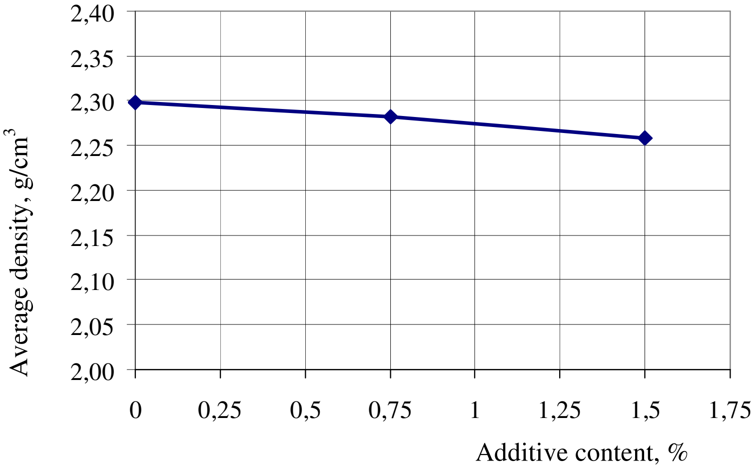

Figure 3 – Plot of the research samples average density from the presence and fiber content supplements

Analysis of the research samples average density dependence from the presence and plastic fiber content supplements (see Fig. 3) shows that this relationship is more or less linear character, i.e with plastic fibers content increasing in the mixture the average samples density is reduced by reducing the particle of stone material (recycled mix asphalt average density from pure milled asphalt is $\rho_{m\ aver}$ = 2,30 g/cm^3) and a lighter plastic fibers gradual increase.

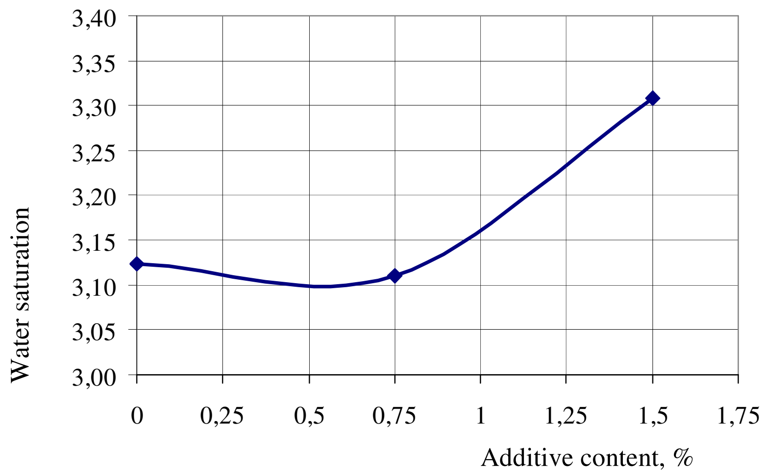

Figure 4 – Plot dependence of water saturation indicator of research samples from the presence and plastic fiber content supplements

Analysis of the water saturation dependence of reserch samples from the presence and plastic fiber content supplements (see Fig. 4) shows that with recycled mix asphalt content implementation of plastic fibers water saturation at first decreases (the recycled mix asphalt water saturation from pure milled asphalt is W_{aver} = 3,12 %), but with fiber content increasing, this indicator starts to increase due to the porosity increasing.

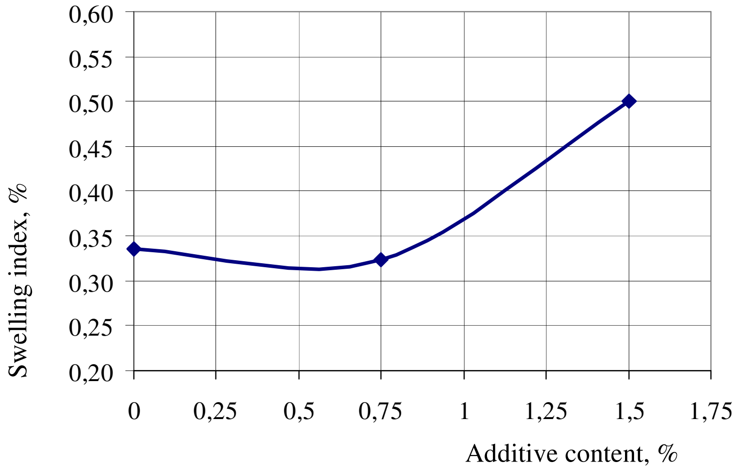

Figure 5 – Plot dependence of research samples swelling from the presence and plastic fiber content supplements

Analysis of the research samples swelling dependence from the presence and plastic fiber content supplements (see Fig. 5) shows that with recycled mix asphalt content implementation of plastic fibers swelling index at first decreases (the recycled mix asphalt water saturation from pure milled asphalt is H_{aver} = 0,34 %), but with fiber content increasing, this indicator starts to increase due to the porosity increasing.

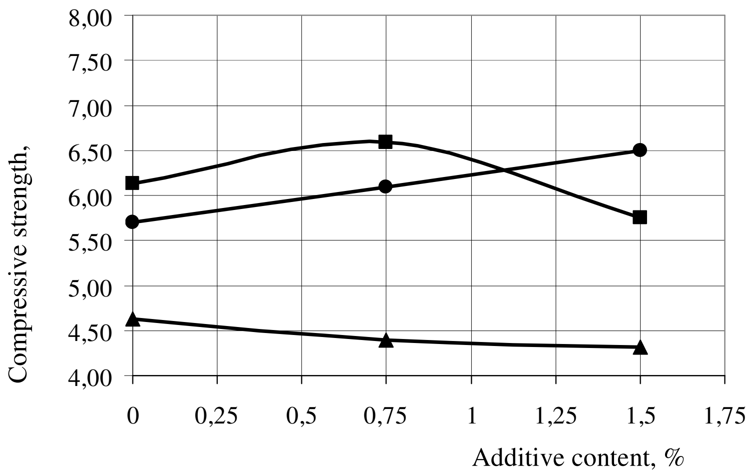

Figure 6 – Plot dependence of test samples compressive strength from the presence and fiber content supplements:

round markers – in the dry condition at T = 20°C;

square markers – in the dry condition at T = 50°C;

triangular markers – in the moisten condition at T = 20°C.

Analysis of the research samples compressive strength dependence from the presence and plastic fiber content supplements (see Fig. 6) shows that at the sample temperature T = 20°C in the dry condition the strength value R_{20} increased with the implementation of plastic fibers to the recycled mix asphalt content by the origination effect of milled asphalt reinforcing. However, with temperature samples increasing to T = 50°C in the dry condition strength value R_{50} at first increases (with fiber content of 0,75%) due to the partial reinforcement effect of milled asphalt, and then strength value R_{50} decreases (with 1,5% of fiber content) due to the loss of milled asphalt reinforcing effect. At the sample temperature T = 20°C in the moisten condition the strength value R_{20}^{M} with the implementation of plastic fibers to the recycled mix asphalt content decreases, is illustrative of complete milled asphalt reinforcing effect absence.

Conclusions. The results of physical and mechanical research samples properties determining of recycled hot mix asphalt based on milled asphalt with plastic fibers addition indicate this research area prospects, because they allow not only to obtain the economic effect from the cost reducing of new road-building materials acquiring, but also to improve the environmental situation through the use of postconsumer plastics.

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Received 16.02.2017

UDC 625.88

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STUDIES OF COATINGS FROM FIGURED OF THE ELEMENTS OF PAVING (FEP) WITH CORRUGATED BASE FROM TOOTHED ELEMENTS OF PYRAMIDAL SHAPE ON THE HORIZONTAL AND INCLINED SURFACES

The new constructive-technological solutions of coatings from FEP (figured of the elements of paving) with the modified geometric by shape of the base were developed. Base FEP consisted of one, five or nine toothed of the elements of pyramidal shape was corrugated. The plan of the experiment and conducted laboratory studies on the effect geometric form of the base of a single FEP on the qualitative characteristics of coatings, located on the horizontal and inclined surfaces under the influence of a longitudinally applied load, was compiled. The experiment results analysis showed that the coatings from FEP with five and nine of the toothed elements in the base prevents the shift more effectively than the traditional coatings.

***Keywords**: figured element of paving – FEP, coatings, a pyramidal shape, a longitudinal shift.*

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ДОСЛІДЖЕННЯ ПОКРИТТІВ ІЗ ФІГУРНИХ ЕЛЕМЕНТІВ МОСТІННЯ (ФЕМ) З РИФЛЕНОЮ ОСНОВОЮ ІЗ ЗУБЧАСТИХ ЕЛЕМЕНТІВ ПІРАМІДАЛЬНОЇ ФОРМИ НА ГОРИЗОНТАЛЬНИХ І ПОХИЛИХ ПОВЕРХНЯХ

Розроблено нові конструктивно-технологічні рішення покриттів з фігурних елементів мостіння (ФЕМ) зі зміненою геометричною формою основи. Зясовано, що рифлена основа таких ФЕМ складається з одного, п'яти або дев'яти зубчастих елементів пірамідальної форми. Складено план експерименту та проведено лабораторні дослідження щодо впливу геометричної форми основи одиночного ФЕМ на якісні характеристики покриттів, розташованих на горизонтальних та похилих поверхнях під дією поздовжньо прикладеного навантаження. Проаналізовано результати експерименту показав, що покриття з ФЕМ з п'ятьма й дев'ятьма зубчастими елементами в основі найефективніше перешкоджають зсуву, ніж традиційні покриття.

***Ключові слова**: фігурний елемент мостіння (ФЕМ), покриття, пірамідальна форма, поздовжній зсув.*

Introduction. The unit of the coatings of the sidewalks, paths, areas for various purposes from figured paving elements has become actual recently [1 – 4]. Due to the fact that such coatings are environmentally cleaner comparativly to asphalt coating and are aesthetically attractive. Due to the gaps between the tiles provided by the outflow of the water from the surface during roll of the rainfall, their service life is prolonged. Such coatings, if necessary, can be easily dismantled, for example, for lying of the underground communications, or easily be replaced with individual deformation of elements. However, in the course of their operation, sometimes some defects can be observed, including loosening and cracking in the individual elements, potholes and dips in the coatings. Such violations occur due to incorrectly selected structural-technological solutions, non-compliance with the rules of operation and the coating unit technology. The problem solution is development of new variants of coating design from FEP with the modified geometric parameters [5].

Analysis of recent research sources and publications. The FEP applied to the unit coatings has different sizes and shapes [6-10]. Their base is commony flat. In the published work [11] constructive-technological-solution of coating from FEP with a pyramidal base (Fig. 1) are proposed. To exclude the cost of concrete for base, volumes should be equal for the FEP with a pyramidal and a flat by base, when they are the same size and shape in plan. This is achieved by reducing the height of the prismatic part of FEP with a pyramidal base. Such coatings are tested in laboratory and field conditions, and was performed a comparative analysis of the research results [12]. Determined, that the coverage of the FEP with by pyramidal base have several advantages compared to traditional coatings.

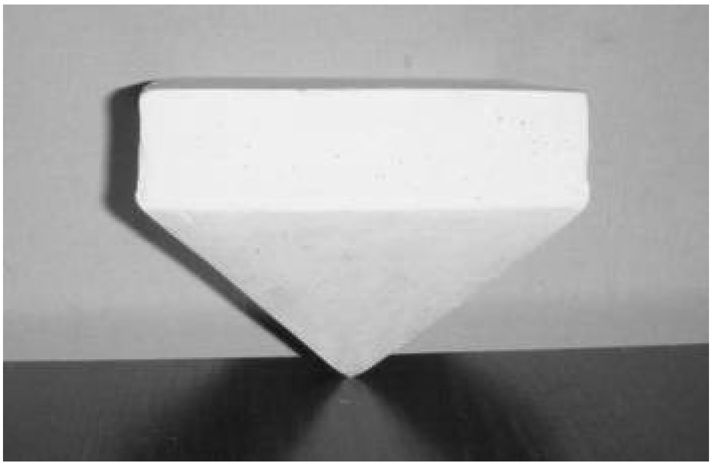

Figure 1 – Model of FEP with a pyramidal base with an angle at the vertex of the pyramid 90^0, made of gypsum

Sediment single element is reduced by half from FEP with the angle at the vertex of the pyramid 70^0 and 1,3 times the FEP with the angle at the vertex of the pyramid 125^0 compared to FEP with a flat base (180^0 at the top of the pyramid). Determined, that the bearing capacity of coating is increases by reducing the angle at the vertex of the pyramid. The study of deformation zones of the constructive sand layer [13] has demonstrated the feasibility of using the developed technological solutions of coatings from FEP with a pyramidal base.

Selection of the unsolved parts of problem. The disadvantage of coating from FEP with a pyramidal base is the complexity of the unit the elements with an acute angle at the vertex of the pyramid. If we take the traditional tile square shape in plan with a side of 120 mm and a height of a prismatic part 50 mm, the height of the prismatic part of the tiles with a pyramidal base with an angle at the vertex of the pyramid 90^0 is equal to 30 mm for the same size and shape in the plan. With a significant reduction of the height of the prismatic part of the tiles with by pyramidal base, her contour part in contrast to the central part is becomes more vulnerable. The impact of a large load on a contour part can lead to the destruction of the edges of such tiles.

For solving problems, related to improving the bearing capacity of the coatings and to simplicity its of the unit, were developed new variants of coatings from FEP, having the corrugated base from the toothed elements of pyramidal shape [5]. The study of the qualitative characteristics of such coatings on horizontal and inclined surfaces was not performed, therefore it is highly relevant is the conduct of such studies.

The purpose of work – to study the impact of the modified geometric shape of the FEP on a shear of the coatings, located on the horizontal and inclined surfaces, when exposed to the longitudinal load, and the determination of the optimal of options of the coatings.

The main material and results. A new constructive-technological solution of coating from the FEP with a corrugated base from the toothed elements of a pyramidal shape is developed. In this case the FEP should have the shape of a regular polygon in plan. The three options such FEP square shape in plan are offered. In the first option the base of the tile comprises nine of toothed elements of pyramidal shape (the three pyramidal elements, located in three rows). In Fig. 2 this is the first sample on the left.

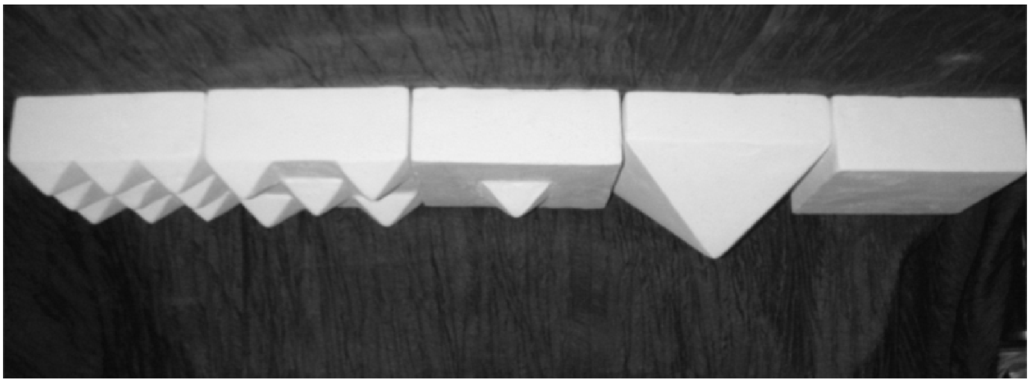

Figure 2 – Samples of the FEP from gypsum, square shape in plan with a side of a square is 120 mm, with different geometric shape of the base and height of the prismatic part

In the second embodiment, the corrugated base is composed of five toothed elements of pyramidal shape (one located at the center and the other four at the corners of the tile). In Fig. 2 is the second sample from the left. The third embodiment – is tile with one toothed element of pyramidal shape, positioned centrally. In Fig. 2 is the central sample. The fourth sample on the left has the base a pyramidal shape with the angle at the vertex of the pyramid 90^0, and the fifth on the left, with a flat base. In our opinion a constructive solution, which was developed, has several advantages. One of them is providing hard pinched of pyramidal elements of each FEP of the coating in the underlying structural layer, therefore, its displacement will tend to minimum. By changing the geometric shape of base of the FEP is receive an additional seal of the underlying layer under the tile. An increase of the area of bearing the base of the FEP on the underlying structural layer leads to transfer the load across larger volume of the structural layer.It leads to the increase of the bearing capacity of the coating from the FEP with corrugated base.

To determine the qualitative characteristics of the proposed coverage from the FEP plan and methodology for conducting the experiment were developed, the materials and equipment were selected. The purpose of the experiment - to determine the value of the longitudinal load applied to the sample, considering longitudinal shift of the sample on 1 mm on the horizontal and inclined surfaces. Parameter longitudinal load acting on the sample with shift gives us the opportunity to evaluate the work of the coating of tiles with various geometric shape of the base. The tile having the greatest option of the load for fixed shear 1 mm has a greater shear resistance. Coverage of such tiles is committed to maintain its original position.

In order not to get increasing of material costs for toothed elements, it is necessary to reduce the height of the prismatic part of the tiles, so that the volumes of all specimens were equal. By prototype traditional tile of square shape in plan is selected with a side of a square 120 mm and 50 mm high. For test specimens with the same configuration in plan with the angle at the vertex pyramids 90^0 the height of the prismatic part was as follows: the FEP with nine of toothed elements 43,3 mm; with five of toothed elements 46,3 mm; with by one toothed element is 49,3 mm. It should be noted, that reducing of the height of the prismatic part of the FEP with corrugated base slightly compared by height of the prismatic part of traditional tile. However, the difference between the height of the prismatic part of the FEP with a pyramidal base and a traditional FEP with a flat base (with a similar size in plan) is about 20 mm.

Pilot-plant stand was made of metal pan with sandy structural layer (its thickness is 160 mm (Fig. 3).

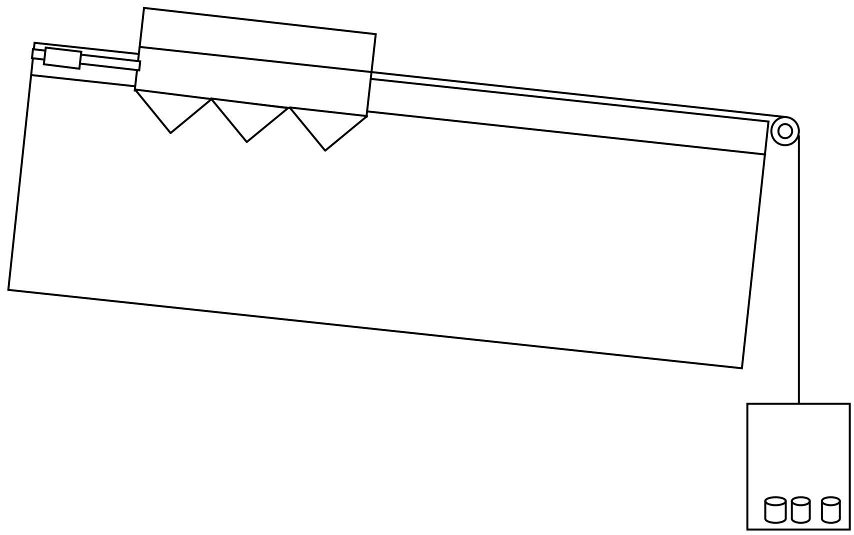

Figure 3 – Scheme of experimental-production bench

The block is installed in the right top side of the tray from the outside. End-to-end to the left end of the tray above the sandy layer was equipped with two indicator of gages, with graduation 0,01 mm (model IG 10 MN) with accuracy class 1, designed to measure the linear dimensions of both absolute and relative methods, definitions of deviation from the desired geometric shape and arrangement of objects (tray and sample). According to the indicators tracked the shift of samples under the influence of the horizontally applied load, and to the right, next to the indicators, which were in a fixed state, was mounted the test sample on a sand base. After mounting the sample in the design position initial performance of indicators were obsered. For the experiments the samples were manufactured from gypsum with the above parameters. Given the different density, the same samples of concrete on 0,98 kg is heavier than gypsum. Thus, during tests the gypsum samples were uploaded on this value.

To the experimental sample the cable was attached to transfer the load. The cable (depending on tasks) was located between the sample and the block, either horizontally or with a slope, and behind the block – vertically downwards. One of the biggest factors, that affecting the operation of coatings is the slope of the surface. Maximum allowable slope when the unit of pavements coatings are equal to 0,08, so the studies were carried out with the minimum, maximum and intermediate of slope values. On a horizontal surface, the slope is equal to zero, therefore, the intermediate slope is equal to 0,04.

To the lower end of the cable hanging the container on which incrementally was added the weight in steps of 0,01 kg. When the indicators showed values of the shift 1 mm, the cable with hanging on it loaded container was removed and weighed on an electronic Libra. The value of the horizontal load was measured. For all experiments the density of the sand layer before installation of the specimens was equal 1.65 g/cm^3 at humidity of sand 5 %. The results of the experiments are presented in the table (Tab. 1).

Table 1 – The longitudinal load applied to the coating of single FEM flat and corrugated shape of the base of the longitudinal shift element 1 mm

Number of experiment	Form of the base of FEP	The slope of the surface	The value of a longitudinally applied load, g
1	Flat	0	990
2	corrugated with one toothed element	0	1360
3	corrugated with five toothed elements	0	1950
4	corrugated with nine toothed elements	0	1720
5	Flat	0,04	690
6	corrugated with one toothed element	0,04	1340
7	corrugated with five toothed elements	0,04	1420
8	corrugated with nine toothed elements	0,04	1560
9	Flat	0,08	680
10	corrugated with one toothed element	0,08	1030
11	corrugated with five toothed elements	0,08	1180
12	corrugated with nine toothed elements	0,08	1200

The experimental results indicated that the tile with nine teeth, located on a horizontal surface, the resistance a horizontal load (load value at a fixed shift) is less than the tiles with five teeth, ceteris paribus. But compared to a tile with a flat base and tile with one toothed element in the base, the resistance at the horizontal load is higher. It is determined that the resistance at the horizontal load is higher in tile with five toothed elements in 49,2 % comparativly to tile with flat base, and in 30,3 % comparativly to tile with one toothed element. In the case, when the tile has five and nine toothed elements the resistance at the horizontal load should be larger for tiles with large number of teeth at the base. However, the experiments showed that the horizontal load which is perceived by the tile with five toothed elements, is in 11,8 % higher, than in the tile with nine toothed elements. This phenomenon can be explained by the fact that the zone of deformation under by the projecting parts of tile with nine teeth in the base overlap each other within the boundaries area of tile, and only the contour part of the base of tiles resists to horizontal load. Sand is under the base of tiles, between the teeth works, as a unit, and displacement of tile depends on the coefficient of adhesion of sand particles between them. In the case of tile with five toothed elements, partially the side faces of the pyramidal elements and the plane of the base tiles work.

If compare the results of the experiment on inclined surfaces, the resistance to the horizontal load is not much higher off the tiles with nine of toothed elements, than for tiles with five (9 % with a slope of 0,04 and 1,7 % with a slope of 0,08). This can be explained by the fact, that the perception of external influences by coatings located on sloping surfaces, greater volume of sand off FEP with the nine toothed elements in base is actevated, than the tiles with five teeth. If we compare the obtained parameters when the slope of the surface equal to 0,04 we see that the resistance to horizontal load in the tiles with the nine gear elements is 55,8 % is higher, than that of the tile with a flat base, and in 14,1 % higher than that of tiles with a single toothed element. If the surface slope is equal to 0,08 resistance to horizontal load in the tiles with nine toothed elements is higher in 43,3 % than in tiles with flat base, and in 14,2 % higher than in tiles with 6 single toothed element. On the surface with a slope of 0,08 resistance to horizontal load is higher by 29,1 % for tiles with one toothed element, than tiles with a flat base, and for coating with a slope of 0,04 – 41,7 %.

At increasing of the area of the FEP base large amounts of an underlying layer are utilized. To obtain the best option coating, the area of the protruding elements of corrugated base should be calculated so that deformation zones under them do not overlap [14, 15].

Research of the coatings from FEP with a modified geometric shape of the base under the influence of longitudinally applied load, have proved their advantages in comparison with traditional coatings. However, during operation, on the coating not only horizontal but also other types of loads (vertical, combined, etc.) are afected. In order to recommend in the construction industry the FEP coverage with base modified geometric shape, it is necessary more widely to study the characteristics of these coatings located on of horizontal and inclined surfaces, that requires further laboratory and field studies.

Conclusions:

1. Coatings with base corrugated form of single FEP, located on horizontal and inclined surfaces under the influence of longitudinal load, have been studied.
2. Values of the longitudinal load is the parameter which icharacterizes the ability to prevent the FEP shift are determinate.
3. It was established FEP with a modified geometric shape of the base more efficiently prevents the shift of coverage than the traditional FEP.
4. It was established experimentally that on the horizontal surfaces are more efficient a coating of the FEP with a corrugated base of the five toothed elements, and on the inclined surfaces – of the nine.
5. In order to recommend the FEP coatings with corrugated base in the construction industry it is also necessary to study the effect of other types of acting loads.

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Received 28.08.2016

UDC 621.926:622.7

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TECHNIQUE AND TECHOLOGY OF RARE-METAL ORES DESINTEGRATION AND GRAVITY-BASED BENEFICATION

On the base of the analysis of centrifugal concentrators designs there has been justified the selection of apparatus for the rare-metal ores benefication. Process of pyrochlore ore grinding in mills of different types has been investigated and the expediency of use of impact-centrifugal action mills to ensure selectivity of minirals disclosure has been founded. Efficient technical and technological parameters of the centrifugal action mills have been justified on the base of active experimental method. The influence of disintegration metheds on technologival indication of rare-metal ore benefication of Mazurovske deposits in centrifugal Nelson concentrator has been analyzed.

Keywords: centrifugal concentrator, ore preparation, gravity-based beneficiation in centrifugal field, disintegration, a mill of impact-centrifugal action type.

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ТЕХНІКА ТА ТЕХНОЛОГІЯ ДЕЗІНТЕГРАЦІЇ Й ГРАВІТАЦІЙНОГО ЗБАГАЧЕННЯ РІДКІСНОМЕТАЛІЧНИХ РУД

На основі аналізу конструкцій відцентрових концентраторів обґрунтовано вибір апарата для збагачення рідкіснометалічних руд. Виконано дослідження процесу подрібнення пірохлорової руди в млинах різного типу та встановлено доцільність застосування млинів ударно-відцентової дії для забезпечення селективності розкриття мінералів. Методом активного експерименту обґрунтовано раціональні технічні та технологічні параметри млина ударно-відцентрової дії. Проаналізовано вплив способів дезінтеграції на технологічні показники збагачення рідкіснометалевої руди Мазурівського родовища у відцентровому концентраторі Нельсона.

Ключові слова: *відцентровий концентратор, рудопідготовка, гравітаційне збагачення у відцентровому полі, дезінтеграція, млин ударно-відцентрового типу.*

Introduction. Development and improvement of technical devices for the enrichment of rare-metal ores is an important issue of mining machinery manufactoring. This is due to specific properties of processed raw materials - rare metals, which are more often poor and finely disseminated. Traditional gravity-based machines and methods of beneficiation of such ores do not provide sufficiently high recovery of metals into rough concentrates. Thus, long-term investigations of rare metals ores enrichment of Mazurovskoe deposit (the only in Ukraine) by various research organizations in order to develop effective technologies of enrichment did not reach positive results. Extracting the most valuable mineral – pyrochlore, – at the best case did not exceed 35÷40 % [1].

The general trend of involving the poor rare metal raw materials into processing and reducing the size of valuable components grains in it need to solve the problem of processing of finely disseminated ores and encouraging of dump products of enrichment, accumulated over a long period of processing plants operation.

Low in pyrochlores recovery values independently from methods of enrichment are due to dissemination of niobium ores, and related to it necessity of grinding them to a particle size 0,071 – 0,044 mm. Due to the high fragility of pyrochlore and its lower strength, in comparison with other minerals of the ore, it is taking place its regrinding and formation of oily spot on the surface of rock constituents, inclusions of pyroclore on the high layer of albite, microcline, nepheline grains. Surface properties of these minerals are changed, contrast of their properties is redused and, as a consequence, the efficiency of enrichment.

Improvement of ore preparation by investigation of the known methods of selective disintegration, selection and justification of equipment and technologies that provide the least regrinding of pyrochlore, and high indicators of its extraction in the conditions of gravitational enrichment, is a relevant task.

Analysis of recent research sources and publications. In terms of broad involving poor disseminated rare metal rau materials in to processing, the main direction of development of gravity-based means of separation has become the theoretical substantiation and development of technologies and devices for the enrichment of fine-grained and fine-dispersed materials.

The gravity-base method is used to enrich the ore of a sufficiently wide range of grain size from 500-600 mm (heavy-medium separation) to 0,005 mm (in centrifugal concentrators). This method is environmentally friendly and in many cases the least expensive. Principles of gravity-based separation is widely used both in the direct enrichment of various ores, and in ores preparation for products classification and dehydration.

There has been created a scientific base of gravity concentration [2 – 3]. The efforts of researchers and designers has always been focused on two main and interrelated issues: improving the accuracy of ore components separation and improving processing equipment productivity.

At first this problem was solved by reducing the thickness of the slurry flow in the devices of gravity concentration, and decrease in flow rate of the pulp. There have been proposed gateways of small filling with different kinds of trapping coatings, which were operated successfully. There appeared slurry concentration tables with shallow ariplane and without them (Hollman), multideck slurry concentrators with periodic flushing of concentrate (Moseley), tape slurry concentrators with continuous discharge (Crossbelt), were improved multideck cone separators (Reichert), were widely used screw-gateways [4]. However, while reducing the particle size of enriched material to ensure high quality performance it was always necessary to get significant diminished performance of separators. Attempts to apply high-frequency vibrations of pulp in the apparatus with a thick layer of concentrated particles during isolation and on vibrating chutes were unsuccessful.

Today the greatest prospects in creation of machines and technologies for the enrichment of fine-grained and finely disseminated ores, sands, and anthropogenic materials are associated with the use of centrifugal separators of various types [5], in particular, the pressure and free-flow. The former appeared in the middle of the last century and represented the hydrocyclones with the truncated conical portion, the so-called short-head hydrocyclones. They provided a relatively high degree of concentration of high density valuable components under a small output of concentrate. In devices of such type the movement of slurry flow is turbulent by nature, that definitely leads to intensive intermixing of the material, which is divided, and decreasing accuracy of the gravity separation.

At the end of the twentieth century there were developed the first designs of free-flow centrifugal gravity concentrators, that have given new opportunities in gravity-base concentration [6]. The successful and wide application of these devices in mining and concentrating plants confirmed the high perspectivity of further research and new design developments in this direction.

Centrifugal field in separators or concentrators is created by means of twisting of flow that freely moves in the apparatus, the wall of the bowl that rotates. A necessary condition for centrifugal enrichment of minerals in water environment is the availability of transport (flushing) flows in the direction that does not coincide with the force vector of the centrifugal field. In free-flow centrifugal gravity concentrators, feeding moves along the axis of rotation to the central part of apparatus bowl. Suspension is tightened to forced rotational movement, forming a three-dimensional spiral stream. The material is stratified throughout the trapping surfaces of rotating [7].

The main advantages of centrifugal concentrators in comparison with other devices of gravity concentration are: large specific capacity, high degree of concentration; high recovery of small and fine particles of heavy minerals; the ability of operational management of concentration degree.

Numerous designs of free-flow centrifugal gravity concentrators have different technical and technological parameters. The choice of apparatus design which will provide high rates of separation of ore minerals of a specific type of ore, and justification of machine technical and technological parameters remain non formalized procedure.

Taking into accound that pyrochlore ore, to which Mazurovskoe deposit ore belong to are easily overgrinded and centrifugal concentrators do not provide a high separation ore performance while working with material smaller than 0.02 mm, it is important to choose a mill for grinding.

For a long time the study of the selective grinding of pirochlore ores did not attract appropriate attention. Ores disclosure is carried out by methods which are based on minerals grinding, which are not specifically aimed at the destruction of their planes of cleavage [8]. When the final aim of grinding is to form the possibly new material surface per unit of energy expended, this goal is not only inconsistent with, but contrary to the basic objectives of ore preparation for beneficiation, as it does not conform to the principle «not to split anything extra». This is one of the reasons that the preparation of the ore remains the most energy-intensive, cost and badly managed operation in ore processing technology [9]. With the increasing of specific surface area of the grinded material, it occurs the aggregation of particles, there arise considerable efforts of molecular interactions that begin to exceed the force of separation, that leads to the reduction of the benefication selectivity [10].

The ores selective disintegration in apparatus of different designs was studied in many works [11, 12]. Ores desintegration process in mills of impact-centrifugal type were studied in many works [13, 14]. In the work [15] for the first time there were identified. Some commen factors of rare-metales ore grinding of several foreign deposits. Some conclusions have been done. They are: in these ways of the disintegration the bulk of massive material is being destroyed by the planes of cleavage. Ores and mineral grains destruction character as a result of milling is largely determined by structural features. Even at the same compressive, tensile or shear loads, the stress state and deformation of the ore lumps and separate grains will be difficult. So, under the destruction of the calcite grains by separation in terms of cleavage area on the destruction surface there are formed ledges (steps) on which shear deformation are develop. Under the destruction of polycrystalline ore by stretching, displacement or compression there are always plane shear and detachment [11, 12, 15].

The selection of the general problem parts, which were not resolved before. In the work [16, 17] there were initiated the study of the process of disintegration of rare metals ore of Mazurovskoe deposit. This work is a continuation of previous studies that have not yet evaluated the choice of the rational design of centrifugal concentrator to enrich finely disseminated ores of Mazurovske deposits and to justify the technical and technological parameters, to select grinding mill type and study the influence of grinding ore methods on the technological indicators of enrichment in a centrifugal field.

Statement of the problem. The aim of this work is to justisy the rational design of centrifugal concentrator and study the effect of disintegration methods on the ore benefication index in the separator of centrifugal type, to study the process of selective disintegration of Mazurovskoe niobium ore deposits in grinding apparatus of various types, to justify rational method of ore grinding and technical and technological parameters of the process.

Main material and results. When investigating there were used the basic types of Mazurovskoe ore deposit – *mariupolite*, mineral and chemical composition of which is given in the work [17]. Pyrochlore is the only the actuall niobium mineral of Mazurovskoe deposit, in which the mineral grains are in small amount (0,15-0,17 %) and forms small breeding clusters of small grains in albite, seldom - in lepidomelane and aegrine [18]. Pyrochlore has the form of automorphic isometric small grains (0,005-0,030 mm, sometimes up to 0.05 mm) and crystals of octahedrites shape of reddish-brown or yellowish-brown color, with a greasy luster. Pyrochlore density is 4,2 g/cm^3, the strength according to the Mohs scale – 2...3 [19].

Experimental study of ores grinding and determination of the optimal design of the disintegrator included the study of the kinetics of comminution and disclosure of rare-metals ore in the mills that implement various physical principles of grinding. There has been tested a bullet, rod milling, grinding in the mills of impact-centrifugal type.

The presence of fine pyrochlore ore minerals disseminations in Mazurovskoe deposit ores requires ore grinding to the size of 0,063÷0,044 mm for a more complete disclosure of aggregates. On the other hand, such fine grinding leads to the formation of large amounts of sludge (up to 27 % of the original ore), and it is lost at least 15-20 % of Nb_2O_5. Extraction of valuable components from the sludges is rather problematic. Therefore, the main criterion for the choice of the mill construction was a low level of valuable components losses from the sludge by crushing, and a high degree of disclosure on cleavage contacts of minerals.

In the study of different disintegrants, the primary assessment of ore grinding effectiveness, in accordance with the first criterion (reducing losses with sludge) was carried out by the comparison of the sieve composition of crushed products and distribution of component in size classes. These data are presented in Fig. 1, 2.

From these data it follows that for the main types of Mazurovskoe deposit ore – mariupolite, – for grinding methods in ball and rod mills one can see an increase of content of niobium pentoxide in grinded products in the interval of –0,09 +0,010 mm size (maximum in the interval of –0,02 +0,01 mm). For impact-centrifugal milling method, high content of pentoxide of niobium in the grinded product was observed in the range of –0,056 +0,044 mm (Fig.1).

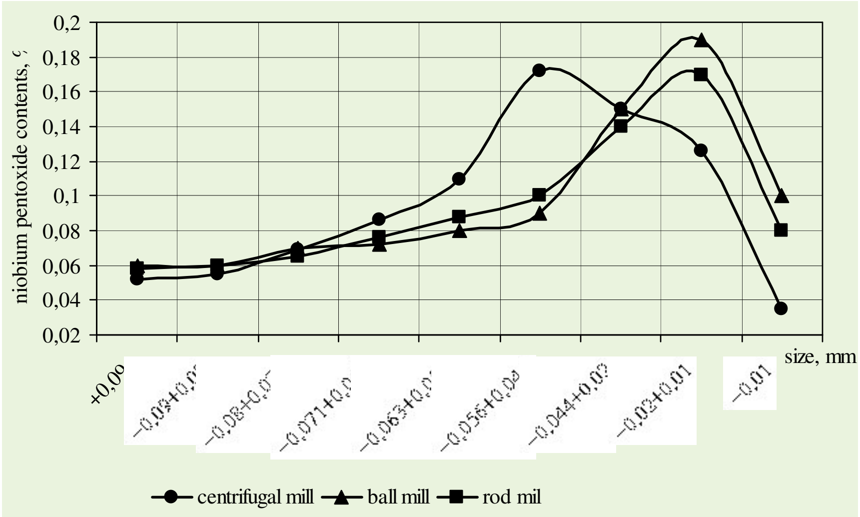


Figure 1 – Dependence of the content of niobium pentoxide from the particle size of crushed product and the method of disintegration

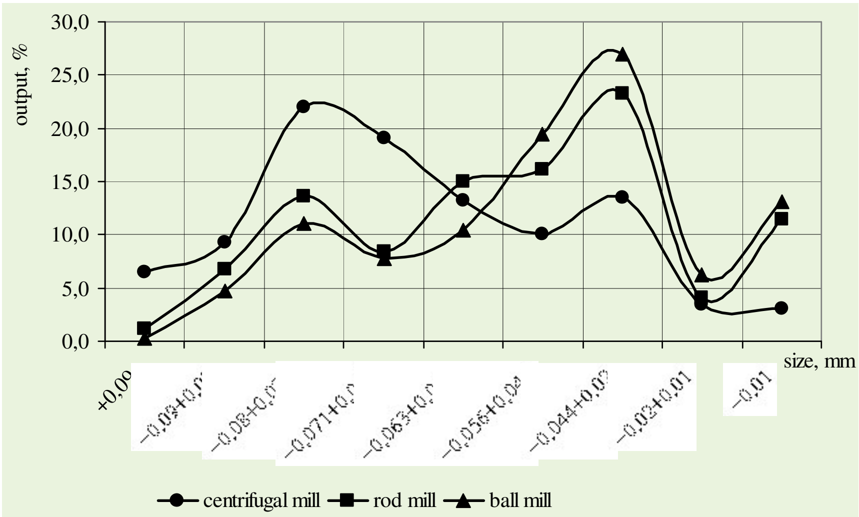


Figure 2 – Dependence of the classes emission from the particle size and the method of disintegration

The maximum output of the crushed material for grinding in a ball and rod, mills have the class from –0,044 to +0,02 mm (Fig. 2). For centrifugal grinding mill the maximum is in the class from –0,08 to +0,071 mm. This is probably due to the different strength of pyrochlore and host rocks.

The analysis of niobium pentoxide distribution on size classes showed that the removing of pentoxide of niobium in sludge (–0,01 mm) is the lowest when grinding in a centrifugal-impact action mill – not higher than 0,03 % (relative), the highest is up to 12 % of niobium pentoxide – when grinding in a ball mill.

The results of the gravitational analysis of the crushed material made by the method [20], showed that unlike with other methods of disintegration, the maximum value of niobium pentoxide extraction in a heavy fraction with a density above 2,8 g/cm^3 is in size of 0,071÷0.050 mm when grinding in a mill of impact centrifugal action. In this type of disintegration niobium pentoxide losses are minimal.

Disclosure data of the investigated variety of ore – mariupolite – during disintegration in apparatus of different designs are given in Table 1 (assessment of disclosure was conducted on non-metallic component). As you can see, the degree of their disclosure after various grinding methods varies widely. Evaluation of the results was performed using the indicator of ore preparation effectiveness of ore disclosure in accordance with the formula for its determination [15].

The calculated value of this index for each method of disintegration is given in Table 1. Based on this comparison, the most acceptable were centrifugal impact grinding, which greatly exceeds in the efficiency the grinding in the rod mill type. It is characteristic, that the value of the indicator of the effectiveness of ore pretreatment on disclosure of ore for this method of disintegration is close to unity, which indicates a preferential disclosure of ores for the planes of cleavage.

Comparing the evaluation results of the ores disclosure selectivity by various methods, we must say that for ores of Mazurovskoe deposits, grinding in a mill of centrifugal-impact type gives the best results. Thus there is a selective destruction of both ore and non-metallic (albite, microkline, nepheline) minerals. The degree of destruction is determined by the characteristics of ores and methods of disintegration. The data, confirming the existence of a selective destruction of ore, is shown in Fig. 1, 2 and in Table 1.

Table 1 – Evaluation results grinding of mariupolite

The methods of ores disintegration	Size (% class –0,1 mm)	Output of class, %				Contents Nb_2O_5, %				The degree of disclosure of non-ores material, %	Index of intracrystalline fracture (i)
		–0,044 mm		+0,044 mm		–0,044 mm		+0,044 mm			
		<2,8	>2,8	<2,8	>2,8	<2,8	>2,8	<2,8	>2,8		
Ball mill	96,3	43,84	0,96	51,77	3,43	0,070	2,95	0,040	0,59	77,4	0,38
Rod mill	94,8	37,68	1,22	56,87	4,23	0,068	3,01	0,038	0,82	71,2	0,48
Impact-centrifugal mill	93,5	18,30	1,60	74,16	5,94	0,027	0,92	0,023	4,70	93,7	0,87

In cours of mariupolite crushing, - there is a significant increase in the mass fraction of niobium pentoxide in thin classes. This increase can be used for fractionated enrichment of large and small classes after grinding. But such fractionation should provide high performance technology to extract of niobium pentoxide from thin classes. Based on the criterion of better disclosure of ores, as determined by the degree of expansion and the efficiency coefficient of pretreatment on disclosure of ores, the rational design of the mill for Mazurovskoe deposit ores is a centrifugal-impact type apparatus.

To justify the optimal technological parameters of ore selective crushing, investigations were carried out in ДЦ-0,36 centrifugal mill of centrifugal impact type. The mill was equipped with a dynamic centrifugal separator and dust cleaning system.

The dependence of the content of class –0,044 mm (C, %) in the crushed product on the peripheral speed of the rotor of centrifugal mill (V), the load on the mill in the original ore (Q) and size of original ore (d) were studied using active experiment. For planing of the experiment there was used rotatable central compositional plan of second order [21]. The main levels, the intervals of variation of factors and the study area boundary given in Table 2, have been selected on the basis of apriori information and the results of preliminary experiments.

Processing of results of experiments and analysis of regression models was performed using the module «Design of experiments» of statistical software Statgraphics Plus 5.1 [23].

Adequate model (R^2 = 99,5 %) taking into account the importance of factors given in normalized (coded) form received as the regression equation:

$$\begin{aligned}&C = 50{,}409 + 9{,}71388X_1 - 8{,}01511X_2 - 4{,}98155X_3 - 1{,}97968X_1^2 - \\ &- 0{,}99625X_1X_2 - 0{,}89375X_1X_3 + 0{,}848765X_3^2\end{aligned} \qquad (1)$$

Table 2 – The main levels, the intervals of variation of factors and investigation field boundaries

Parameters	Marking	Code	Unit	Main levels				
				+1,682	+1	0	-1	-1,682
Angular velocity (peripheral speed) of centrifugal mills rotor	*V*	X_1	m/sec	86,88	80	70	60	53,18
Size of output ore	*d*	X_2	mm	18,7	16	12	8	5,3
Consumption of output ore	*Q*	X_3	kg/h	2841	2500	2000	1500	1159

The values of the regression coefficients in equation (1) determine the strength of influence of the relevant factors or their combinations on the magnitude response function, and the sign before the coefficient, - the nature of this influence. As you can see, the content of the class –0,044 mm in the original product are mostly affected by peripheral speed of the rotor of the centrifugal mill and the particle size of the original ore. But we must take into acrount that the factors X_1 and X_3 enter the equation in the form of quadratic terms, that leads to an underestimation of their influence on the response function when evaluating the magnitude of the regression coefficients.

Analysis of the regression model, implemented using the statistical software «Statgraphics Plus 5.1», showed that the increase of peripheral speed of the centrifugal mill accelerator leads to a notable increase in the content of the class –0,044 mm in the original grinding product. The growth of the content of this class is slowing down slightly when you change the peripheral speed of the accelerator from 80 to 85-87 km/h. Speed 85-87 m/sec. can be considered optimal for the model of centrifugal-impact action mill.

The increase of original ore size, feeding to the grinding, leads to a decrease of the content of the class –0,044 mm. The highest content of this class occurs when grinding small ore particles size 6-4 mm.

Finally, the increased load on the centrifugal mill in the original ore, also leads to a decrease in the content of the class –0,044 mm in the finished product. Optimal one according to this criterion of load should take 1100-1200 kg/h.

As it is known, the separation efficiency of a mixture of minerals in a centrifugal field depends on the mobility of the mineral bed in the bowl of the concentrator. Therefore, when designing devices of this type, the main efforts are aimed at finding effective ways of breaking the minerals in the zones of concentration of heavy fractions in the separator. T oday there are several ways of loosening of mineral bed designs of centrifugal concentrators are differed [24]:

– due to the reversal translational harmonic vibrations of housing - drum, which rotates;

– with the help of special scrapers;
– with the help of a water jets applied to the surface of the pulp, which rotates;
– with the help of trapping cone vibrations;
– by changing the radius of the trapping cone surface;
– by supplying pressurizing water from the external side of trapping cone through the holes in it.

Let`s analyze the basic structural differences of these separators (Fig. 3). In separators of the first type of the separation of minerals occurs in a thin layer of slurry, which is fed onto the inner surface of sebokeng rotating drum (Fig. 3a). For loosening of adhering particles the drum makes along the axis the reciprocating harmonic vibrations with frequency and amplitude, which provide loosening (rosperity) of grains of the bed in which heavy particles of minerals are trapped.

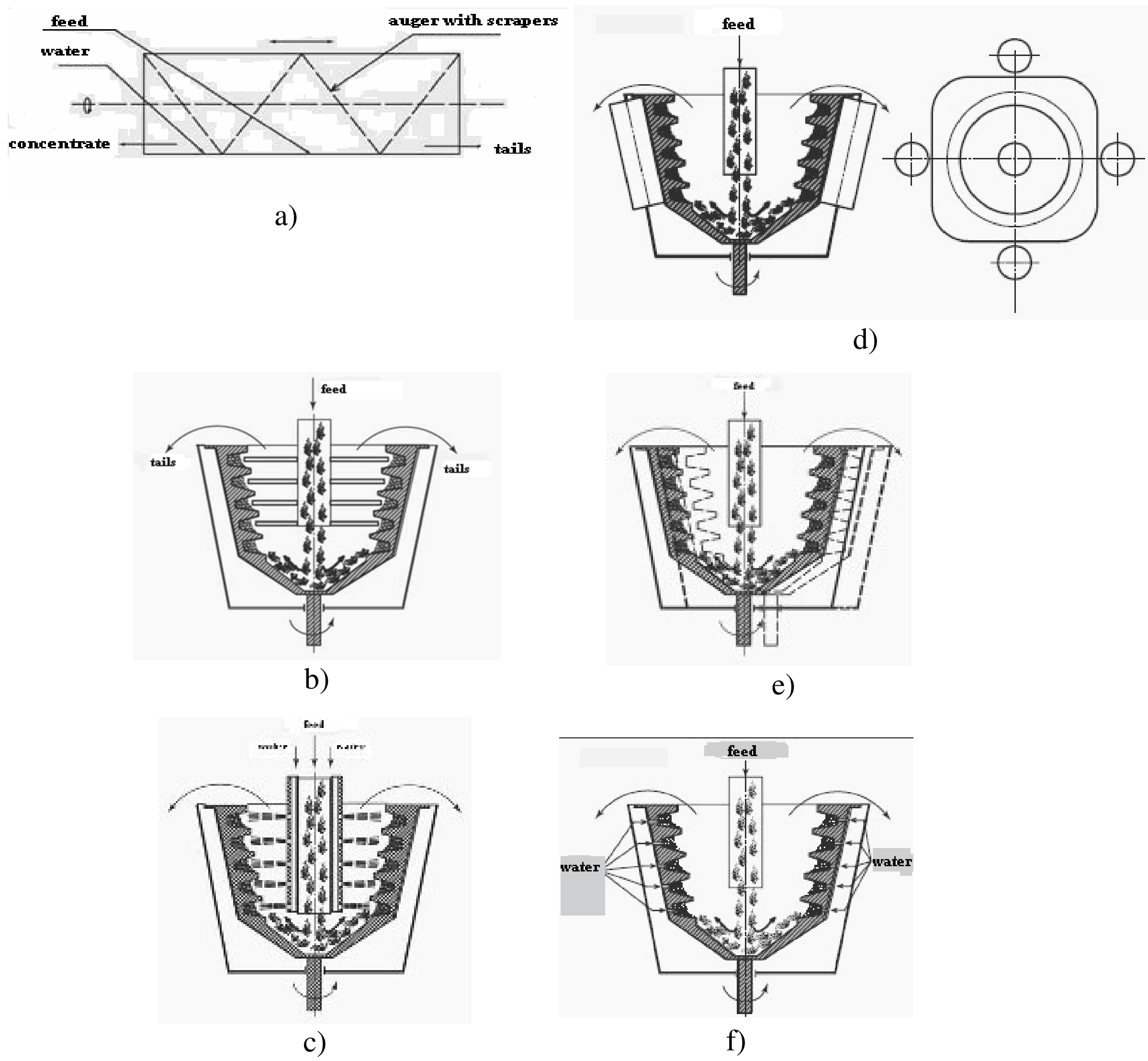


Figure 3 – Design features of mineral bed loosening [11]:
a – due to the reciprocal harmonic of the drum shell; b – using knives;
c – using jets of water that are fed to the pulp surface which rotates;
d – due to the change of the radius of the surface of the bowl (floating bed);
e – using the vibration of the bowl; f – by supplying water under pressure from the external side of the bowl through the holes in it.

Discharge of the concentrate is carried out using scrapers attached to the edges of the screw, which rotates with the frequency slightly lower the frequency of the drum rotation. This ensures the movement of the concentrate in the direction opposite to the direction of the pulp flow. The concentrate is additionally washed with fresh water. The enrichment in a thin layer of the pulp provides the highest separation index of small size particles of 5-6 microns. However, the productivity of such machines is small due to the fact that the pulp layer has a thickness of only 1-2 mm. For example, MGS industrial separators with drum diameter of about one meter have a capacity of 1,5-3,0 t/h [24].

Most of the existing separators of other types have a conical working bodies – cup, installed vertically or horizontally. The process of separating in them is involved in such a way. Pulp by itself, enters the lower part of the bowl that rotates. Under the action of centrifugal force, mineral particles are discarded to the walls of the cup and, rising up, fill annular recesses (riffle) in it. It is formed a bed of mineral particles. Slurry passing over the surface of the bed, loosens the layer of material to a small depth where sequestration of heavy grains take place. Light particles are displaced by the heavy ones and fall in the tails. Heavy particles are concentrated in riffles, until the filling of merifluor space and compaction of the bed. If not to loosen the bed, it will quickly be thicken, and in 10-15 minutes separation of heavy particles almost stop.

Rosperity of concentration of heavy fractions zone contributes to the penetration of the heavy minerals in the depth of bed so that lengthens the process of effective enrichment up to several hours. A way to hang the linen greatly affects the efficiency of the separation process in a centrifugal concentrator.

Centrifugal concentrator with a loosening of bed by scrapers (Fig. 3b) appeared on the market of mineral processing equipment in the early 1980-es in Australia [25]. While processing output material in such separator the amount of useful material in the concentrate to output ore is increased in 100-600 times. The removal of minerals with density of 8000 kg/m^3 and a particle size of −2,0+0,2 mm – up to 80 %. A serious drawback in the design of the separator, and primarily in the method of bed loosening, is that even small performance gains (10÷15 g), imposed on the pupl flow, compress the material in that part of riffles where scraper is missed, to such an extent that it is necessary to cut down while unloading concentrate. Howerer this part of valuable components is lost.

The design of the concentrator with bed loosened by water jets (Fig. 3c), which is supplied to the inner surface of the bowl is rotating, similar to the previous one, where instead of the scrapers along the radius through the holes of the hollow shaft they receive jets of water which loosen up the surface of the bed [26]. Many models of this concenterator are offered abroad, but the difficulty of adjusting the mode of separation, a significant impact on the process of loosening vibrations of the granulometric composition of pulp feeding, pressing interriffles space, difficulty rinsing of concentrate is not allowed to create a competitive machine.

Another construction of centrifugal concentrator with the bed loosening by changing the surface radius of the catching bowl (Fig. 3d) appeared in the 90-es of XX c. almost simultaneously in several countries. A special feature of its design is wrought catching vertical bowl made from polyurethane. A bowl, obtienen from three or four sides by the rollers, takes in its upper part the form of a triangle or square with smoothed angles. Settling in the interriffles space of the bowl, the particles, moving, periodically approach and receed from the axis of the bowl rotation. The bed is loosened mechanically with level of loosening depending on the frequency of the bowl rotation. On the heavy minerals concentrated in bowl riffles, there are overlaid variables on the rate of acceleration, and changes of the shape of the bowl surface provide the hanging of particles, that creates favorable conditions for separation. The main disadvantage of these separators is the lack of strength of the cup in rough use, which can withstand large alternating loads and connected with it energy intensity drive due to the need of the compressive rollers moving in the bowl [28].

Centrifugal concentrators with a bed loosening by using vibrations (Fig. 3d) of trapping cone (bowl) in which the regulation of concentrate solidification process is carried out by the blow vibration creating a circular planetary oscillations of the axis of the bowl rotation with a frequency of 150 Hz and an amplitude of 1-3 mm, were not used widely in industry on core operations through low productivity [27].

In the apparatus with the bed loosening due to water pressure (3e), which is supplied from the external side of the catch bowl through the holes in the bowl container to prevent the pressing of concentrate in riffles from the outside of the bowl towards the settling of mineral particles, water is supplied under pressure. Water jets create a common pressure gradient in riffle directed toward the centrifugal field, loosen concentrated layer of minerals, creating favorable conditions for the enrichment process. The ability to control the pressure of water supplied to the cup, allows to adjust the degree of bed loosening in concentrate riffles, which in turn creates optimal conditions for the concentration of minerals of different density and size. The degree of concentration in the separator of this type is up to 3000. The disadvantages of this device include low extraction of particle size of less than 0,02 mm (40-60 %) and the need for individual, experimental selection of water pressure for each type of mineral raw. At the same time, these concentrates are most widely used in industry, as in practice of enriching the necessity of crushing ore to a particle size less than 0,02 mm for the disclosure of minerals rarely occurs, and selection of the required water pressure for a particular type of mineral raw materials is not difficult [29].

Analysis of structures for free-flow concentrators, results of comparative tests of equipment of various types [30], allows to recommend centrifugal separator of the latter type for the benefication of rare-metal ores of Mazurovskoe deposit.

In Fig. 4 it is showd a scheme of a centrifugal concentrator construction, which explains the principle of the appararus operation. Separation of minerals in this concentrator occurs in the followsing way. Pulp a long the supply tube 6 is fed into the bowl 3 and sinks to the bottom. Under the action of centrifugal forces particles of the solid phase are displaced on the inner walls of the bowl to the top and consistently fill the riffles 2. Towards the particles motion from the holes 4 in riffle water jets are supplied from water jacket of the cup 5.

The water flow in the separator cup provides the advantage that the feed, concentrate and tails are less thickened, which prevents the silting-up of discharge lines.

The combination of gravitational acceleration high force (from 60g to 200g) and the original process of cultivation bed loosening by water provide such design with the higher degree of extraction of heavy mineral grains. With that it is achieved the maximum degree of reduction, full automation and concentrate saving (which is important, because of the high cost of rare metals concentrates). The capacity factor of such equipment reaches 98 %.

The design of the concentrator of periodic action, which implements this method of bed loosening is shown in Fig. 5. In the enclosure of the apparatus, there closed a removable cover from the top 5 there is a shaft unit 8, mounted in two bearing supports. On the rotor there is mounted the bowl (cone) 6 with a water jacket. On the inner surface of polyurethane bowls special annular grooves – riffles are made, which accumulate the heavy fraction of material, which is separated. Water from the water jacket is fed into the middle of the bowl under slight pressure through the openings systam in the bore walls. At the bottom of the bowl there is a special plug-in cone 7, which prevents rapid wearing process of the bowl bottom. On the alternating cone, it is fixed baffle plate 1, which serves for distribution of supply pulp. Concentrate supply by the pulp is carried out through the supply pipe 2, which passes through the removable lid. Through a multiport node 11 and the sealing sleeve 9 it is used the supply of water into the bowl of the concentrator. Erosion of compacted sludge is carried out through a chute 12. Benefication tails are discharged from the apparatus via a plug-in nozzles 4, a removable lid 5 and slot. Rotation of the concentrator bowl is provided from the electric drive through a belt drive and pulley on the rotor shaft 8.

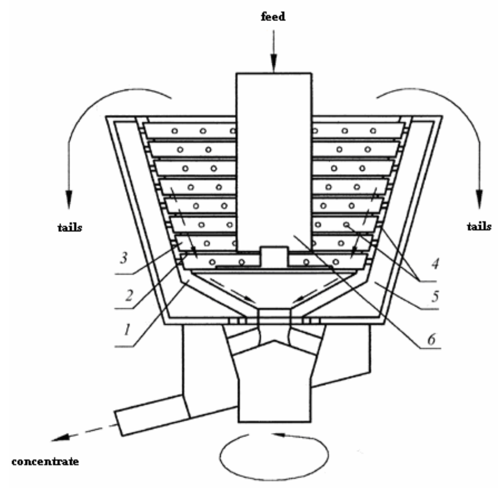
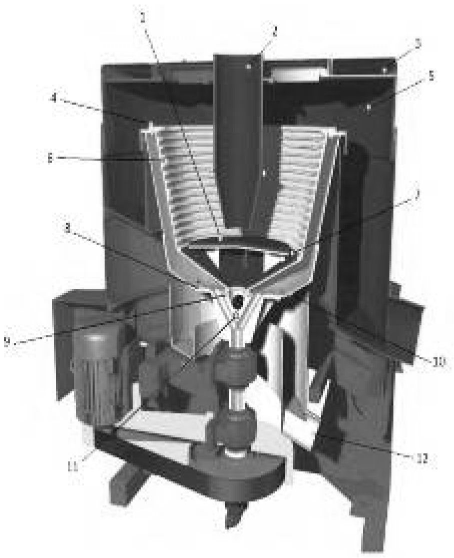


Figure 4 – diagram of the concentrators:
1 – concentrate removal chute; 2 – the annular grooves (riffles); 3 – concentrator bowl;
4 – holes for water flow; 5 – water jacket; 6 – supply pipe



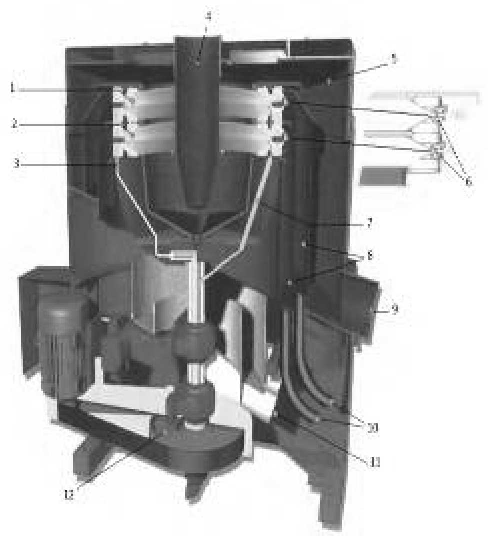


Figure 5 – KNELSON KC-XD concentrator with periodic loading out of the concentrate:
1 – reflective plate;
2 – supply pipe;
3 – changeable wearing skirt;
4 – changeable nozzles for discharge of sludge;
5 – removable cover;
6 – resistant polyurethane bowl;
7 – changeable wearing cone;
8 – rotor with removable shaft;
9 – seal sleeve;
10 – drain plug sediment;
11 – multiport junction;
12 – concentrate washing out shute

Figure 6 – KNELSON CVD-32-2 concentrator with continuous loading out of the concentrate:
1 – the second concentration ring;
2 – the first concentration ring;
3 – air supply;
4 – supply pipe;
5 – collection of tails;
6 – valves;
7 – water jacket;
8 – concentrate collections;
9 – tailings loading out chute;
10 – concentrate loading out nozzles;
11 – drain pipe;
12 – electric drive

The concentrator operation is carried out in the following way. Enrichment cycle begins with water supply from the water jacket through the holes system in the walls of the bowl (in riffles) in the middle of the bowl under a slight pressure. Then along the supply pipe in the bowl pulp is fed. Upon reaching the flow allocator at the bottom of the cocentration bowl, pulp is directed upwards under the action of centrifugal force. The solid phase of the pulp fills the annular grooves (riffles) from the top to bottom. Upon accumulation of solidphase in the bowl there is formed a concentrational bed. Particles with high specific gravity are concentrated in riffles of the bowl, while lighter particles of gangue material are forced out of the bowl into the tailings chute.

After the completion of the established cycle of enrichment it occurs flush of accumulated concentrate in the concentrator chute through the original multi-port node. This operation is carried out automatically and takes maximum 2-3 minutes. While washing out staff access to concentrate is excluded which ensures their safety.

During machine operation all the particles in the concentrator bowl are under the influence of centrifugal force, the magnitude of which is regulated depending on the particular application conditions. The choice of the optimal rotor speed is based on several technological factors, such as the results of laboratory testing, specific gravity of heavy mineral grains and rocks, granulometric composition and content of useful mineral in the ore.

The concentrators with continuous discharge of the concentrate (Fig. 6) differ structurally from the series XD. They are specially designed for the case when it's necessary to have high output of the concentrate, which cannot be achieved in concentrators with periodic loading out. In these concentrators water is used for loosening the bed that serves in the middle of the bowl through the system of holes in the bowl walls, and high centrifugal accelerations. Pulp feed supply and loading out of the concentrate takes place simultaneously.

The work of the concentrator is taking place in the following way. First, in the middle of the bowl from a water jacket water is supplied through a system of holes in riffles. Then the supply pulp is fed through the supply pulp into the bowl.

Having reached the bottom of the bowl, the pulp under the centrifugal force effect is pushed up on the cup walls to a concentration of grooves (riffles). The pulp solid phase fills riffles, forming a bed. Water supplying into the bowl, provides more effective catching of the grains of the heavy fraction in riffles. The valves are actuated by compressed air, so the operator can regulate the concentrate output, if it's necessary, - apart from each ring. The concentrate is sent into the concentrates chute, and the tails are dropped through the upper part of the bowl into the tailings chute.

Under the pilot conditions the experimental «Azov-Mineraltechnika» wash house there were carried out investigations on the influence of methods of desintegration on the ore benefication index in the separator of centrifugal type. The original output ore was divided into two parts of approximately of 120 tons, one of which was grinded in the traditional ball mill of the MPs 1200*1200 type, the other - in a centrifugal mill of DC-0,36 throwing type. Both grinding devices worked in closed circuit with a separator of a transmission type. Milling was carried out to a particle size of 0,10 mm.

Ready-crushed material after each of the grinding devices again were divided into two parties, one of which was used directly for the preparation of pulp and served in a STS-400 centrifugal concentrator – the equivalent of the KNELSON CVD concentrator. The other party was subjected to magnetic separation at drum magnetic separators of СБ-25-100/0,25 and СБ-25-100/0,6 type for providing of optimum content of intermediate density minerals in the ore; then also it was served for the pulp preparation.

There were compared the results of minerals separation in a centrifugal field with different methods of grinding, and with different contents of minerals of intermediate density.

We investigated the nature of the pyrochlore grains disclosure with chosen grinding methods, the nature of pyrochlor grain distribution in size, and content of pentoxide of niobium in various size fractions, and the impact of weaken effect of the centrifugal mill on the process of consequent recovery of pyrochlore in gravity-based ore benefication in a centrifugal concentrator.

The main method of determining the content of chemical elements in grinded and enriched products was x-ray analysis. The mineral composition of samples was controlled by optical method using a microscope, particle-size composition – sieve and sedimentation method.

The results of the analysis of ore grain-size classes are given in Table 3.

The ore crushed in the ball mill was divided into two technological samples weighing about 60 tonnes each, after that one sample was sent for benefication in a centrifugal concentrator, and the other one to magnetic separation. Magnetic separation was carried out in induction on the drum separator surface of 0,15 Tesla and 0,55 Tesla.

Table 3 – Grain-size classes distribution $(Nb,Ta)_2O_5$ in maripolite, grinded to 0,1 mm

Grain-size classes, mm	Grinding in a ball mill			Grinding in a mill of impact-centrifugal action		
	Output, %	Mass fraction $(Nb,Ta)_2O_5$, %	Distribution $(Nb,Ta)_2O_5$ %	Output, %	Mass fraction $(Nb,Ta)_2O_5$ %	Distribution $(Nb,Ta)_2O_5$, %
+0,074	6,4	0,060	3,84	16,3	0,052	8,48
-0,071+0,063	19,3	0,070	13,51	27,2	0,066	17,95
-0,063+0,050	29,5	0,080	23,60	20,1	0,095	19,00
-0,050+0,040	15,0	0,090	13,50	17,7	0,150	26,55
-0,040+0,020	10,5	0,101	11,36	8,9	0,170	15,13
-0,020+0,010	6,2	0,150	9,30	4,1	0,126	5,20
-0,01	13,1	0,190	24,89	5,7	0,135	7,69
Output material	100,0	0,100	100,00	100,0	0,100	100,00

The results of the ore minerals separation into heavy and light fractions without a preliminary partial recovery of minerals of intermediate density are given in table 4; with preliminary removal – in table 5.

Table 4 – Indicators of ore benefication in Nelsons concentrator without preliminary extraction of intermediate density minerals

Products of benefication	Grinding in a ball mill			Grinding in a mill of impact-centrifugal action		
	Output, %	Mass fraction $(Nb,Ta)_2O_5$, %	Distribution $(Nb,Ta)_2O_5$ %	Output, %	Mass fraction $(Nb,Ta)_2O_5$ %	Distribution $(Nb,Ta)_2O_5$, %
Heavy fraction	4,39	1,110	48,52	3,22	2,030	65,37
Light fraction	95,61	0,054	51,48	96,78	0,036	34,63
TOTAL:	100,0	0,1000	100,00	100,0	0,100	100,0

Table 5 – Indicators of ore benefication in Nelsons concentrator with the preliminary extraction of intermediate density minerals

Products of benefication	Grinding in a ball mill			Grinding in a mill of impact-centrifugal action		
	Output, %	Mass fraction $(Nb,Ta)_2O_5$, %	Distribution $(Nb,Ta)_2O_5$ %	Output, %	Mass fraction $(Nb,Ta)_2O_5$ %	Distribution $(Nb,Ta)_2O_5$, %
Heavy fraction	3,79	1,36	55,54	2,52	3,22	77,25
Light fraction	96,21	0,049	44,46	97,48	0,028	22,75
TOTAL:	100,0	0,1000	100,00	100,0	0,100	100,0

Analysis of research results shows that the use of selective grinding in a mill of impact-centrifugal action type allows to get more complete disclosure of pyrochlore in coarser heavier grain-size classes. As a result, a centrifugal concentrator allows to increase in such grinding the mass fraction of niobium pentoxide in the rough concentrate to 2,03 % in extraction 65,37 %.

Partial recovery of intermediate density minerals prior to gravitational enrichment in a centrifugal field leads to a substantial increase in the degree of pyrochlore extraction, especially in the case of ore grinding in the mill of impact-centrifugal type. The mass fraction of niobium pentoxide in this case is 3,22 % in extraction of 77,25 %.

Prepared in this way ore is enriched in a centrifugal gravity-based separator with high recovery – 35 % (relative) higher then pyrochlores extractions from the ore, prepared in a ball mill. At this the content of niobium pentoxide in draft concentrate (2,03 %) is almost twice higher its value in grinding ore in a ball mill (1,11 %).

Preliminary removal of intermediate density minerals from the ore leads to a further increase in the degree of pyrochlores extraction in centrifugal grinding - by 39.1 % (relative), in comparison with the grinding in the ball mill.

Conclusions.

1. The mill of impact-centrifugal action is an efficient machine for grinding of rare metals ore of Mazurovskoe deposit from the point of view of minimization of niobium valuable component losses with slurries, and disclosure of minerals. At this there takes place selective destruction of both ore and non-metallic (albite, microckline, nepheline) minerals.

2. Disclosure of minerals in this mill is taking place in coarser class – 0,08÷0,071 mm. It promotes the growth of gravitational enrichment indicators. The minerals output of the «technological» (the most open) class at this significantly higher.

3. Optimal technical and technological parameters of the mill of impact-centrifugal action are: peripheral speed of the rotor – 85÷87 m/sec., the ore output productivity – 1100÷1200 kg/h., the initial ore particle size of 4÷6 mm.

4. Design of centrifugal concentrator with bed loosening by jets of water is optimal for the separation of pyrochlore ore minerals of Mazurovsky Deposit.

5. The ore, grinded in a mill of the impact-centrifugal action type, is beneficated in the centrifugal gravitation separator with the recovery extractions higher (relative) on 35 % then pyrochlore extractions from the ore, prepared in a ball mill. At this the content of niobium pentoxide in rough concentrate (2,03 per cent) is almost twice higher its value in grinding ore in a ball mill (1,11 %).

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Received 01.11.2016

UDC 69.055:65.012.32:004.942

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RATIONAL ORGANIZATIONAL AND TECHNOLOGICAL DECISIONS ON THE GRAIN STORAGES CONSTRUCTION OR RENOVATION SITES

The article presents a method for choosing rational organizational and technological decisions at the grain storages construction or reconstruction sites using the results of the author's experimental and theoretical studies. It is given conditions analysis for the investigated enterprises operational activity. The obtained experimental statistical dependencies of the indicators change of such operational activity from the varied organizational and technological factors and the developed algorithms allow choosing rational organizational and technological decisions for the elevators construction and reconstruction based on the construction site conditions analysis and on the effective planning.

***Keywords:** organizational and technological solutions, grain storages construction and renovation, experimental and statistical modeling, algorithm.*

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Одеська державна академія будівництва і архітектури

ВИБІР РАЦІОНАЛЬНИХ ОРГАНІЗАЦІЙНО-ТЕХНОЛОГІЧНИХ РІШЕНЬ НА ОБ'ЄКТАХ З БУДІВНИЦТВА АБО РЕКОНСТРУКЦІЇ ЕЛЕВАТОРІВ

У статті представлена методика вибору раціональних організаційно-технологічних рішень на об'єктах з будівництва або реконструкції елеваторів з використанням результатів експериментально-теоретичних досліджень автора. Наведено аналіз умов, в яких здійснюється операційна діяльність розглянутих підприємств. Отримані експериментально-статистичні залежності зміни показників такої операційної діяльності від організаційно-технологічних факторів і розроблені алгоритми дозволяють вибрати раціональні організаційно-технологічні рішення при будівництві та реконструкції елеваторів на підставі аналізу умов будівництва об'єкта і ефективного планування.

Ключові слова: організаційно-технологічні рішення, будівництво і реконструкція елеваторів, експериментально-статистичне моделювання, алгоритм.

Introduction. Experts estimate the volume of certified facilities for the grains and oilseeds storage in Ukraine at 31-33 mln. tons. Silo capacity deficit is about 15-20 mln. tons, considering the annual carryover grain stocks in Ukraine (about 10 mln. tons) and the expected crop volume at 40 mln. tons. Special conditions of grain storages construction and reconstruction projects realization require systematic studies on the optimization of organizational and technological enterprises solutions in focus. Such research will improve the organizational, technological and economic efficiency of the grain storages construction and renovation companies management.

Analysis of the latest sources of research and publications. Data on the segmentation of the grain storages construction market in the world [1, 2] show that a significant proportion of the work is to upgrade existing storage facilities. Typically, this modernization involves the commissioning of new silos, the upgrading of technological equipment, productivity enhancement of transport lines and individual technological units of grain storage, associated with this dismantling work and the construction of small additional structures. As a rule, grain storage modernization has rarely large scale. Grain storages reconstruction projects may have a budget up to 1 million UAH and labor input of construction and installation works up to 3 thousand hours [3]. Nevertheless, there are still tendencies to build new wide-scale grain storages and carry out large-scale renovation of existing ones. It can be concluded that the largest object for typical grain storages construction and renovation enterprise will have budget for about 25-30 million UAH and the total labor intensity of construction and installation works for about 40 thousand hours [3].

Statistical methods for solving optimization problems applying are widely used [9, 10]. Analysis of works, devoted to the optimization of organizational and technological solutions for construction and reconstruction [5, 6], allows to conclude that the application of experimental statistical modeling is an effective way of solving similar problems and can be used in modeling and optimizing the operating activity of grain storages construction and renovation enterprises.

The application of experimental statistical modeling for the methods of optimization is discussed in [4, 7, 8]. It is advisable [5, 6] to use specialized programs for project management to create operating activity model of the construction organization.

Allocation of unresolved parts of the general problem. According to the results of the information sources analysis, it was established that a number of outstanding scientists were engaged in the development of methods for choice of rational organizational and technological decisions. Dikman L., Chernenko V., Kirnos V., Zalunin V., Dadiverina L., Ushackiy S., Berezyuk A. are among them. However, algorithmized solutions for solving this problem have not been developed when managing works on individual construction projects and the enterprise as a whole, which is done in special conditions of grain storages construction and renovation: territorial fragmentation of construction or reconstruction sites; differences in their scales; specificity of construction and installation works.

Formulation of the problem. The purpose of the article is to develop algorithms for choosing rational organizational and technological solutions at individual grain storages construction and renovation sites.

The essence of the method of choosing rational organizational and technological method solutions is the consistent decision of the following tasks:

– Identify the specificity of the conditions of grain storages construction and renovation.

– Construct experimental statistical dependencies of the studied indicators from the varying organizational and technological factors.

– Develop an algorithm for calendar and network planning of the grain storages construction and renovation, taking into account the analysis and improvement of organizational and technological solutions.

– Create an algorithm for choosing organizational and technological solutions at the grain storages construction or reconstruction sites.

Main part and results. To evaluate the efficiency and to select optimal organizational and technological solutions for the management of the grain storages construction and renovation enterprise, it is proposed to use the experimental statistical modeling theory. The essence of this theory is in observing the system by fixing the values of the outgoing parameters when specifying input parameters. The system under investigation in this study is presented in the form of a computer model of the company's operating activity. As the investigated indicators, the following factors were considered:

– Total production cost change (Y_1) – percentage of total production cost change, depending on the impact of organizational and technological factors. The cost change is zero in a basic model, which reflects the most typical operating activity conditions of the grain storages construction and renovation enterprise. In the present study, such model is observed at the middle levels of the considered factors. Total production costs are the sum of direct and general production costs.

– Ratio of direct and general production costs (Y_2) – the percentage of total production to the amount of direct costs for a totality of projects.

– Cost of construction product unit – direct costs, which are necessary for the production of a construction product unit of the enterprise: reinforced concrete structures (Y_3 – 1 m^3); load-bearing metal structures (Y_4 – 1 ton); cubic meter of grain silo storage (Y_5 – 1 m^3 of storage); section of transport equipment (noria (Y_6), conveyor (Y_7) – 1 m.).

Varying organizational and technological factors and their numerical characteristics are presented below:

– X_1 – average complexity of projects totality (average arithmetic complexity of construction and installation works of the projects under consideration, mln. UAH).

– X_2 – average relocation distance (average arithmetic distance of the relocation of resources between any two projects from the totality under consideration, km.).

– X_3 – ownership of the used resources (the percentage of own resources use to the total volume of used resources).

– X_4 – industrialization of applied solutions (percentage ratio of industrial methods use in the total amount of work).

The results of the numerical experiment are shown in the Table 1.

As a result of the experimental statistical modeling, change dependencies of studied parameters (1 – 7) from the variable factors were obtained.

The algorithm of calendar and network planning for the grain storages construction and renovation is shown below (Fig. 1), considering the analysis and improvement of organizational and technological solutions. It can be used for the implementation of grain storages construction and reconstruction projects of any scale or remoteness.

The general organization of technological flows during the grain storages construction and renovation is desirable to implement using the following principles:

– Planning of technological flows is rationally to implement with the help of project management software. The automation of planning is possible by the preparation of templates, containing a particular set of work, technological flow, the construction of typical objects.

Table 1 – Results of experimental statistical modeling

№	Actual values of the factors				Indicators						
	X_1, thousand hours.	X_2, km.	X_3, %	X_4, %	Total production cost change, Y_1, %	Ratio of direct and general production costs, Y_2, %	Cost of reinforced concrete structures unit, Y_3	Cost of load-bearing metal structures unit, Y_4	Cost of cubic meter of grain silo storage, Y_5	Cost of noria section, Y_6	Cost of conveyor section, Y_7
1	6	7	8	9	10	11	12	13	14	15	16
1	37	1000	100	100	-0,222	8,20	3 276,17	4 653,77	41,50	1 196,46	794,88
2	37	1000	100	0	5,223	7,75	3 766,31	5 170,86	49,66	1 329,40	883,20
3	37	1000	0	100	-4,647	10,68	3 162,74	4 046,76	36,13	1 040,40	709,00
4	37	1000	0	0	0,373	10,09	3 627,29	4 496,40	43,24	1 156,00	787,77
5	37	100	100	100	-1,691	6,61	3 276,17	4 653,77	41,50	1 196,46	794,88
6	37	100	100	0	3,753	6,24	3 766,31	5 170,86	49,66	1 329,40	883,20
7	37	100	0	100	-7,587	7,27	3 162,74	4 046,76	36,13	1 040,40	709,00
8	37	100	0	0	-2,566	7,27	3 627,29	4 496,40	43,24	1 156,00	787,77
9	2,2	1000	100	100	2,301	16,13	3 888,06	4 653,77	72,88	1 218,85	843,68
10	2,2	1000	100	0	-1,015	16,76	3 722,22	5 170,86	87,43	1 314,07	937,42
11	2,2	1000	0	100	3,225	27,84	3 736,82	4 046,76	63,49	1 059,87	752,42
12	2,2	1000	0	0	0,333	28,87	3 586,13	4 496,40	76,16	1 142,67	836,02
13	2,2	100	100	100	-5,141	7,69	3 888,06	4 653,77	72,88	1 218,85	843,68
14	2,2	100	100	0	-8,457	7,99	3 722,22	5 170,86	87,43	1 314,07	937,42
15	2,2	100	0	100	-11,658	9,41	3 736,82	4 046,76	63,49	1 059,87	752,42
16	2,2	100	0	0	-14,550	9,76	3 586,13	4 496,40	76,16	1 142,67	836,02
17	37	550	50	50	-0,967	7,93	3 452,90	4 591,95	42,55	1 180,56	793,71
18	2,2	550	50	50	-2,274	15,02	3 733,31	4 591,95	74,84	1 183,86	842,39
19	19,6	1000	50	50	0,895	10,98	3 448,75	4 591,95	43,04	1 188,16	818,38
20	19,6	100	50	50	-2,125	7,65	3 448,75	4 591,95	43,04	1 188,16	818,38
21	19,6	550	100	50	1,896	8,35	3 511,51	4 912,32	46,01	1 271,05	865,33
22	19,6	550	50	100	0,063	9,25	3 286,01	4 653,77	41,97	1 205,04	819,87
23	19,6	550	0	50	-3,127	10,35	3 385,98	4 271,58	40,06	1 105,26	771,43
24	19,6	550	50	0	1,742	9,08	3 678,66	4 833,63	46,98	1 249,87	861,37
25	19,6	550	50	50	-0,615	9,31	3 448,75	4 591,95	43,04	1 188,16	818,38

$$Y_1 = 0{,}557\,X_1 - 13{,}083 - 0{,}006\,X_1^2 - 2\times10^{-4}\,X_1X_2 + 8\times10^{-4}\,X_1X_3 - 0{,}002\,X_1X_4 + 0{,}018\,X_2 - 4\times10^{-6}\,X_2^2 - 5\times10^{-5}\,X_2X_3 + 0{,}06\,X_3 + 0{,}037\,X_4. \quad (1)$$

$$Y_2 = 9{,}281 - 3{,}746\,X_1 + 2{,}469\,X_{12} - 2{,}839\,X_1X_2 + 1{,}3\,X_1X_3 + 3{,}745\,X_2 - 1{,}466\,X_2X_3 - 1{,}99\,X_3. \quad (2)$$

$$Y_3 = 3634{,}4 - 16{,}475\,X_1 + 0{,}453\,X_1^2 - 0{,}183\,X_1X_4 + 1{,}339\,X_3 - 1{,}801\,X_4. \quad (3)$$

$$Y_4 = 4576{,}419 + 8{,}664\,X_3 - 0{,}019\,X_{32} - 0{,}007\,X_3X_4 - 8{,}308\,X_4 + 0{,}041\,X_{42}. \quad (4)$$

$$Y_5 = 82{,}312 - 2{,}932\,X_1 + 0{,}051\,X_{12} - 0{,}001\,X_1X_3 + 0{,}002\,X_1X_4 + 0{,}112\,X_3 - 1{,}5\times10^{-4}\,X_3X_4 - 0{,}126\,X_4. \quad (5)$$

$$Y_6 = 1180{,}606 + 2{,}221\,X_3 - 0{,}005\,X_{32} - 0{,}002\,X_3X_4 - 1{,}461\,X_4 + 0{,}011\,X_{42}. \quad (6)$$

$$Y_7 = 844{,}439 - 0{,}449\,X_1 - 0{,}024\,X_{12} + 1{,}216\,X_3 - 2{,}8\times10^{-3}\,X_{32} - 1{,}42\,X_4 + 6{,}08\times10^{-3}\,X_{42}. \quad (7)$$

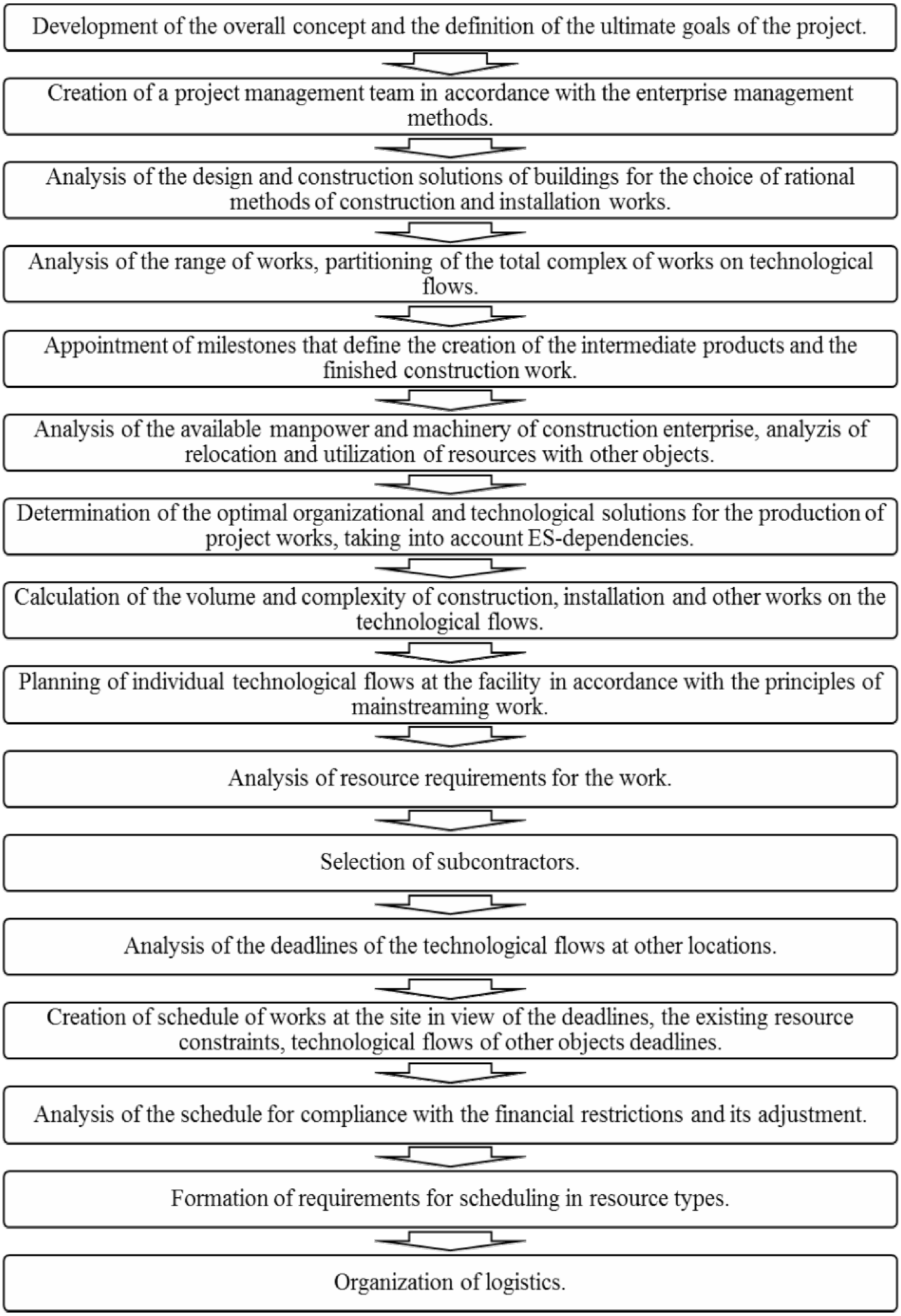

Figure 1 – Algorithm of calendar and network planning of grain storages construction and renovation sites

– The linking of technological flows to each other can be carried out sequentially, in parallel, in combination. Linkage is regulated by technological and organizational links. Technological links ensure compliance with the technology of works production. Organizational links regulate the supply of labor resources and equipment and can be placed either between the flows of one project, or between different projects.
– The the assessing criterion of the correctness of the calendar plan development is a schedule of labor resources consumption of each qualification and in general. Correctly designed plan will ensure smooth increase and decrease in the consumption of labor resources in time.
– The division of the site is different for small and large projects. The work is often done at one or two places of the grain storage plan in case of small construction or renovation projects. In such a case, it is advisable to divide the site according to the technological nodes of the grain storage, to the individual places of construction work production. For large projects, it is rational to tie more closely to the places of the grain storage plan or to combine several small places into one capture. Such capture will correspond from the work complexity viewpoint to one large place of the grain storage plan.

The experimental statistical dependencies can be used for the adoption of optimal organizational and technological solutions at the grain storages construction and renovation sites. The algorithm for making such decisions is shown in Fig. 2.

The algorithm, shown in Fig. 2, assumes acceptance of the compromise administrative decision for each kind of the construction or installation works executed on object. It is necessary to choose the optimal organizational and technological decisions for performing a particular type of work, considering the rational level of reduction of total production costs (Y_1) by solving a system of inequalities (8):

$$\begin{cases} Y_1 \geq f(X_3; X_4) \\ Y_n = f(X_3; X_4), n \in (3,...7) \end{cases}. \tag{8}$$

The upper inequality of system 8 allows to set rational level for total production costs reduction, the lower equation is to choose the optimal pair of factors X_3 (ownership of the used resources) and X_4 (industrialization of applied solutions). Different combinations of levels of X_3 and X_4 factors are possible when solving such a system. The final choice depends on the availability of the company's own resources for the performance of a particular type of work, the availability of high-performance equipment or mechanisms, the possibility and feasibility of using industrial methods of production.

For convenience of utilization, the experimental statistical dependences were calculated on the basis of formulas 1-7 in a separate approach for each combination of strategic organizational and technological solutions (levels of factors X_1 and X_2). Calculation of economic benefit can be made by finding the difference between the maximum value of construction products and unit production cost, obtained on selected levels of factors X_3 and X_4, and multiplying this difference by the physical volume of the respective work type. If necessary, steps 3 and 4 of the algorithm can be repeated several times.

Conclusions:

1. Analysis of grain storages construction and renovation industry, and the use of experimental and statistical modeling allowed building change dependences of the most important indicators on the organizational and technological factors.
2. The developed algorithm of calendar and network planning allowed optimizing the process of grain storages construction and renovation.
3. The built experimental statistical dependencies made it possible to optimize the organizational and technological solutions of the certain area of building production using a specially developed algorithm.

1) Calculate the volume of the project construction work.

2) Select the volume of construction and installation work for the following construction unit:
Y_3 – reinforced concrete structures;
Y_4 – load-bearing metal structures;
Y_5 – cubic meter of silo storage;
Y_6, Y_7 – section of noria, and plain conveyor.

3) Using the ES-dependencies, listed below, determine the optimal levels of factors, X_3 and X_4 for each type of work in the following way:
- Select suitable ES-dependence and set the optimum level of total operating costs reduction in accordance with the strategic enterprise solutions (levels of factors X_1 and X_2).
- Solve the system of inequalities for the construction work types, select the optimal levels of factors X_3 and X_4.

<table>
<tr><td>X_1</td><td>X_2 = 100 km.</td><td>X_2 = 550 km.</td><td>X_2 = 1100 km.</td></tr>
<tr><td rowspan="2">37 thousand hours.</td><td>$Y_1 \leq 3{,}51\%$
$Y_1 = 0{,}332 + 0{,}085\ X_3 –$</td><td>$Y_1 \leq 3\%$
$Y_1 = 3{,}932 + 0{,}062\ X_3 – 0{,}037\ X_4$</td><td>$Y_1 \leq 8{,}56\%$
$Y_1 = 6{,}077 + 0{,}09\ X_3 – 0{,}037\ X_4$</td></tr>
<tr><td colspan="3">$Y_3 = 3644{,}982 + 1{,}339\ X_3 – 8{,}572\ X_4$
$Y_4 = 4576{,}419 + 8{,}664\ X_3 – 0{,}019\ X_3^2 – 0{,}007\ X_3X_4 – 8{,}308\ X_4 + 0{,}041\ X_4^2$
$Y_5 = 43{,}647 + 0{,}075\ X_3 – 1{,}5\times10^{-4}\ X_3X_4 – 0{,}052\ X_4$
$Y_6 = 1180{,}606 + 2{,}221\ X_3 – 0{,}005\ X_3^2 – 0{,}002\ X_3X_4 – 1{,}461\ X_4 + 0{,}011\ X_4^2$
$Y_7 = 704{,}07 + 1{,}216\ X_3 – 2{,}8\times10^{-3}\ X_3^2 – 1{,}42\ X_4 + 6{,}08\times10^{-3}\ X_4^2$</td></tr>
<tr><td rowspan="2">19.6 thousand hours.</td><td>$Y_1 \leq 5{,}73\%$
$Y_1 = – 3{,}103 + 0{,}071\ X_3 –$</td><td>$Y_1 \leq 5{,}22\%$
$Y_1 = 2{,}063 + 0{,}048\ X_3 –$</td><td>$Y_1 \leq 10{,}79\%$
$Y_1 = 6{,}122 + 0{,}076\ X_3 –$</td></tr>
<tr><td colspan="3">$Y_3 = 3485{,}51 + 1{,}339\ X_3 – 5{,}388\ X_4$
$Y_4 = 4576{,}419 + 8{,}664\ X_3 – 0{,}019\ X_3^2 – 0{,}007\ X_3X_4 – 8{,}308\ X_4 + 0{,}041\ X_4^2$
$Y_5 = 44{,}437 + 0{,}092\ X_3 – 1{,}5\times10^{-4}\ X_3X_4 – 0{,}087\ X_4$
$Y_6 = 1180{,}606 + 2{,}221\ X_3 – 0{,}005\ X_3^2 – 0{,}002\ X_3X_4 – 1{,}461\ X_4 + 0{,}011\ X_4^2$
$Y_7 = 826{,}419 + 1{,}216\ X_3 – 2{,}8\times10^{-3}\ X_3^2 – 1{,}42\ X_4 + 6{,}08\times10^{-3}\ X_4^2$</td></tr>
<tr><td rowspan="2">2.2 thousand hours.</td><td>$Y_1 \leq 7{,}96\%$
$Y_1 = – 10{,}432 + 0{,}057\ X_3 +$</td><td>$Y_1 \leq 7{,}45\%$
$Y_1 = – 3{,}444 + 0{,}034\ X_3 – 0{,}007\ X_4$</td><td>$Y_1 \leq 13{,}02\%$
$Y_1 = 2{,}584 + 0{,}007\ X_3 –$</td></tr>
<tr><td colspan="3">$Y_3 = 3600{,}35 + 1{,}339\ X_3 – 2{,}204\ X_4$
$Y_4 = 4576{,}419 + 8{,}664\ X_3 – 0{,}019\ X_3^2 – 0{,}007\ X_3X_4 – 8{,}308\ X_4 + 0{,}041\ X_4^2$
$Y_5 = 76{,}109 + 0{,}11\ X_3 – 1{,}5\times10^{-4}\ X_3X_4 – 0{,}122\ X_4$
$Y_6 = 1180{,}606 + 2{,}221\ X_3 – 0{,}005\ X_3^2 – 0{,}002\ X_3X_4 – 1{,}461\ X_4 + 0{,}011\ X_4^2$
$Y_7 = 843{,}335 + 1{,}216\ X_3 – 2{,}8\times10^{-3}\ X_3^2 – 1{,}42\ X_4 + 6{,}08\times10^{-3}\ X_4^2$</td></tr>
</table>

4) Determine the economic impact of the application of organizational and technological decisions for each type of work.

5) Make a conclusion about the use of various organizational and technological decisions for each type of work, based on the work load of the enterprise resources.

Figure 2 – Algorithm of choosing rational organizational and technological decisions on the grain storages construction or renovation sites

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Received 02.03.2017

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