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Dynamics research of unbalanced vibrator with variable static moment

Lyutenko Vasyl[1]*, Rudyk Rostyslav[2], Nasulloev Shahrom[3]

[1] National University «Yuri Kondratyuk Poltava Polytechnic» https://orcid.org/0000-0002-2131-2578
[2] National University «Yuri Kondratyuk Poltava Polytechnic» https://orcid.org/0000-0001-8386-977X
[3] National University «Yuri Kondratyuk Poltava Polytechnic» https://orcid.org/0000-0002-7634-5203
*Corresponding author E-mail: vlutik@ukr.net

Every year the use of vibrating and vibro-impact equipment becomes more and more common. This machinery is successfully used in the construction of solid foundations for a different building. The main element of these vibrating machines is an unbalanced vibrator. We considered the operation of the unbalanced vibrator in the interaction of mechanical and electromagnetic processes and the result was obtained as a mathematical model of dynamic processes during working the vibrator. The developed mathematical model makes it possible to carry out the analysis of the transients during the operation of the unbalanced vibrator, taking into account the inseparable interaction of the electric machine and the mechanical part of the drive.

Keywords: dynamics, mathematical model, oscillations, static moment, unbalanced vibrator, vibrating deepener, vibrating hammer, vibration

Дослідження динаміки дебалансного вібратора зі змінним статичним моментом

Лютенко В. Є.[1]*, Рудик Р. Ю.[2], Насуллоєв Ш.[3]

[1, 2, 3] Національний університет «Полтавська політехніка імені Юрія Кондратюка»
*Адреса для листування E-mail: vlutik@ukr.net

З кожним роком все більше набуває використання вібраційного та віброударного обладнання. Ця техніка успішно використовується при спорудженні надійних фундаментів під різні споруди. Основним елементом цих вібраційних машин являється дебалансний вібратор. В даний час, при проектуванні дебалансних вібраторів динамічні фактори при їх експлуатації не враховуються. Тому надійність можна підвищити, якщо на стадії їх проектування враховувати хвильовий характер навантажень. Робота дебалансного вібратора нами розглядалася у взаємодії механічних і електромагнітних процесів і в результаті була отримана математична модель динамічних процесів при роботі вібратора, котра включала нелінійні диференціальні рівняння руху мас вібратора і лінійне диференціальне рівняння електромагнітних явищ в двигуні його приводу. Можна акцентувати, що вібраційному а також віброударному методу мало приділено уваги і широка інформація практично відсутня. Тому являється актуальним створення продуктивних зразків вібраторів, методик їх розрахунків і проведення наукових досліджень динаміки робочих процесів цих машин на що і направлена дана робота. При розрахунку вібраторів на статичну й утомлену міцність коливальні процеси конструкцій та їх динамічні навантаження, в цей час, не враховуються. Однак їх несучу здатність можна значно підвищити, якщо у розрахунках при їх проектуванні враховувати їхні амплітудно-частотні характеристики. Відсутність ж уточненої методики розрахунку дебалансних вібраторів сучасних вібраційних машин ускладнює їхнє проектування і експлуатацію. Метою статті є висвітлення результатів математичного моделювання коливальних процесів при дослідженні дебалансного вібратора зі змінним статичним моментом та визначення динамічних навантажень на його елементи. В роботі теоретично досліджено, з використанням математичного програмного застосунку MathCAD, динаміку механізму привода дебалансного вібратора і отримано результати які можуть бути використані при проектуванні, розрахунку та визначенні динамічних навантажень подібних вібраторів вібраційних машин.

Ключові слова: динаміка, математична модель, коливання, статичний момент, дебалансний вібратор, віброзаглиблювач, вібромолот, вібрація

Introduction

In our time, unbalanced vibrators have become widely used, which are the basis of vibrating machines used in construction and agriculture. Unbalanced vibrators have become especially widely used in vibrating deepeners and vibrating hammers in the construction of pile foundations for various buildings and structures. But modern requirements for such vibrating machines require changing their vibration parameters during working. Therefore, the creation and research of unbalanced vibrators with variable static moments are relevant.

Review of the research sources and publications

As we noted earlier, nowadays, much attention is paid to vibration machinery, which is successfully used, especially in the construction of foundations and piles. Much attention is also paid to the improvement of vibration machineries using unbalanced vibrators. To perform the above, you should constantly study the vibration machinery and its main component - the unbalanced vibrator.

Recalling the past, it should be noted that the theoretical foundations of the creation of vibrating machines were laid in the former Soviet Union, as well as the post-Soviet society and modern Ukraine. The following prominent scientists and leading engineers, such as Artobolevsky I., Babichev A., Bliekhman I., Bykhovskyi I., Honcharevych I., Kriukov B., Lavendel E., Lanets O., Nadutyi V., Nazarenko I., Spivakovskyi A., Strelnikov L., Panovko Y., Povidailo V., Poturaiev V., Franhuk V., Sergieiev P. and others.

In the '50s, the leadership in the field of vibration machinery was for scientists in Western Europe and North America. But, for some time, domestic scientists have already been ahead of foreign scientists in the main areas.

A significant contribution to the improvement of unbalanced drives of vibrators was made by scientists of the National Mining University and the Institute of Geotechnical Mechanics Kriukov B., Franhuk V., Nadutyi V., including their supervisor, Academician Poturaiev V. [1]. Their main assets focused on the imbalance drive, which required the development of simple structures with big perturbing forces for bulky vibrating machines of the mining industry.

The creation of methods and calculation methods, including experimental samples, was carried out by scientists of the Kyiv School of Vibrotechnics Chubuk Y., Nazarenko I., Yakovenko V. [2] and others. Their created vibrotechnics were successfully used for the consolidation of concrete mixes in the construction industry.

The creation of three-mass vibrating machines was carried out by scientists and leading engineers of Lviv Polytechnic Bespalov A., Havrylchenko O., Povidailo V., Silin R., Shchihel V., Sholovii Y., Ufimtsev V. [3]. They have developed and researched a number of small and medium-sized vibrating machines for many areas where dynamic dampers have been used. The designs of these vibrating machines were three-mass, but in the calculation schemes, they were considered mainly as two-, one- mass.

It should also be noted a significant contribution to the research of oscillatory processes of mechanical systems of the following foreign scientists Kollatc L. [4], Tondl A. [5], Jagadish N. [8], Kaplan D. [9].

Until early this century, machines and structures usually had very high mass and damping, because heavy beams, timbers, castings and stonework were used in their construction. Since the vibration excitation sources were often small in magnitude, the dynamic response of these highly damped machines was low. However, with the development of strong lightweight materials, increased knowledge of material properties and structural loading, and improved analysis and design techniques, the mass of machines and structures built to fulfill a particular function has decreased. Furthermore, the efficiency and speed of machinery have increased so that the vibration exciting forces are higher, and dynamic systems often contain high energy sources which can create intense noise and vibration problems [11].

Definition of unsolved aspects of the problem

Analyzing the received information it is possible to accent that little attention was paid to the vibrating and also vibro-impact methods and the wide information is practically absent. Therefore, it is a revolt to create productive vibrators samples, methods of their calculations and conducting scientific research of the working processes the dynamics of these machines, which is the purpose of this work.

When calculating the vibrators for static and fatigue strength, the oscillatory processes of constructions and their dynamic loads, at this time, are not taken to account. However, their supporting ability can be significantly increased if their amplitude-frequency characteristics are taken to account in the calculations during their design. The absence of a refined method of calculating unbalanced vibrators of modern vibrating machines complicates their design and exploitation.

Problem statement

The article aims to highlight the results of mathematical modeling of oscillatory processes in the research of unbalanced vibrators with variable static moments and to determine the dynamic loads on its elements.

Basic material and results

Modern unbalanced vibrators and their drives, which are part of vibrating machines, have a complex structure.

We can also state about the unbalanced vibrator with the electric drive as about the electromechanical machinery. In the process, it performs a vibrating or vibro-impact action on the object. Unbalanced causes vibrating action, resulting arising in vertical centrifugal forces. We accepted for research the designed unbalanced vibrator with a variable static moment and a drive from the electric motor.

Given the fact that the vibrating mechanical system includes elastic elements, these are metal structures and a vibrator, the acting forces of which are variable in nature, we can state that during the operation of the vibromachine elastic links oscillate and create auxiliary loads.

We use the mathematical program MathCAD to simplify calculations.

Vibration load on the unbalanced vibrator, including deepening of piles is carried out by the special mechanism which is called the vibrating deepener or the vibrating hammer.

This mechanism is an electromechanical machine. It gives the pile a vibrating effect. It is installed on the pile drive and connected to the pile head.

Unbalances create a vibrating action of the deepener, resulting in vertical centrifugal forces, which are transmitted through the head to the pile. When performing the process of deepening the pile there is a destruction of the soil structure in which there are irreversible deformations.

During operation, transients take place in the drive mechanism of the unbalanced vibrator, which causes the rise of additional forces. In turn, it is necessary to note that inertial and hard parameters of links of the mechanism significantly influence passable dynamic transients.

The beginning of the movement, and also a stop of the electric drive of the mechanism of the unbalanced vibrator, is carried out at loading, that is, during its operation.

Considering the dynamic processes that take place when starting directly the drive of the unbalanced vibrator, we take the movement under load (work process).

The composite system of differential equations of motion and electromagnetic state of the electric motor has the following form (Fig. 1):

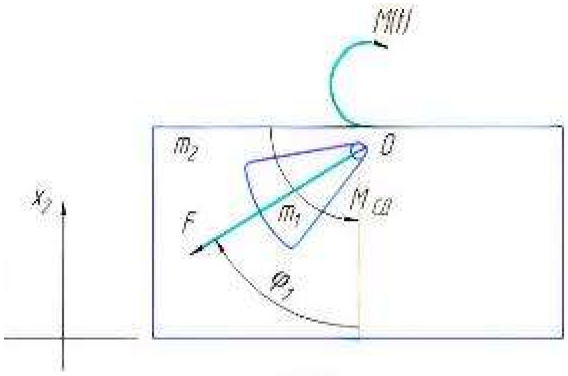

Figure 1 – Calculation scheme of the electric drive unbalanced vibrator with a variable static moment during transients

$$J_1\ \varphi_1 - m_1\ e\ x_1 \sin\varphi_1 = M(t); \tag{1}$$

$$(m_1+m_2)\ x_2 - m_1\ e\ (\varphi_1 \sin\varphi_1 + \varphi_1^2 \cos\varphi_1) = -M_{CD}. \tag{2}$$

The calculation scheme is presented in Figure 1: where J_1 – the moment of inertia of the drive rotor of the electric motor;

m_1 – the given mass of unbalances;

m_2 – the given mass of the bearing hull of the unbalanced vibrator;

Π_1, x_1, x_2 – angular and linear displacements of all two masses, respectively;

$x = 0.0174e\Pi$ – numerical dependence between angular and linear mass displacements;

e – eccentricity of the established unbalances. $e_1 = 100$ mm, $e_2 = 300$ mm;

M_{CD} – the given moment of the certain forces of resistance of imbalances;

M_{CP} – the given moment of the certain forces of resistance to the immersion of a pile;

$M(t)$ – the moment developing the electric motor.

The moment of the electric motor we are written by the following differential expression [6, 7]

$$M(t) = A_0 + A_1\ M'(t) + A_2\ x'_1\ (t), \tag{3}$$

where A_0, A_1, A_2 – engine constants.

The constants have the next form:

$$A_0 = \frac{2M_k}{S_k};\ A_1 = \frac{1}{\omega_0 S_k};\ A_2 = \frac{2M_k}{\omega_0 S_k}, \tag{4}$$

where M_k – the critical moment of the engine;

S_k – critical sliding of the electric motor rotor;

ω_0 – the angular speed of the electric motor;

t – time.

The limits of application of equation (3) have the following limitation [6, 7]

$$-0{,}8\ M_k \le M \le 0{,}8\ M_k\ . \tag{5}$$

After performing the transformation, the nonlinear equations (1–2) after the replacement and lowering the order, to be able to solve them, using the program MathCAD, will have the next form:

$$\begin{aligned}
&q'(t) = w(t)\ ;\\
&w'(t) = \frac{-0.0174e \cdot q(t) + M(t)}{J_1 - 0.0174e \cdot \sin q(t)}\ ;\\
&u'(t) = p(t)\ ;\\
&p'(t) = \frac{m_1 \cdot e \cdot w^2(t) \cdot \cos q(t) - M_{CD}}{(m_1+m_2) - 2831 \cdot \sin q(t)}\ ;\\
&M'(t) = \frac{M(t)}{A_1} - \frac{A_2}{A_1 \cdot 0.0174e} w(t) - \frac{A_0}{A_1}\ ;\\
&o(t) = \frac{-0.0174 \cdot e \cdot q(t) + M(t)}{J_1 - 0.0174 \cdot e \cdot \sin q(t)}\ ;\\
&d(t) = \frac{m_1 \cdot e \cdot w^2(t) \cdot \cos q(t) - M_{CD}}{(m_1+m_2) - 2831 \cdot \sin q(t)}\ ;\\
&F(t) = m_1 \cdot e \cdot w^2(t)\ ,
\end{aligned} \tag{6}$$

where $o(t)$, $d(t)$ – angular and linear accelerations, respectively, of the electric motor rotor and mass m_2;

$F(t)$ – centrifugal force of inertia of unbalances.

The next replacement is made:

$$\varphi_1 = q(t)\ ; \qquad \varphi'_1 = w_1(t) = q'(t)\ ;$$
$$x_2 = u(t)\ ; \qquad x'_2 = p(t) = u'(t)\ ; \qquad (7)$$
$$o(t) = w'(t) = q''(t); \quad d(t) = p'(t) = u''(t).$$

The initial conditions are as follows:

$$(t) = 0; \quad q(0) = 0; \quad w(0) = 0; \quad u(0) = 0;$$
$$p(0) = 0; \quad o(0) = 0; \quad d(0) = 0; \qquad (8)$$
$$M(0) = 0; \quad F(0) = 0.$$

The solution of the system of nonlinear equations (6) is carried out to drive an unbalanced vibrator with an electric drive, which has the parameters shown in tables 1 and 2.

When implementing the MathCAD application, we get the magnitudes of the moment of the electric motor of the unbalanced vibrator drive mechanism, angular, linear displacements of unbalances, vibrator hull, electric motor and their speed, as well as their angular accelerations (Fig. 2–7).

Constructed, as a result of the implementation of this program, graphs of changes in the moment of the electric motor of the unbalanced vibrator drive mechanism *M(t)* and the centrifugal force of inertia of unbalances *F(t)* as a function of time (Fig. 6, 7) show that the growth of inertia of unbalances F (t) continues about 3 s from the moment of its inclusion reaching, thus, the maximum value.

Considering the constructed graph (Fig. 6), we can also state that the centrifugal force of inertia of unbalanced after 4 s becomes constant.

Figure 4 shows the change in the mass movement, which can modulate the process of deep piles during the operation of the unbalanced vibrator in the mode of the vibrating deepener.

Table 1 – Parameters of the drive unbalanced vibrator

Parameters	Units of measurement	Numerical values
J_1	кг·м^2	1,275
m_2	Кг	11600
M_{CD}	Nm	50
e	M	0,1 – 0,3
v	N_s / m	25
M_{CP}	Nm	1550

Table 2 – Values of the constant electric motor drive unbalanced vibrator

Type of electric motor	Mode of operation of the electric motor	The angular speed of the electric motor rotor, rad/s
MTB 512-8 N = 50 kWt n = 720 t/min	Performance	75,4
Constant of electromotor		
A_0	A_1	A_2
67105	-0,9471	-1495

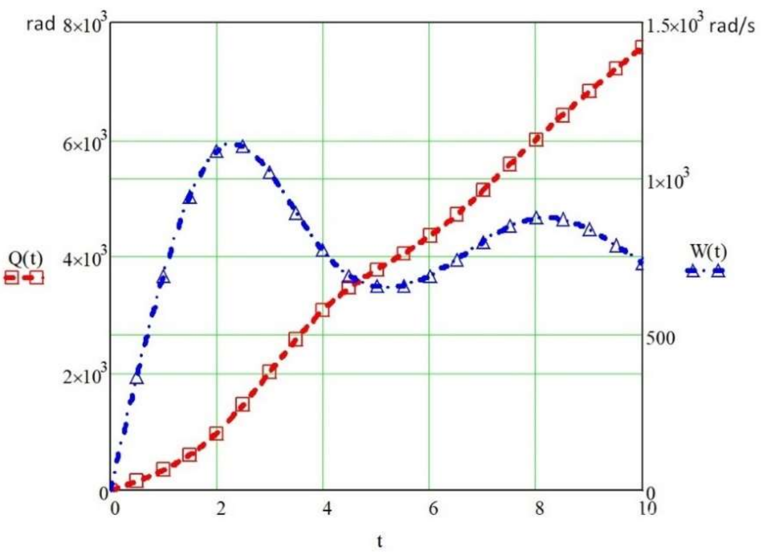

Figure 2 – Movement of mass *Q(t)* =ϕ_1 and its speed *W(t)*= ϕ_1\`=*Q\`(t)*

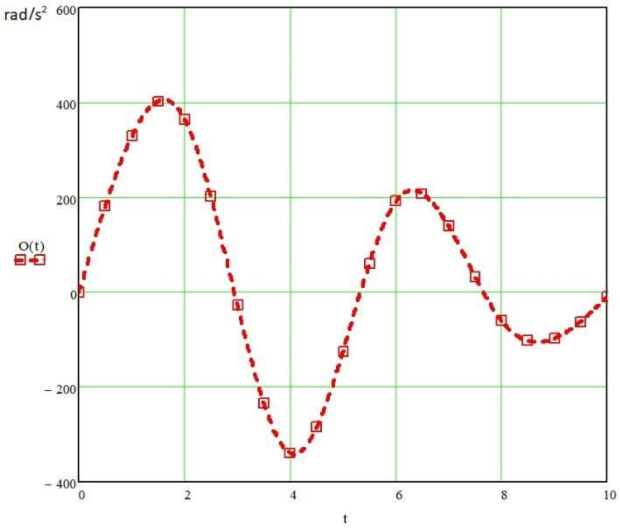

Figure 3 – Change in acceleration *O(t)* of mass *Q(t)* =ϕ_1

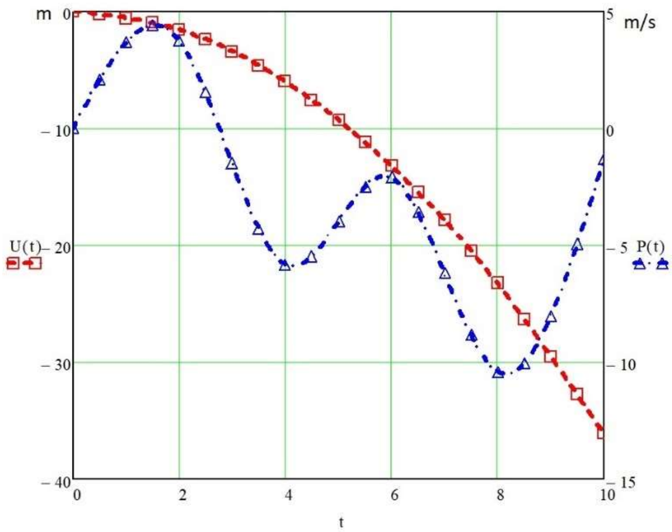

Figure 4 – Movement of mass *U(t)* = m_2 and its speed *P(t)* = *U(t)*

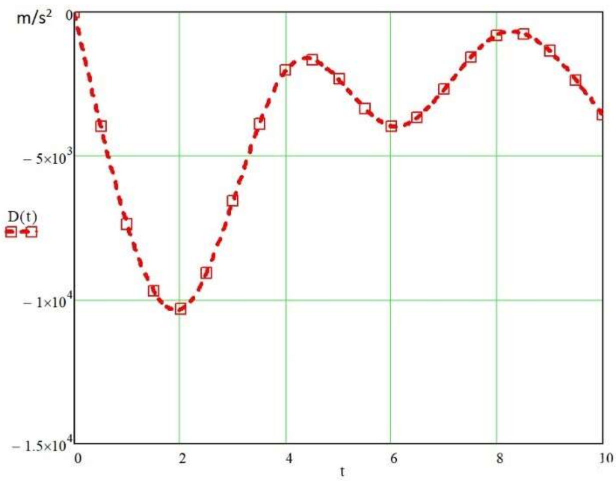

Figure 5 – Changing in acceleration *D(t)* of mass *U(t)* = m_2

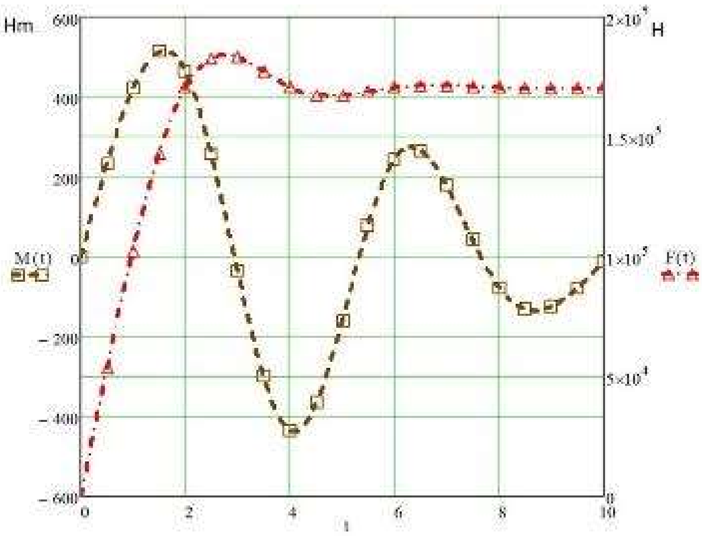

Figure 6 – Graphs of change of the moment *M(t)* of the electric motor of the vibrator and centrifugal force of inertia of unbalances *F(t)* at work (with attenuation)

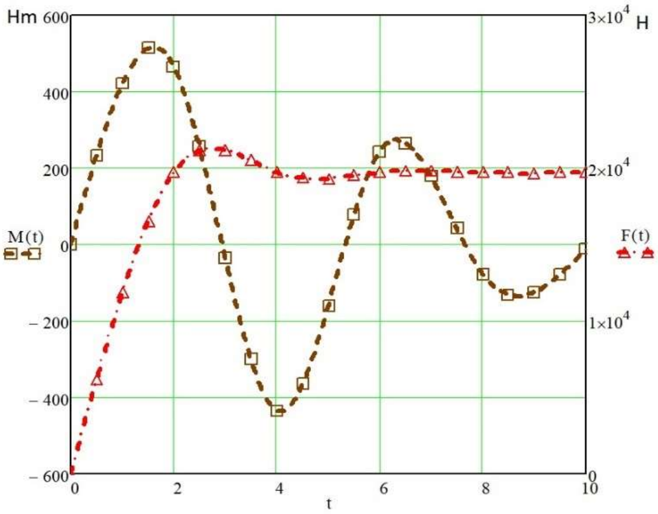

Figure 7 – Graphs of change of the moment *M(t)* of the electric motor of the vibrator and centrifugal force of inertia of unbalance ш at work (without attenuation)

The graphs constructed by us (Fig. 2 – 7) showed that nonlinear oscillating processes are observed during the operation of the unbalanced vibrator. The following information can be obtained from nonlinear equations (1 – 2):

– uneven rotation of imbalances is carried out;

– oscillations of the base of the unbalanced vibrator and the object are non-sinusoidal;

– oscillations of the angular velocity of rotation of the unbalances and oscillations of the base of the unbalanced vibrator and the object mutually affect each other;

– statement of the problem of oscillations of the basis of the unbalanced vibrator and the object is nonlinear.

At the dynamic phenomena shown in figures 2 – 7, fluctuations of angular and linear displacements, speeds, and also their accelerations are characteristic. Based on this, mathematical modeling of the unbalanced vibrator should be carried out using the equations of electromechanical interaction of the system.

The use of numerical methods for integrating nonlinear differential equations of motion and electromagnetic state expands the possibility of using the developed method to determine dynamic loads in electromechanical and mechanical systems vibrating deepener (vibrating hammer) where unbalanced vibrators are successfully used, including vibrators with the variable static moment.

Conclusions

The developed mathematical model makes it possible to carry out the analysis of the transients during the operation of the unbalanced vibrator, taking into account the inseparable interaction of the electric machine and the mechanical part of the drive.

In the work, the dynamics of the unbalanced vibrator drive mechanism are theoretically researched using the mathematical program MathCAD.

Results obtained from research of the drive mechanism of the unbalanced vibrator with using the mathematical program MathCAD can be used in the designing, calculation and determination of dynamic loads of similar vibrators of vibrating machines are obtained.

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Reflection of statistical nature of steel strength in steel structures standards

Pichugin Sergii[1*], Hajiyev Mukhlis[2]

[1] National University «Yuri Kondratyuk Poltava Polytechnic» https://orcid.org/0000-0001-8505-2130
[2] Azerbaijan University of Architecture and Construction https://orcid.org/0000-0001-6782-0941
*Corresponding author E-mail: pichugin.sf@gmail.com

The article contains a publications systematic review on the problem of construction steels strength statistical description. Mechanical characteristics of modern steels have a statistical variance, which is well described by normal law. The main attention is paid to the steels' statistical strength characteristics selection of different periods, such as mathematical expectation, standard deviation (standard), coefficient of variation, etc. The analysis confirmed the high security of normative and design resistances of low-carbon and low-alloy steels rolling profiles. The data presented in the article are intended for use in numerical structural reliability calculations. The design standards evolution for steel structures is analyzed in the justification sense of normative and design resistances and the experimental statistics involvement

Keywords: strength of steel, yield strength, normative resistance, design resistance, coefficient of homogeneity

Відображення статистичного характеру міцності сталі у нормах проєктування сталевих конструкцій

Пічугін С.Ф.[1*], Гаджієв М.А.[2]

[1] Національний університет «Полтавська політехніка імені Юрія Кондратюка»
[2] Азербайджанський університет архітектури і будівництва
*Адреса для листування E-mail: pichugin.sf@gmail.com

Стаття містить систематизований огляд публікацій у ведучих науково-технічних журналах із проблеми статистичного опису міцності будівельних сталей. Міцність сталі – це вирішальний параметр несучої здатності будівельних металевих конструкцій. Тому об'єктивне оцінювання міцності сталі має велике значення для забезпечення і розрахунку надійності конструкцій та належного обґрунтування норм проєктування. За наявності численних факторів, що впливають на міцність сталі, цілком природно, що показники міцності мають певну змінність, наочне уявлення про яку дають статистичні криві розподілу різних характеристик сталі. Межа текучості й інші механічні характеристики сучасних сталей мають статистичний розкид, котрий добре описується нормальним законом, що було неодноразово підтверджено даними випробування зразків сталі. Заводські випробування міцності сталі виконуються багато років у великих масштабах, створюючи значний масив статистичної інформації, проте загальна інформаційна база цих даних відсутня. Головну увагу приділено вибірці статистичних характеристик міцності сталей різного періоду, таких як математичне сподівання, середньоквадратичне відхилення (стандарт), коефіцієнт варіації та ін. Аналіз підтвердив високу забезпеченість нормативних опорів і кутової сталі, швелерів та балок з маловуглецевої сталі марок Ст3пс і Ст3сп. У більшості випадків виконується вимога про забезпеченість значень нормативних опорів будівельних матеріалів з імовірністю 0,95 для сталі марки Ст3. Показано, що забезпеченість розрахункових опорів профільного прокату зі сталей Ст3сп і Ст3пс завжди вища від імовірності 0,999. Наведено дані статистичної обробки результатів механічних випробувань низьколегованих будівельних сталей 14Г2, 10Г2С1, 15ХСНД, 10ХСНД та високоміцних сталей. Подані у статті дані призначені для використання у чисельних розрахунках надійності конструкцій. Проаналізовано еволюцію норм проєктування сталевих конструкцій у сенсі призначення нормативних і розрахункових опорів та залучення до цього дослідних статистичних даних

Ключові слова: міцність сталі, межа текучості, нормативний опір, розрахунковий опір, коефіцієнт однорідності

Introduction

The steel strength is a crucial parameter of the metal structures load-bearing capacity. Therefore, an objective steel strength assessment is of great importance for ensuring and calculating the structures reliability and the proper design standards justification. It is known that the steel smelting process is quite complex and not perfectly controlled (high temperature, melting process time, the content of alloying impurities, etc.). Subsequently, during rolling, the metal is compressed, the grains are crushed and their orientation along and across the rolled metal is changed, which affects the metal mechanical properties. The steel properties are also affected by the rolling temperature and subsequent cooling. In addition, with increasing rolling thickness, the metal mechanical properties decrease. In the presence of such numerous factors that affect the steel strength, it is natural that the strength indicators have a certain variability, a clear idea of which is given by statistical distribution curves of different steel characteristics. The yield strength and other modern steels mechanical characteristics have a statistical variance, which is well described by normal law, which has been repeatedly confirmed by steel samples test data. Therefore, the undoubted relevance of regular steel strength statistical studies is associated with the constant design standards revision.

Review of research sources and publications

The initial data on the steel mechanical characteristics are obtained as a result of molten steels samples standard acceptance tests in the metallurgical plants laboratories. The main purpose of these data is to assess the quality and rejection of substandard metallurgical products. In addition, steels statistical test results are used in the design standards preparation and revision. This process was especially intensified with the limit states calculation method introduction [1, 2]. Numerous publications of domestic researchers since the 1940s [3-16] are devoted to the steel mechanical characteristics statistical description, in particular its strength. This problem is actively discussed by foreign experts [17-20]. Reliable steel strength statistical parameters are especially needed to assess the metal structures reliability. This is emphasized, in particular, in the publications prepared by the scientific school "Reliability of building structures" of the National University "Yuri Kondratyuk Poltava Polytechnic" [21-24].

Definition of unsolved aspects of the problem

Steel strength factory tests are performed for many years on a large scale, creating a significant array of statistical information. However, there is no common information database for these data. Some of them have been published in various scientific and technical journals, articles collections, conference proceedings. Access to these publications is difficult, especially since some institutions have begun to destroy paper magazines in recent years, citing the transition to electronic publications. However, in reality, the translation into electronic form has so far occurred only for publications published after 2000.

Problem statement

The article contains a systematic publications review in leading scientific and technical journals on the construction steels strength statistical description problem. The main attention is paid to the statistical strength characteristics selection of different period steels, such as mathematical expectation, standard deviation (standard), variation coefficient, etc. These data are intended for use in numerical designs reliability calculations. In addition, the evolution of the steel structures design norms is analyzed in terms of changes in the purpose of normative and design resistances and the experimental statistics involvement.

Basic material and results

The article content is an organized publications review of such scientific and technical journals as "Industrial and civil construction" (formerly "Construction industry" and "Industrial construction"), "Industrial construction and engineering structures", "Construction mechanics and calculation of structures", "News University. Construction and architecture", "Building materials", "Automatic welding", etc. The review is compiled for the period from the 40s of the twentieth century to the present. The paper version was mainly used for journals published before 2000, which were in the scientific and technical library of the National University "Poltava Polytechnic Yuri Kondratyuk", one of the most complete book storages in Ukraine. Information on later editions digitized from electronic libraries and electronic versions of journals.

Low-carbon steel St3. Statistical low-carbon steel strength studies were initiated before World War II V. Kuraev under the leadership of prof. N. Streletsky [1]. Based on them, the minimum value of yield strength and allowable stress was determined for steel grade St3 equal to $[\sigma]$ =1600 kg/cm^2.

With the transition in 1955 of the steel structures calculation to the limit states method NiTU 121-55 "Standards and technical conditions for the design of steel structures" were introduced. For steel grade St3 in these standards was introduced normative resistance R^n = 2400 kg/cm^2, which was equal to the defective minimum in the steel samples acceptance tests according to the relevant GOST. The strength deviation possibility from the normative resistance in the smaller direction due to the selectivity of rolled product size control and variability was taken into account by the homogeneity coefficient k = 0.9. The design tensile, compression and bending resistance was defined as $R = kR^n = 0.9 \cdot 2400 = 2100$ kg/cm^2. The design resistance was equal to the minimum probable value of the yield steel strength, which was defined as

$$R = \bar{\sigma}_y - 3\hat{\sigma}_y, \qquad (1)$$

where $\bar{\sigma}_y$ and $\hat{\sigma}_y$ – the yield strength mathematical expectation and standard deviation (standard).

The design resistance was determined on the basis of statistical processing of steel grade St3 6 thousand factory tests results of different plants [2].

In the 60s of the last century in the metallurgical industry there were significant changes in low-carbon steel production: developed oxygen-converter smelting, mastered new deoxidation schemes (semi-quenched steel), increased open-hearth furnaces capacity, increased ingot weight. This is reflected in the next edition of SNiP II-B.3-62 "Steel structures. Design standards". They introduced two calculated supports – the yield strength of $R = 2100$ kg/cm^2 (as before) and the temporary resistance of $R_p = 2600$ kg/cm^2. It were separated open-hearth and converter steels, as well as steel deoxidation degrees: calm (sp), boiling (kp) and semi-calm (ps).

The mentioned metallurgical technology development and design norms revision had a certain influence on the steels mechanical properties. Therefore, CNDIBK carried out statistical processing of the open-hearth thick steel St3 acceptance tests results according to GOST 380-60 with a thickness of 2 – 60 mm at three metallurgical plants: Magnitogorsk Metallurgical Plant (MMK), Kommunarsky Metallurgical Plant and Ilyich Metallurgical Plant (Mariupol) [4]). The obtained results are given in table 1. These studies have shown that the rolled low-carbon steel St3 mechanical properties in these years have decreased significantly (especially in terms of yield strength and toughness). Therefore, it was concluded that the steel acceptance testing method at that time (especially the yield strength determination) needed significant improvement.

With the data given in table 1 the statistical processing results of various metallurgical enterprises steel VSt3 mechanical tests results published a little earlier are connected [3]:

– yield strength:

$\overline{\sigma}_y = 281{,}0$ MPa; $\hat{\sigma}_y = 23{,}4$ MPa; $\sigma_{y\max} = 350{,}0$ MPa;

– temporary resistance:

$\overline{\sigma}_u = 456{,}4$ MPa; $\hat{\sigma}_u = 216$ MPa; $\sigma_{u\max} = 520{,}0$ MPa.

It is known that in cases where the control tests results meet the GOST and TU standards, the consumer can get a metal with the strength characteristics values below the standard resistances. In the article [5] the deviations probabilistic analysis are considered in norms by homogeneity coefficient (coefficient of reliability on material) is executed.

In preparation for the next steel structures revision, in the late 70's CNIDBK conducted a large-scale data processing of steel St3 26 thousand acceptance tests [7], the results of which are partially given in table 1. They generally correspond to the previous tests results and confirm a smaller data statistical scatter on the temporary resistance compared to the yield strength. The statistical information resulting array was linked to the main steel structures calculation provisions at the limit states. In particular, the steel St3 normative and design resistances probabilistic provision estimation was carried out (Table 2).

Data analysis of table 2 allowed substantiating the following conclusions:

– the standard sheet metal resistances provision up to 10 mm St3ps and St3kp steels thick is low, which is explained by a significant less strong rolled steel share;

– high security of normative resistances R_{yn} and R_{un} of angle steel, channels and beams from the St3ps and St3sp brands steel;

– the requirement to ensure the building materials normative resistance values with a 0.95 probability for steel St3 in most cases is met;

– the calculated resistances values security after strength limit is higher, for which the security in all cases is close to $P \approx 1.00$, and the safety characteristic $\gamma = 5 – 9$;

– the rolled steel design resistances probabilistic provision from St3sp and St3ps steels is always higher than the 0.999 probability, with safety characteristics $\gamma = 4 – 6$. Therefore, CNDIBK proposed to increase the design resistance BSt3sp to 230 MPa and BSt3kp to 220 MPa, which was implemented during the revision of design standards.

In the new edition of SNiP II-23-81 «Steel structures» it were introduced for steel St3 two strength groups (at the suggestion of the Institute of Electric Welding named after EA Paton), grades were replaced by classes (steel St3 was assigned to classes C235, C245 and C255 depending on the degree of deoxidation and strength groups), differentiation was introduced depending on the rolling type (sheet or shaped) and the profiles thickness. To move from the normative to the design resistance instead of the homogeneity coefficient now the reliability coefficient for the material is used:

$$R_y = R_{yn}/\gamma_m\ ; \quad R_u = R_{un}/\gamma_m\ , \qquad (2)$$

where R_{yn} and R_{un} – normative resistances, respectively, after the yield strength and temporary resistances;

R_y and R_u – similar design resistances.

Substantiated statistically new reliability coefficients on the material differ insignificantly from the unit: $\gamma_m = 1.025 – 1.100$.

The CNDIBK staff article [8] summed up the results of the first SNiP II-23-81 implementation years, which led to significant savings in steel in construction. Subsequent editions of the Ukraine DBN B.2.6-198:2014 «Steel structures. Design standards» norms and Russia's SP 16.13330.2017 «SNiP II-23-81*» do not differ in principle from SNiP II-23-81 in terms of steels strength rating of [16].

Recently, the light thin-walled steel structures use has expanded. It was found that the cold steel profile formation leads to their strengthening. To detect it, the statistical processing of two steels samples test results was performed [9]. The obtained strengthening factor is well described by normal law and has the following parameters:

– 14G2 – $\overline{m} = 1.17$, $\hat{m} = 0.082$, $V = 6{,}4$ %;

– VSt3sp – $\overline{m} = 1{,}31$, $\hat{m} = 0{,}066$, $V = 5{,}0$ %.

Table 1 – Statistical data on the mechanical characteristics of sheet steel St3

Yield strength				
Steel	Date, source	$\bar{\sigma}_y$, MPa	$\hat{\sigma}_y$, MPa	V_y, %
St3kp	1968 [4]	284,1 – 310,7	21,9 –25,7	7,55
	1980 [7]	266,0	29,0	10,9
St3ps	1968 [4]	293,6 – 312,2	21,5 – 26,8	7,30
	1980 [7]	265,0 – 289,0	25,0 – 30,0	9,9
St3sp	1968 [4]	232,6 – 294,0	15,9 – 25,9	5,8 – 9,1
	1980 [7]	268,0 – 294,0	22,0 – 27,0	8,7
Temporary resistance				
Steel	Date, source	$\bar{\sigma}_u$, MPa	$\hat{\sigma}_u$, MPa	V_u, %
St3kp	1968 [4]	422,4 – 433,0	23,4 – 29,1	5,83
	1980 [7]	410,0	30,0	7,32
St3ps	1968 [4]	441,8 – 436,0	20,6 – 27,1	4,75
	1980 [7]	420,0 – 437,0	25,0 – 27, 0	6,07
St3sp	1968 [4]	417,0 – 459,0	19,2 – 23,4	5,54
	1980 [7]	433,0 – 440,0	20,0 – 25,0	5,15
Designations: $\bar{\sigma}_y$, $\hat{\sigma}_y$, V_y – accordingly average value, standard, coefficient of yield strength variation; $\bar{\sigma}_u$, $\hat{\sigma}_u$, V_u – the same as the strength limit (temporary resistance).				

Table 2 – Probabilistic provision of normative and design resistances of steel St3

Profile	Steel	Normative resistance		Design resistance			
				after yield strength		after limit of strength	
		$P(R_{yn})$	$P(R_{un})$	γ_y	$P(R_y)$	γ_u	$P(R_u)$
Sheets	St3kp	0,893	0,841	1,94	0,974	5,00	≈ 1
	St3ps	0,894-0,991	0,929-0,989	1,97-3,12	0,976-0,9986	5,92-7,07	≈ 1
	St3sp	0,921-0,998	0,984-0,996	2,15-3,82	0,984-0,9999	6,96-8,65	≈ 1
Steel angular	St3kp	0,989	0,913	3,09	0,999	5,65	≈ 1
	St3ps	0,999	0,985	4,05	0,99997	7,72	≈ 1
	St3sp	0,999	0,993	3,92	0,9998	7,07	≈ 1
Chan-nels, beams	St3kp	0,999	0,985	4,04	0,99997	7,95	≈ 1
	St3ps	0,9999	0,9996	5,24	≈ 1	8,95	≈ 1
	St3sp	0,9999	0,999	5,67	≈ 1	6,46	≈ 1
Designations: γ_y , γ_u – normalized deviations of calculated resistances from average values (safety characteristics)							

Low-alloy steels. It is no coincidence that statistical studies of the ordinary strength low-carbon steel properties were the most extensive. According to the data from the end of the 1980s, 80% of this rolled steel with yield up to 245 MPa strength was used for the building steel structures production. High strength low-alloy steels with 325 –345 MPa yield strength of were 15%, rolled high-strength steels with at least 390 MPa yield strength – only 5% [8]. Therefore, it was important to deploy research on strength steels.

Back in the 1955 – 1957 postwar period, the Chelyabinsk Plant of Metal Structures performed large-scale statistical mechanical tests of natural alloy steel NL2 (15HSND) (30 thousand tons), supplied by MMK, Kuznetsk Metallurgical Plant (KMK), Nizhny Tagil Metallurgical Plant and Plant named after Dzerzhinsky [10]. The NL2 steel yield strength distribution was well described by the normal law with the characteristics of $\bar{\sigma}_y$= 382.0 MPa, $\hat{\sigma}_y$= 27.3 MPa. The author of the publication, a well-known specialist B. Belyaev calculated the coefficient of homogeneity K_{cp} = 0.757 according to the author's method, taking into account the minus tolerances on the cross rolled sections dimensions, which gave the following steel NL2 (15HSND) design resistance value:

$$R = K_{cp}\, \bar{\sigma}_y = 0.767 \times 382 \approx 290 \text{ MPa}.$$

Therefore, a reasonable conclusion was made that the 290 MP design resistance adopted in the norms of

NiTU 121-55 is in full compliance with the actual mechanical properties of NL2 steel. However, B. Belyaev criticized the rejecting steel system, because the 340 MPa rejection minimum was at a standard 1.43 distance from the average value, which led to the probable steel 7.6% rejection. Therefore, the author of the article proposed to accept the rejection minimum at the level of 3 standards, ie 382 – 3×27.3 = 300 MPa.

In the mid-1960s, statistical processing of the mechanical tests results of low-alloy construction steels 14G2, 15 HSND, 10 HSND in the amount of 225, 575 and 507 factory tests, respectively, at MMK, NTMZ, KMK, OHMK (Orsko-Khalilovsky) and other metallurgical enterprises was performed [3]. The obtained results are summarized in a table. 3, the data for steel 15HSND differ from the previous ones [10] by a higher standard – 34.5 MPa compared to 27.3 MPa at the same average values.

A detailed low-alloy steel 10G2S1 statistical study was conducted in the late 60's B. Uvarov at the Ilyich Metallurgical Plant (Mariupol) [11]. Sheets with a 26 – 119 mm thickness were studied, the samples number was 1200. The mechanical characteristics distribution curves the were close to normal with a slight asymmetry. A decrease in the steel mechanical characteristics with increasing sheet thickness was found. This general trend was described by the following regression equations:

– average value:

$\bar{\sigma}_y = 41.3 - 0.085\delta$;

$\bar{\sigma}_u = 56.5 - 0.039\delta$;

$\bar{\sigma}_5 = 27.7 - 0.019\delta$.

– standard:

$\hat{\sigma}_y = 2.67 - 0.006\delta$;

$\hat{\sigma}_u = 2.70 - 0.014\delta$;

$\hat{\sigma}_5 = 2.29 - 0.007\delta$.

Here the strength σ is in kg/mm^2; thickness δ in mm; relative elongation σ_5 in %.

The yield strength standards and temporary resistance decrease with increasing thickness due to the mechanical properties alignment with slow thicker sheets cooling.

The homogeneity coefficient was determined in the usual way

$$k = \frac{1-\gamma\sqrt{V_y^2+V_f^2}}{1-\gamma^2 V_f^2},$$

where γ – safety factor (accepted in the norms equal to 3);

V_y – yield strength variation coefficient;

V_f= 0.043 – variation coefficient by area.

After substituting the numerical values into the formula, the homogeneity coefficient was obtained k = 0.79. The formula for the design resistance was obtained by the regression line equation

$$R = 32.6 - 0{,}068\delta$$

It turned out that with increasing sheet thickness for every 15 mm, the design resistance decreases by 10 MPa. This was taken into account in table 4.

The recommended rolled products division into groups narrower than the norms could have some economic effect, but was not fully implemented.

In the early 80's, experts from the Moscow Institute of Civil Engineering (MIBI) conducted high-strength steels statistical studies [12]. Data on strength class C70/60 steel 12GN2MFAYU were obtained by the acceptance tests results at OHMK, the sample size – 4 thousand tests. Sheet metal with a 12 – 40 mm thickness was tested.

The obtained results: the average yield strength value $\bar{\sigma}_y$ = 710,4 MPa; temporary resistance $\bar{\sigma}_u$ = 806,4 MPa; average elongation $\bar{\sigma}_5$ = 16,11%. Steel within the batch is heterogeneous (327 tested batches): the properties distribution standard within the batch in the general distribution standard shares is: 0,53 after the yield strength and 0,48 after the temporary resistance.

The investigated rolled metal meets the requirements for steel of class C 70/60: $\int_y$ ε 60 MPa; $\int_u$ ε 70 MPa; ™$_5$ ε 12 %. According to the test results, high-strength steel grade 12GN2MFAYU can be considered promising for responsible welded metal structures operating under dynamic loads and operated at negative temperatures below -40 ° C.

Table 3 – Statistical data on the mechanical characteristics of low-alloy steels

Steel	Yield strength σ_y, MPa			Temporary resistance σ_{uy}, MPa		
	$\bar{\sigma}_y$	$\hat{\sigma}_y$	$\sigma_{y\,max}$	$\bar{\sigma}_u$	$\hat{\sigma}_u$	$\sigma_{u\,max}$
14G2	398,8	36,0	510,0	552,0	38,6	670,0
15XSNДD	389,2	34,5	500,0	562,4	30,0	660,0
10XSNДD	458,7	37,6	580,0	597,5	34,6	710,0

Table 4 – Recommended sheet steel design resistances

Thickness, mm	До 38	39 –52	53 – 68	69 – 82	83 – 98	99 – 110
R, MPa	300	290	280	270	260	250

The new high-strength steel properties statistical analysis with nitride hardening grade 16G2AF was performed at OHMK on the basis of a 6.5 thousand tests sample [13]. Sheet metal with a thickness of 10 – 40 mm was tested. Steel in the normalized state had an yield strength average value $\bar{\sigma}_u$ = 470 MPa; temporary resistance $\bar{\sigma}_u$ = 650 MPa. Heat-treated steel had slightly higher characteristics – the yield strength average value $\bar{\sigma}_y$= 550 MPa; temporary resistance $\bar{\sigma}_u$ = 680 MPa. Steel within the batch is heterogeneous (816 batches were tested): the normalized steel properties distribution standard within the batch in the general distribution standard shares is: 0,518 after the yield strength and 0,607 after the temporary resistance. It was concluded that the developed steel in terms of both strength and plastic characteristics meets the requirements for high-strength steels.

Recent publications describe new high-strength steels of large thickness [14, 15]. Rolled steels C345, C375, C390 and C440 have high engineering properties and good weldability. Thermomechanical high purity hardened steels can be referred to the third construction steels generation and be used in the most responsible and unique building metal structures.

Conclusions

A systematic works review on the problem of construction steels strength statistical description is realized. The main attention is paid to a sample of different periods steel strength statistical characteristics, such as mathematical expectation, standard deviation (standard), variation coefficient, etc. These data are intended for use in structures reliability numerical calculations. The steel structure design norms evolution is analyzed in terms of changes in the purpose and provision of normative and design resistances and the experimental statistics involvement.

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Structural system collapse risk limitation strategy

Pichugin Sergiy[1]*, Klochko Lina[2]

[1] National University «Yuri Kondratyuk Poltava Polytechnic» https://orcid.org/0000-0001-8505-2130
[2] National University «Yuri Kondratyuk Poltava Polytechnic» https://orcid.org/0000-0002-6064-2887
*Corresponding author E-mail: pichugin.sf@gmail.com

This article presents a general strategy for limiting the structural system risk collapse a using the real construction object example. In the course of the work, an analysis of possible accident scenarios at a construction site was carried out. A statistical data analysis is presented to create the most systematic method for the possible accident scenario in construction. The article presents the calculations results of the building frame spatial model for progressive structure destruction to determine the accident possibility. When changing the monolithic floor geometry and removing the columns, the failure probability of the supporting structures and the consequences to which this accident could lead were considered. Also, in this work, the economic consequences question is raised. The scheme calculation result is presented and the corresponding conclusions are drawn

Keywords: statistics, modeling, accident, building, probability of an accident, destruction, consequences, systematization

Стратегія обмеження ризику обвалення конструктивної системи

Пічугін С.Ф.[1]*, Клочко Л.А.[2]

[1] Національний університет «Полтавська політехніка імені Юрія Кондратюка»
[21]Національний університет «Полтавська політехніка імені Юрія Кондратюка»
*Адреса для листування E-mail: pichugin.sf@gmail.com

Наведено загальну стратегію обмеження ризику обвалення конструктивної системи на прикладі реального будівельного об'єкта. Проаналізовано різноманітні підходи до розрахунку можливості виникнення аварії або, іншими словами, ймовірності відмови тієї чи іншої конструкції. Надано результати розрахунку виникнення можливої відмови конструкції будівельного об'єкта на прикладі створеної моделі. У ході роботи був проведений аналіз сценаріїв можливого виникнення аварії на будівельному об'єкті. Виділяються й описуються характерні особливості найбільш систематизованого алгоритму сценарію можливого виникнення аварії в будівництві, для якого має велике значення збір даних про аварії, що виникли, їх систематизація та осмислення, тому що подібна практика збільшує шанси на безаварійне будівництво. Значну увагу приділено підходу до опису аварії, який може бути розглянутий із її вірогідності. Надано класифікацію аварій за ймовірністю їх виникнення. Розрахунок ризиків прогресуючого обвалення розглядався при певному переліку загроз. Наведено результати розрахунків просторової моделі каркаса будівлі на прогресуюче руйнування конструкції для визначення можливості виникнення аварії. При зміні геометрії монолітного перекриття та вилученні окремих колон розглядалася ймовірність відмови несучих конструкцій і наслідки, до яких така аварія може призвести. У результаті чисельного моделювання отримано якісну оцінку характеристик стійкості конструкції стосовно прогресуючого руйнування. Розглянуто найпростішу схему конструкції будівлі, за якою відома ймовірність відмови об'єкта. Для підрахунку можливих матеріальних збитків і (або) соціальних втрат від відмови об'єкта, пов'язаних із припиненням експлуатації чи втратою його цілісності, визначалися найбільш імовірні прогнози можливої аварії, що сталася з техногенних або природних причин.

Ключові слова: статистика, моделювання, аварія, будівля, імовірність аварії, руйнування, наслідки, систематизація

Introduction

Nowadays, the failure probability calculation and predicting accidents at the construction site design stage is being introduced with government regulations in developing countries. This fact makes it clear the need to develop an appropriate methodology in this matter. Studying the historical experience of creating various algorithms types, it is necessary, in turn, to improve them. The reasons for this are the technical and informational development of the construction industry as a whole, the increase in the complexity of engineering tasks, the concept renewal of architectural forms and buildings purposes. [1].

In connection with the approaches spread to predicting the construction objects accidents in the design documentation, various approaches are being created to the development and situation modeling building fail individual structural elements. The main task at the moment is to create an algorithm for modeling and calculating such emergency situations and the creation of uniform building codes on this issue.

Review of the research sources and publications

Speaking about the accidents analysis in construction, it is worth noting such scientists as Belyaev B.I. [2], Laschenko M.N. [3], Sakhnovsky M.M. [4], Shkinov F.N. [5], Perelmuter A.V. [6] and other authors, who in turn also paid considerable attention to the accident statistics development and tried to classify them. For example, a fairly detailed accidents statistic for 2009 was presented in the Eremin K.I. work [7]. The accident possibility calculation, or in other words, the particular structure failure probability, was carried out by such scientists as Raiser V.D. [8, 9], Pichugin S.F. [10 11], Semko A.V., Voskoboynik O.P. [12] and others [13].

Stewart M.G., Melchers R.E. dealt with an engineering system probabilistic risk assessment issues [14]. Milchers R.E in his works raised the reliability and forecasting analysis issue in the construction industry [15]. It is also advisable to pay attention to the works of such scientists as Ellingwood B.R., Smilowitz R., Dusenberry D., Duthinh D. [1] in whose works the buildings progressive destruction issue was raised.

A striking example of the legal basis creation for calculating progressive collapse is the first rules set edition, entitled "Protection of buildings and structures from progressive collapse. Design rules [16].

Definition of unsolved aspects of the problem

To date, an important issue is the development of a methodology and algorithm for analyzing the predicting probability the accident possibility in construction.

Problem statement

The work purpose is to determine a general strategy for limiting the danger (risk) of structural system collapse, to reveal the concept of a possible accident scenario at a construction site and to calculate the possible failure occurrence of a construction site using the created model example.

Basic material and results

General strategy for limiting the hazard (risk) of structural system collapse

When designing in construction, an important factor after the structure reliability is its financial side of the issue, that is, the direct construction cost. Ensuring structural reliability must be provided by cost-effective design solutions. In order to the costs invested in the structure safety to be justified, a risk assessment should be carried out, the magnitude of depends on the failure probability and its consequences. Thus, risk analysis consists of two independent tasks: determining the failure probability and assessing its consequences.

Calculating the risks associated with the structural system collapse, in particular when exposed to special (abnormal) influences, the most difficult in all respects life and health valuation of people exposed to potential threats.

The progressive collapse risks calculation is considered with a certain threats list, including in the general case:

1) pathological, special effects (natural or errors in design and construction);
2) violation of operation;
3) object crashes.

The general strategy for limiting the danger (risk) of the structural system progressive collapse should contain the following stages:

1) risk assessment and probabilistic formulation of structural criteria (for example, the structural elements destruction probability is allowed);
2) impact damage characteristics;
3) strategy development to limit threats from special impacts;
4) implementation in professional practice.

As can be seen from the general strategy description for limiting the structural system progressive collapse danger, the first and one of the main strategy elements is the risk assessment and, accordingly, the probabilistic structural criteria formulation for further calculation.

The concept of the possible accident scenario at the construction site

To calculate the possible material losses and (or) social losses from the facility associated refusal with the operation termination or with the integrity loss, the most probable predictions of a possible accident are determined (for example, damage, failure, building destruction, structure, linear engineering and transportation facility, infrastructure or their parts) that occurred due to man-made or natural causes. Potential losses are assessed based on the predicted accident scenario, taking into account the measures provided for by the project to localize a possible accident (for example, dividing the construction object into separate parts).

It is recommended to consider the possibility of the following events, for example:

- failure and destruction of a separate supporting structure due to its overload in excess of combinations loads and effects;
- the occurrence of soil foundations large subsidence during their emergency soaking;

- the possible karst sinkhole impact, landslides, etc .;
- the impacts from vehicles collisions;
- the structures failure possibility in the fire event;
- damage to building structures by accidental explosions (for example, household gas);
- the technological regulations violation possibility or damage to equipment (pipeline ruptures, falling loads, others for design impacts) [10].

For s high-rise apartment buildings and structure, the hypothetical collapses listed in clause E.1.2 of DBN V.2.2-24:2009 should be considered as initiating events [17].

Potential social losses from abandonment should be weighed against risk factors such as:
- danger to people health and life;
- a sharp ecological situation deterioration in the territory adjacent to the object (for example, when the storage of toxic liquids or gases is destroyed, the sewerage treatment facilities fail, etc.);
- history and culture monuments loss or other spiritual society values;
- termination of the communication systems and networks functioning, power supply, transport or other elements of population or public safety life support;
- impossibility to organize assistance to accidents and natural disasters victims;
- the threat to the country's defense.

Possible economic losses should be assessed by the costs associated with both the need to restore the facility and with incidental losses (losses from stopping production, lost profits, etc.).

The possible consequences characteristics are the basis for the construction objects classification according to three consequences (responsibility)classes – CC1, CC2 and CC3.

When calculating structures, the following design situations types should be considered:
- established, for which the implementation duration of Tsit is the same order as the established service life of the construction site Tef (for example, the operation period between two major repairs or changes in the technological process);
- transitional, for which the implementation duration Tsit is small compared to the established service life Tef (for example, the object construction period, major repairs, reconstruction);
- emergency, which is characterized by a low probability of Psit occurrence and, as a rule, a short duration of Tsit << Tef implementation, but which are quite important from the point of view of the possible failures consequences (for example, situations that arise during explosions, fires, equipment failures, collisions vehicles, as well as immediately after the any structural element failure). [10].

Construction accident statistics

To create the most systematic method for the possible accident scenario in construction, it is of great importance to collect data on accidents that have occurred, their systematization and understanding, since this practice increases the chances of accident-free construction.

Without pretending to cover the problem as a whole, we can single out the most common buildings destruction cases, as: engineers' errors in calculations; builders' negligence in the facility construction, misuse or improper reconstruction, the incidence of which has increased significantly over the past few years.

A sudden collapse leads to a prolonged the building failure, the fires outbreak, the utilities and energy networks destruction, the blockages formation, injury and people death [18].

Using the example of India, we can give the corresponding figures for the statistics of construction accidents victims. India's National Crime Bureau (NCRB) data indicate that between 2001 and 2015, 38,363 people died in the destruction of various buildings. Most of the people died as a result of the residential buildings collapse. Uttar Pradesh recorded the largest number of deaths during this period (5690) [19].

Accidents should also be divided by the class of consequences, in accordance with the National Standard of Ukraine [20]. Taking into account the conducted research, the most common structures accidents can be considered objects with the class of consequences CC2, including residential buildings with the number of people who are constantly in the building, up to 400 people.

The methodology for collecting, processing and presenting statistical information is an almost independent section of probabilistic calculation methods (for example, the theory of building elements reliability and systems). In this case, the theory should specifically determine the quality and quantity of information required for practical use in calculations.

The building materials properties and elements can be represented by parametric distribution laws. This assumption can be substantiated by the fact that the materials and structures properties do not change significantly over time and, as a rule, have a corresponding tendency. The foregoing should be understood in such a way that possible changes in properties are, as a rule, not random in nature, only the numerical value of this property has a random character.

The next part of the statistical information processing is a more detailed division of the resulting general table by the objects type that have collapsed. For example, the accidents types can be divided into three components: buildings and structures destruction at the stage of construction and acceptance into operation, during reconstruction of facilities and accidents due to the large building age. The classification based on these characteristics is due to the high level of their frequency in the course of the issue study, from which it follows that the appearance of such an accident is the highest.

On the research basis carried out, graphs and diagrams are created that reflect the results obtained, on which final conclusions are already drawn.

An example of generalized data processing is the annual accidents statistics created by the Russian company "City Center for Expertise". A work feature of this company is its transparency and results publicity. The statistics provided over the past few years are freely available on the Internet, with the components of which anyone can get acquainted. At the same time, citizens do not have access to official statistics, which are kept by government. Based on this, there is a need to address the issue of transparency in the commission's work the for the accident investigation in buildings and structures.

The possibility of providing public information can be a significant step in resolving the accidents issues that occurred during the construction phase, since the incidents publicity and the work results carried out by a special commission will become a great impetus for the certain accidents type elimination.

In addition, the statistical data processing on accidents at construction sites makes us pay attention to the problem of old buildings that are being decommissioned but not dismantled in the future. Most often, the authorities do not pay attention to their accident rate and the highest collapse probability. Long-term dismantling, and in most cases, its complete absence, can result in human lives.

If until sometime the accident was considered as a probabilistic event that has no regularity, and the results of which cannot be predicted, then at the present stage scientists have made a tangible breakthrough in this knowledge area. With the concepts introduction such as economic and non-economic consequences, the development and implementation of possible losses calculations, depending on the particular structure failure, begins.

The approach to describing an accident can be considered with its probability. That is, an accident can be probable, impossible, or accidental (Fig. 1).

Let us give examples of situations in which an accident is probable. In this case, this is an accident that occurred near the city of Mumbai, April 6, 2016 [21]. The seven-story residential building collapse provoked a number of reasons, such as construction standards violation, negligence during construction, and the construction work illegality. The occurrence probability was the highest in this case.

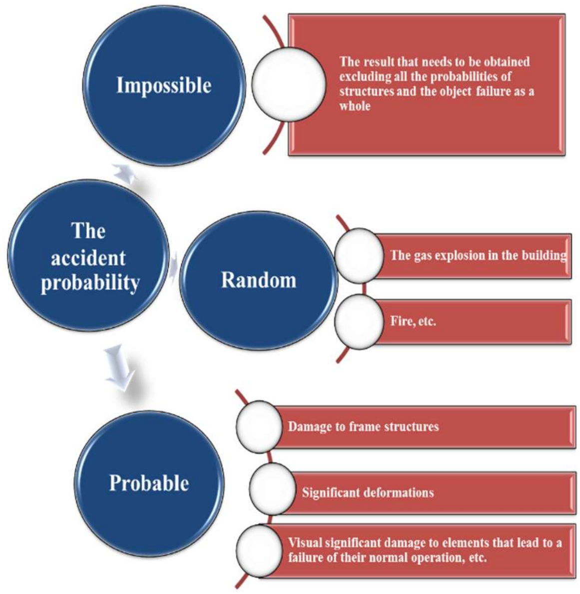

Figure 1 – Classification of accidents with the probability of their occurrence

Accidental accidents include a gas explosion in a residential building in Brussels, which took place on March 18, 2017, as a result of which one person died [22]. One building collapsed completely, only the facade remained from the other. Or the fire that happened on February 21, 2015 in the UAE, where the high building "Torch" lit up [23]. No harm done.

The result of the accidents analysis that occurred in construction should be the accident impossibility. A striking example of the past years' experience processing, the implementation of the necessary improvements and prevention of various accidents types is the modern complex "Federation", which consists of two skyscrapers with a height of 324 meters [8].

Constructive solution and calculation of a construction object

For a more detailed consideration of the modeling possible accident occurrence issue, we will calculate a real construction object, namely, an industrial building (fig. 2).

The frame design was carried out by the finite element method, taking into account the following provisions:

a) introduction of three-line deformation diagrams for concrete and two-line diagrams for reinforcement;

b) the frame structure is considered as a system of frames with rigid nodes, which are located in two mutually perpendicular directions;

c) the non-girder floor structure was calculated for the load, evenly distributed over the entire floor or part of it;

d) the structures were calculated for strength, deformability and crack opening under the static load action.

The structure design model has been created and is shown in Fig 3.

Relying on progressive destruction

In the building structures calculating for progressive destruction in the LIRA software package, the following calculation stages were implemented [24]:

1) performed linear calculation with the determination of the frame deformation (Fig. 4);

2) exclusion from work in the structural scheme of individual load-bearing elements (columns 10, 11, 15), the calculation of the scheme, as a result of which reinforcement is assigned for the model calculation in the non-linear phase;

3) the building calculation taking into account physical and geometric nonlinearity and dynamic coefficient.

It should be noted that at the third calculation stage, the criteria for the structures destruction are the geometric system variability at the nth step, an avalanche-like growth of deformations and the system displacement.

To determine the most dangerous structure section in the spatial building model, the number and location of supporting structures (columns) were experimentally changed (Fig. 5–7).

When the K-15 column was removed from the general scheme, the deformation frame calculation was carried out again. The calculation result is shown in Fig.5.

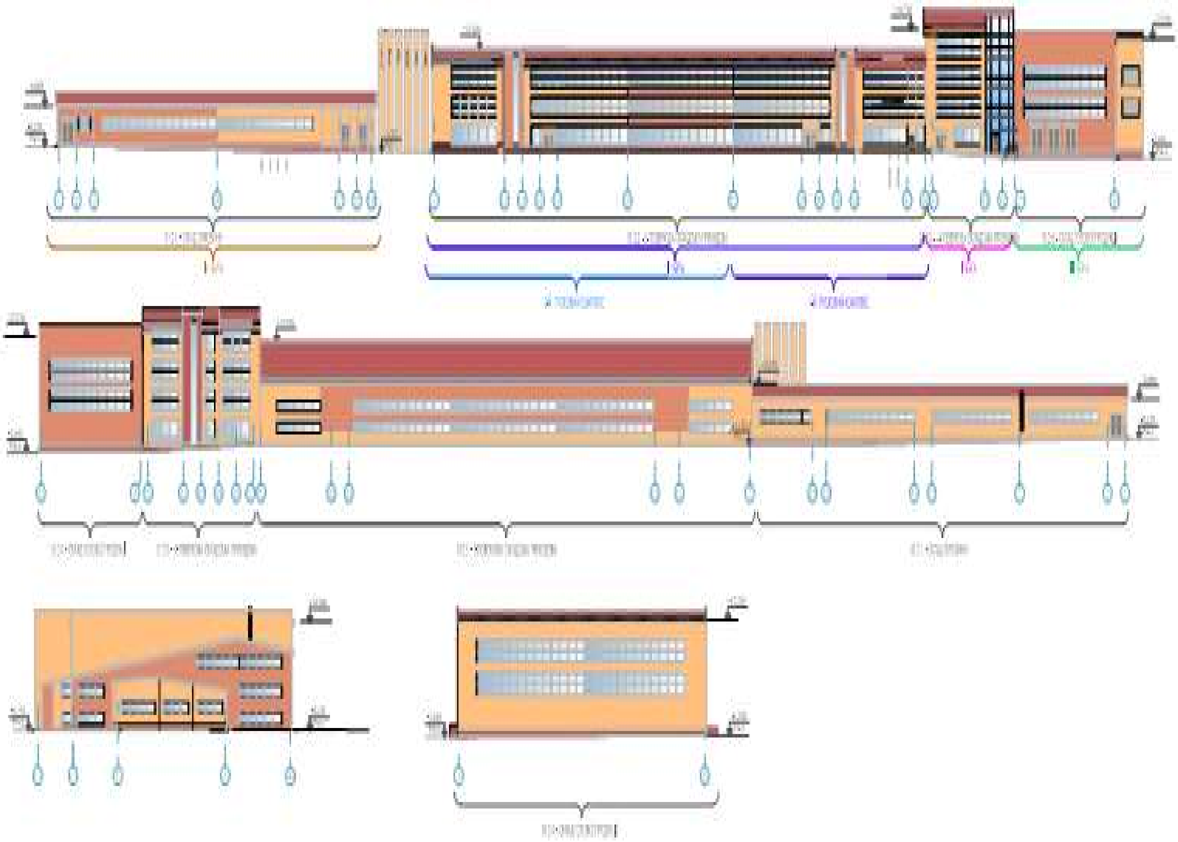

Figure 2 – Factory buildings facades

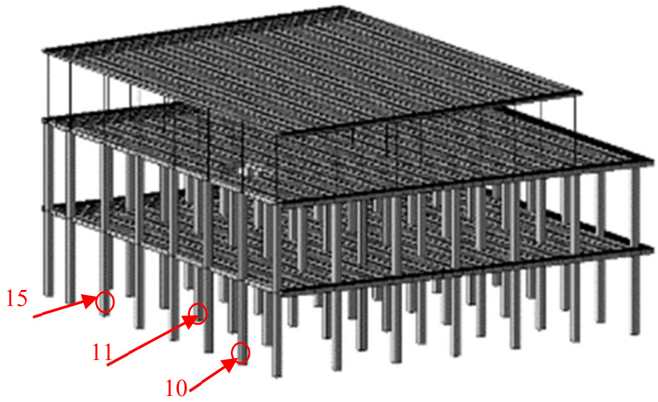

Figure 3 – The design model of the designed structure, with the given structural elements

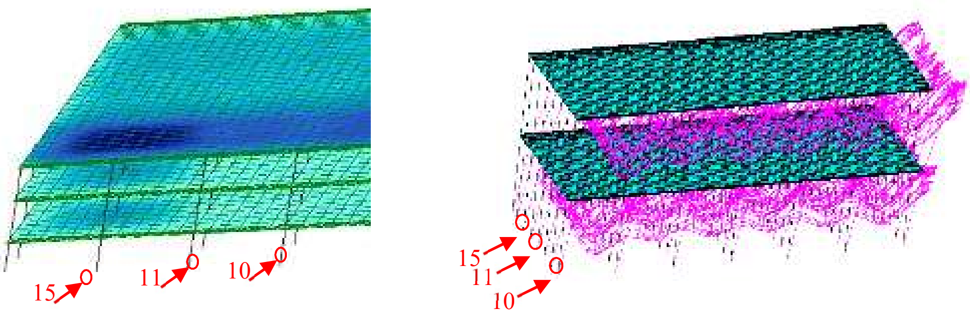

Figure 4 – Results of building deformation analysis

A similar frame calculation for deformation was performed when the column K-10 (Fig.6).

Also, the calculation of the frame was carried out when the middle column K-11 was excluded from the work, which made it possible to carry out further analysis of the progressive failure effect on the structure (Fig. 7).

Situations were also checked when changing the solid floor geometry, by removing the structure timewattle (Fig. 8).

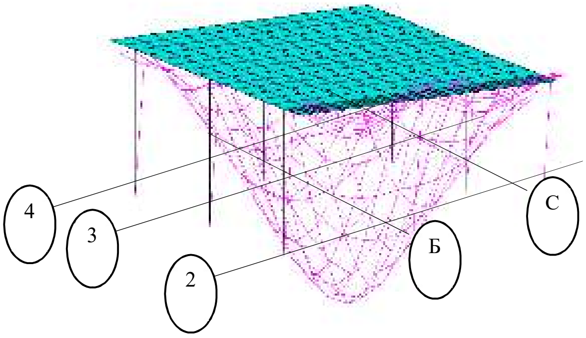

Figure 5 – The frame calculation result on deformation with the decommissioning of the middle column K-15

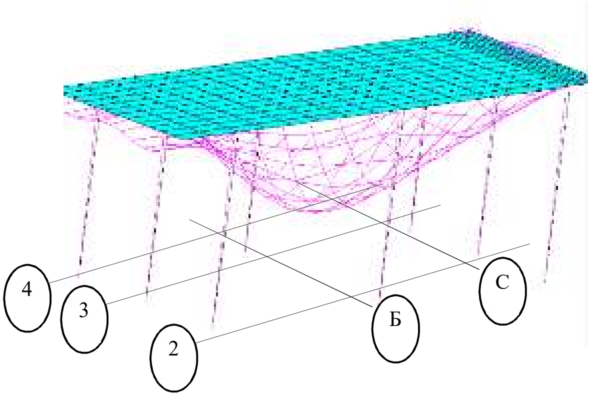

Figure 6 – The result of calculating the frame on deformation with the decommissioning of the second row column K-10

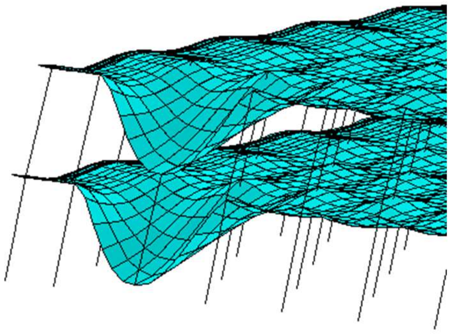

Figure 7 – The result of the calculation of the frame on deformation with the decommissioning of the extreme column K-11

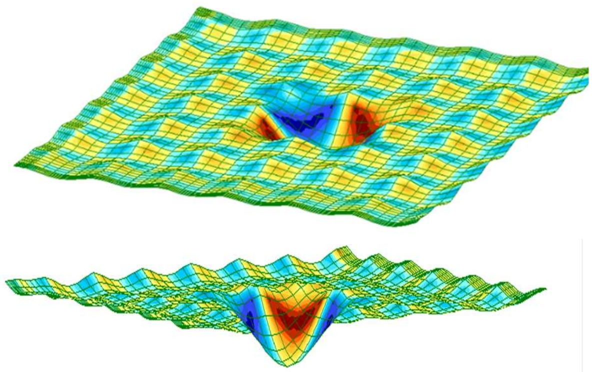

Figure 8 – The results of the design calculation in the accident simulation - removal of the K-15 column (Isofield of stresses on moss)

Conclusions

The prerequisites for the implementation of the methodology and algorithm for modeling the accident possibility at a high-risk construction site gave impetus to the development of such a direction in the scientific activity of the construction industry as predicting the progressive structures destruction.

It is important to take into account that the calculation progressive destruction risks directly considers a number of threats, among which it is advisable to single out the main ones, such as abnormal ones, violations in operation and failures of a construction facility.

At the same time, the main task for our time is to determine the main strategy for limiting the risk of a particular accident.

As a numerical modeling result, it is possible to obtain a qualitative characteristics assessment of the structure stability in relation to progressive destruction, as well as to compare several possible scenarios of destruction in order to identify the weak structure points.

The calculation result is the forces, stresses and displacements at each of the load application stages, cracks patterns in walls and slabs, places of plastic hinges occurrence, information about the elements that are destroyed in the first place. It is also possible to determine the load at which the first structural element collapses and from it to conclude the available reserves in bearing capacity terms.

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Strength analysis of reinforced concrete in a closed space of a metal pipe

Pavlikov Andriy[1], Kochkarev Dmytriy[2], Harkava Olga[3], Andriiets Kateryna[4*]

[1] National University «Yuri Kondratyuk Poltava Polytechnic» https://orcid.org/0000-0002-5654-5849
[2] National University of Water and Environmental Engineering https://orcid.org/0000-0002-4525-7315
[3] National University «Yuri Kondratyuk Poltava Polytechnic» https://orcid.org/0000-0003-2214-3128
[4] National University «Yuri Kondratyuk Poltava Polytechnic» https://orcid.org/0000-0001-5636-5568
*Corresponding author E-mail: kate.andriec@gmail.com

The concrete strengthening coefficient calculating method of concrete-filled steel tubular members at axial compression based on the plasticity theories by Saint-Venant and Huber-Mises-Genk. This method is used for calculating values taking into account the concrete meridional pressure on the pipe and axial stresses in it. The following tubular concrete element strength equation is obtained, where the stresses in the pipe reach the limit values at the element destruction moment, that is the materials strength is used completely. An calculation example is given and the strengthening coefficient calculation results are compared according to both theories. For determining the proposed method accuracy it is planned to compare the results of the calculation with experimental data.

Keywords: concrete, pipe, plasticity, steel, strength.

Аналіз міцності залізобетону в замкнутому просторі металевої труби

Павліков А.М.[1], Кочкарьов Д.В.[2], Гарькава О.В.[3], Андрієць К.І.[4*]

[1, 3, 4] Національний університет «Полтавська політехніка імені Юрія Кондратюка»
[2] Національний університет водного господарства та природокористування
*Адреса для листування E-mail: kate.andriec@gmail.com

На сьогодні проєктування стиснутих трубобетонних елементів являє собою досить складний процес, причиною чого є відсутність розрахункових залежностей, які б враховували явище збільшення несучої здатності елемента за рахунок роботи бетону в умовах об'ємного напружено-деформованого стану, що обумовлює одержання розрахункових значень міцності нормального перерізу трубобетонного елемента, які будуть недовикористані. Можливість розв'язання існуючої проблеми полягає в подальшому дослідженні теорії розрахунків міцності трубобетонних елементів на основі впроваджуваних сучасних поглядів на роботу бетону в поєднанні з арматурою та сталевою оболонкою. Тож було розроблено методику аналізу міцності з використанням коефіцієнта зміцнення бетону трубобетонних елементів при осьовому стисненні для обчислення значень з урахуванням меридіонального тиску бетону на трубу й осьових напружень у ньому на основі теорій пластичності Сен-Венана та Губера – Мізеса – Генкі. Критерієм руйнування трубобетонного елемента обрано його граничний стан при досягненні в сталі труби напружень текучості, завдяки чому отримано таке рівняння міцності трубобетонного елемента, в якому напруження в стінці труби в момент руйнування елемента приймаються граничними, тобто міцність матеріалів використовується повністю. Наведено приклад виконання розрахунку та здійснено порівняння результатів розрахунку міцності з використанням коефіцієнта «зміцнення» за обома теоріями як у табличній формі, так і шляхом побудови графіків залежності значень міцності трубобетонного елемента й коефіцієнта зміцнення бетону від його класу. Відмічено суттєве зростання коефіцієнта зміцнення для низьких класів бетону при відповідному збільшенні товщини стінки труби за обома теоріями, що може свідчити про суттєві резерви міцності та потребує подальшого теоретичного й експериментального дослідження.

Ключові слова: залізобетон, сталь, труба, міцність, пластичність.

Introduction

Currently, many experimental and theoretical studies have been conducted on the composite elements resistance with concrete-filled circular steel pipe sections to longitudinal (axial) compression. The occurrence of the strengthening phenomenon arising in concrete, which filled the pipe, was proved by the studies results. This is mainly due to the deformation limiting artificially created conditions by the outer tube-shell. Also, a way of applying an external load to the concrete-filled steel pipes has not gone unnoticed by researchers in this aspect. The first case is the load application to the pipe and concrete at the same time [1] and as the second case is considered the situation when the load is applied only to the concrete [2, 3]. The adhesion level of the contact "tube – concrete" was taken into consideration as well as the destruction criteria (the first – for the yield stress achievement in the pipe, the second – the state of concrete element complete destruction).

It is known that much research has been done on these issues, but despite this, there is still a wide contradictions range in ideas about the pipe and concrete core joint work. Obviously, the proposed analyzing methods of composite steel and concrete elements strength are significantly affected by these contradictions directly.

Review of the research sources and publications

In this study, as the criterion for the destruction of the composite steel and concrete member the second operation state is adopted. This ultimate limit state is declared the main one in the norms [4, 5]. Widespread introduction in the modern analysis methods of building structures of the ultimate state concept based on the manifestations of composite materials plastic properties [6, 7] sufficiently substantiates the non-availability of need to use the first condition (criterion) for calculating the concrete-filled steel tubular members strength. The number of different methods is constantly increasing, in particular it is observed in the norms of the USA [8, 9], Canada [10], England [11], and Europe [12].

Therefore, this article presents a possible solution to the existing problem in the analysis the composite members strength theory, which is based on the introduced modern views on the concrete work in combination with reinforcement and steel shell [13 – 18].

Definition of unsolved aspects of the problem

From the above contradictions follows an important composite steel and concrete elements designing problem, it is the structural dependencies lack, in terms of concrete volumetric stress-strain state would clearly distinguish the strengthening component. A large number of empirical methods for analyzing the compressed concrete-filled steel tubular members strength have been proposed to eliminate this gap. Unfortunately, their main imperfection is the empiricism accumulation, which does not contribute to a deep understanding of the tubular concrete elements composites complex work.

Problem statement

The aim of this paper is to obtain an analytical expression for the concrete strengthening coefficient calculation of the concrete-filled steel tubular member core at the complete destruction moment and the element strength equation, where the stresses in the pipe would reach the limit values at the element destruction time, in case the materials strength completely using.

Basic material and results

Under the action of axial compression, the normal cross section strength of a cylindrical concrete-filled steel element, taking into account the concrete meridional pressure p on the tube (shell) and the meridional (axial) stresses σ_{s1} in it, will be ensured if condition (1) is fulfilled (figure 1).

$$N_{Ed} \le N_{Rd} = A_c\left(f_c + 4 \cdot p\right) + A_s\sigma_{s1}, \qquad (1)$$

where A_c is the concrete cross-sectional area in the pipe, A_s is the pipe cross-sectional area, f_c is the limit stress value in the concrete at its destruction.

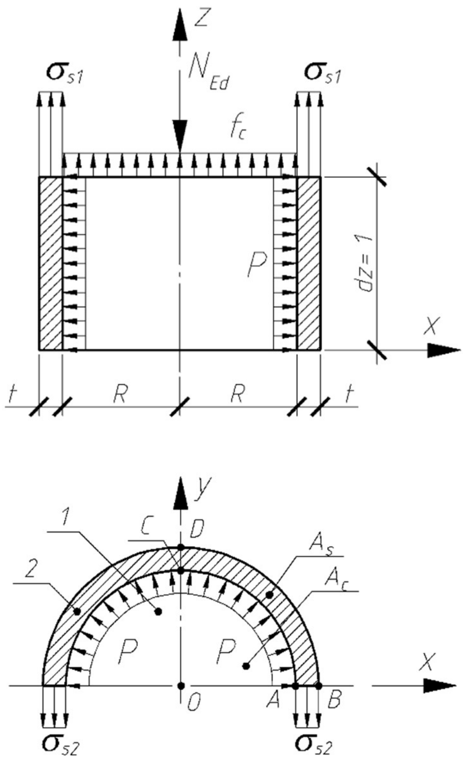

Figure 1 – Design diagram of concrete-filled steel tubular member deformation mode:
1 – concrete; 2 – tube (shell)

The right equation side (1) is a well-known expression [6], which only partially takes into account the shell operation in the boundary state in the axial direction, so the calculated normal cross section values of tubular concrete elements do not allow to use materials completely. Thus, the stress values σ_{s1} in equation (1) can be much less than the stress limit value of the concrete-filled steel tubular element steel pipe f_y in its ultimate state. The calculations' consequence with such in-accuracy is insufficient use of the concrete-filled element normal cross section strength calculated value. Therefore, the purpose of this article is to eliminate calculations shortcoming, namely to obtain the reinforced concrete steel tubular element strength equation, which would allow the materials complete strength.

To obtain the tubular concrete element strength equation, which would allow to obtain the materials strength value, which is used in full, consider the tubular concrete element part with thickness dz = 1 (Fig. 1). It should be borne in mind that the circular force inside the layer of the shell with a thickness dz = 1 (Fig. 1) from the meridional pressure p of concrete can be found from the equation $\sigma_{s2}{\cdot}t{\cdot}dz = p{\cdot}R{\cdot}dz$, whence

$$p = \frac{\sigma_{s2} \cdot t}{R} . \tag{2}$$

If expression (2) is substituted in the right equation part (1), then the result of this action will be such a dependence as:

$$N_{Rd} = A_c \left(f_c + 4\frac{\sigma_{s2} \cdot t}{R} \right) + A_s \cdot \sigma_{s1} . \tag{3}$$

Given that for equation (3) the values of $A_s = 2\pi\ R\ t$ and $A_c = \pi\ R^2$, their ratio was obtained as:

$$\frac{t}{R} = \frac{A_s}{2A_c} . \tag{4}$$

After obtaining equation (4), the next step in mathematical transformations will show us expression (3) in the following form:

$$N_{Rd} = A_c f_c + 2\sigma_{s2} \cdot A_s + A_s \cdot \sigma_{s1} . \tag{5}$$

Now, to represent expression (5) in a more perfect view, it was introduced the notation:

$$\frac{\sigma_{s2}}{\sigma_{s1}} = k \ \Rightarrow \sigma_{s1} = \frac{\sigma_{s2}}{k} , \ \sigma_{s2} = k\sigma_{s1} . \tag{6}$$

from (1) it becomes clear that

$$N_{Rd} = A_c f_c + (2k+1) \cdot A_s \cdot \sigma_{s1} . \tag{7}$$

The above expression (7) has two unknown quantities σ_{s1} (or axial stresses σ_{s1} and meridional stresses σ_{s2}) and k. The first of them, for example σ_{s2}, can be easily determined from the condition of compatibility and shell deformation uniformity and concrete in the radial direction. Given this deformation feature of the tubular concrete element layer dz, it can be determined that at point A the displacement of concrete u_c and the steel shell u_s displacement are equal, i.e.

$$u_s = u_c . \tag{8}$$

Each of the displacements given in equation (8) can be calculated from the expressions obtained during the study. In particular

$$u_c = \varepsilon_c \cdot R = \left(\frac{p}{E_c} \right) R. \tag{9}$$

On the other hand, the displacement u_c in equation (9) can be expressed in terms of the circular stresses σ_{s2} unknown value with the using dependence (2). In this context:

$$u_c = \left(\frac{\sigma_{s2}}{E_c} \frac{t}{R} \right) R, \tag{10}$$

where E_c is the concrete deformation modulus;

t is the thickness of the shell; R is the shell inner radius.

The next step is to express the displacement u_s from equation (8) due to the unknown value of the circular stress σ_{s2}. For this action, it is assumed that all points of the inner ring of the shell (including points A and C) due to the pressure p have a radial displacement. As a consequence of this process, the inner ring expanded by $2\pi\ (R + u_s) - 2\pi\ R = 2\pi \times u_s$, and the relative value of this elongation in the circular direction, in its turn, was $\varepsilon_s = 2\pi \times u_s\ /\ 2\pi\ R = u_s\ /\ R$. However, considering that the relative radial displacement value u_s is also the value of $u_s\ /\ R$, it must be noted that the relative shell displacements in the radial and circular directions are the same. If this proof of the shell relative displacements (deformations) equality in the radial and circular directions was chosen as a basis, that

$$u_s = \varepsilon_s R . \tag{11}$$

The relative shell deformation in the radial direction can be found from the dependences based on Hooke's Law [15 – 17], if the calculation basis is the shell relative displacements equality in the radial and circular directions in case of concrete-filled steel tubular element deformation. Taking into account these dependencies, it becomes clear that

$$\varepsilon_s = \frac{\sigma_{s2} - \nu\sigma_{s1}}{E_s}, \tag{12}$$

here ν is the Poisson's ratio.

From expression (11) the tube shell radial displacement, taking into account equation (12) will take the following form

$$u_s = \left(\frac{\sigma_{s2} - \nu\sigma_{s1}}{E_s} \right) R. \tag{13}$$

In turn, the unknown value of σ_{s2} from equation (7) can be determined from the dependence, which will be obtained by replacing in equation (8) each of the quantities by its corresponding expressions (10), (13) and after performing certain mathematical transformations. This dependence is given below as expression (14).

$$\sigma_{s2} = \frac{\nu}{1 - \frac{E_s}{E_c}\frac{t}{R}} \sigma_{s1} = \frac{\nu}{1 - 2\alpha_s \frac{t}{D}} \sigma_{s1}, \tag{14}$$

here $D = 2R$ is the inner diameter of the shell.

Applying relation (6) and equation (14) at the same time, it can be reasonably noted like a

$$k = \frac{\nu}{1 - \frac{2E_s}{E_c}\frac{t}{D}} = \frac{\nu}{1 - 2\alpha_s \frac{t}{D}}. \tag{15}$$

If expression (15) will chosen as a basis, then the dependence (7), which makes it possible to determine the strength of the concrete-filled steel tubular element normal cross section, will be appropriately presented in the following form:

$$N_{Rd} = A_c f_c + \left(\frac{2\nu}{1 - 2\alpha_s \dfrac{t}{D}} + 1 \right) A_s \sigma_{s1}. \tag{16}$$

Using equation (7) or (16) with the expression of displacements through stress in order to check the strength of the cylindrical tubular concrete element under central compression normal cross section, it is necessary to make an additional equation to calculate the meridional stresses σ_{s1}. Such an equation can be reduced to a form in which it would become the Saint-Venant (17) or Huber-Mises-Genk (18) plasticity theory condition, and would occur as follows:

$$\sigma_{s1} + \sigma_{s2} = f_y, \tag{17}$$

$$\sigma_{s1}^2 - \sigma_{s1}\sigma_{s2} + \sigma_{s2}^2 = f_y^2. \tag{18}$$

When (17) and (18) are used together with (6) and (7), it is possible to obtain the final expression form to determine the tubular concrete element under central compression bearing capacity:

$$N_{Rd} = k_{cs} A_c f_c + A_s f_y. \tag{19}$$

According to the Saint-Venant plasticity theory, in expression (19) the concrete strengthening coefficient will be determined from equation (20)

$$k_{cs} = 1 + \frac{4k}{k+1} \frac{f_y}{f_c} \frac{t}{D}, \tag{20}$$

where

$$k = \frac{\nu}{1 - \dfrac{2E_s}{E_c} \dfrac{t}{D}} = \frac{\nu}{1 - 2\alpha_s \dfrac{t}{D}}, \tag{21}$$

based on relation (6) and knowing that

$$\sigma_{s2} = \frac{\nu}{1 - \dfrac{E_s}{E_c} \dfrac{t}{R}} \sigma_{s1} = \frac{\nu}{1 - 2\alpha_s \dfrac{t}{D}} \sigma_{s1}.$$

If the theory of Huber-Mises-Genka plasticity is applied to (19), then to determine the concrete strengthening coefficient the following dependence become obtained:

$$k_{cs} = 1 + 4\left(\frac{2k+1}{\sqrt{k^2+k+1}} - 1 \right) \frac{f_y}{f_c} \frac{t}{D}. \tag{22}$$

As for the equations for determining the tubular concrete element bearing capacity, after substituting in expression (19) the equations for calculating the value of the concrete strengthening coefficient according to both the above plasticity theories (20, 22) dependencies become obtained by which it becomes possible to calculate the tubular concrete element bearing capacity values by applying the Saint-Venant (23) and Huber-Mises-Genk plasticity theories(24).

$$N_{Rd} = \left(1 + \frac{4k}{k+1} \frac{f_y}{f_c} \frac{t}{D} \right) A_c f_c + A_s f_y, \tag{23}$$

$$N_{Rd} = \left(1 + 4\left(\frac{2k+1}{\sqrt{k^2+k+1}} - 1 \right) \frac{f_y}{f_c} \frac{t}{D} \right) A_c f_c + A_s f_y. \tag{24}$$

***Example of calculation*.** Determine the steel tube concrete-filled steel column bearing capacity with a diameter D = 102 mm and a wall thickness t = 3 mm. The steel yield strength stresses value f_y = 287 MPa (E_s = 210000 MPa, ν = 0,3). The tube is filled with concrete with the characteristics: f_c = 13,5 MPa, E_c = 25600 MPa.

The element bearing capacity is determined based on Saint-Venant's theory by expression (23) and based on the Huber-Mises-Genk theory by expression (24) by the following method.

1. Find the parameter k value the using expression (15):

$$k = \frac{\nu_s}{1 - \dfrac{2E_s}{E_c} \dfrac{t}{D}} = \frac{\nu_s}{1 - 2\alpha_s \dfrac{t}{D}} = \frac{0,3}{1 - 2 \times 8,203 \times \dfrac{3,0}{102}} = 0,580,$$

given that in this equation

$$\frac{E_s}{E_c} = \frac{210000}{25600} = 8,203.$$

2. The concrete k_{cs} strengthening coefficient when applying the Saint-Venant plasticity theory by expression (20) is established:

$$k_{cs} = 1 + \frac{4k}{k+1} \frac{f_y}{f_c} \frac{t}{D} = 1 + \frac{4 \times 0,580}{0,580 + 1} \times \frac{287}{13,5} \times \frac{3}{102} = 1,918.$$

The value of the strengthening k_{cs} coefficient by applying the of Huber-Mises-Genk plasticity theory by expression (22):

$$k_{cs} = 1 + 4\left(\frac{2k+1}{\sqrt{k^2+k+1}} - 1 \right) \frac{f_y}{f_c} \frac{t}{D} =$$

$$= 1 + 4\left(\frac{2 \times 0,580 + 1}{\sqrt{0,580^2 + 0,580 + 1}} - 1 \right) \frac{287}{13,5} \times \frac{3}{102} = 2,231.$$

3. The tubular concrete element bearing capacity the is calculated taking into account theSaint-Venant plasticity theory by expression (23) as:

$$N_{Rd} = k_{cs} A_c f_c + A_s f_y = (1,918 \times 7238,2 \times 13,5 +$$

$$+933,1 \times 287) \times 10^{-3} = 455,2\ kN,$$

given that in this equation A_c = 7238.2 mm^2, A_s = 933.1 mm^2.

The tubular concrete element bearing capacity according to expression (24) when applying the Huber-Mises-Genk plasticity theory is calculated as follows:

$$N_{Rd} = k_{cs} A_c f_c + A_s f_y = (2,231 \times 7238,2 \times 13,5 +$$

$$+933,1 \times 287) \times 10^{-3} = 485,8\ kN.$$

To identify the nature of the change in the concrete strengthening coefficient value depending on various parameters, it was calculated in the range of all concrete classes [19]. For comparison, 2 sections of pipes with a diameter of 102 mm with a wall thickness of 3 mm and 1 mm, respectively, were used. The steel characteristics were taken from the calculation example. The calculations results are shown in Table 1.

Table 1 − The concrete strengthening coefficient values for tubular concrete elements with different pipe wall thickness of the pipe

No.	Concrete class	A wall thickness t = 3 mm			A wall thickness t = 1 mm		
		k_{cs} by (20)	k_{cs} by (22)	$\frac{k_{cs}^{(22)} - k_{cs}^{(20)}}{k_{cs}^{(22)}}$, %	k_{cs} by (20)	k_{cs} by (22)	$\frac{k_{cs}^{(22)} - k_{cs}^{(20)}}{k_{cs}^{(22)}}$, %
1	C12/15	3,198	4,139	22,73	1,379	1,586	13,05
2	C16/20	2,291	2,933	21,89	1,268	1,415	10,39
3	C20/25	1,916	2,389	19,80	1,208	1,321	8,55
4	C25/30	1,739	2,127	18,24	1,175	1,27	7,48
5	C30/35	1,617	1,943	16,78	1,151	1,233	6,65
6	C32/40	1,531	1,814	15,60	1,133	1,205	5,98
7	C35/45	1,453	1,695	14,28	1,116	1,179	5,34
8	C40/50	1,403	1,619	13,34	1,105	1,162	4,91
9	C45/55	1,365	1,561	12,56	1,096	1,148	4,53
10	C50/60	1,328	1,504	11,70	1,087	1,134	4,14

As can be seen from table 1, if the concrete strengthening coefficient values deviation according to two different plasticity theories is analyzed, given as a percentage for each case of different pipe wall thickness, it can be concluded that for samples with a wall thickness of 1 mm in most cases it will be smaller than with a wall thickness of 3 mm.

The calculated concrete strengthening coefficient values, which were obtained on the basis of Saint-Venant and Huber-Mises-Genk plasticity theories, were compared by plotting the concrete values strengthening coefficient dependence on the concrete class (Fig. 2).

According to the same initial data for the same cases of tubular concrete elements pipe wall thicknesses taking into account the obtained concrete strengthening coefficient values, that were given in Table 1, then calculate the bearing capacity value and enter the calculations results in Table 2. Also in this table the error value of the concrete-filled steel tubular elements bearing capacity values, which were calculated on the basis of Saint-Venant plasticity theories by expression (23) and Huber-Mises-Genk expression (24), in percent to facilitate the results and clarity analysis.

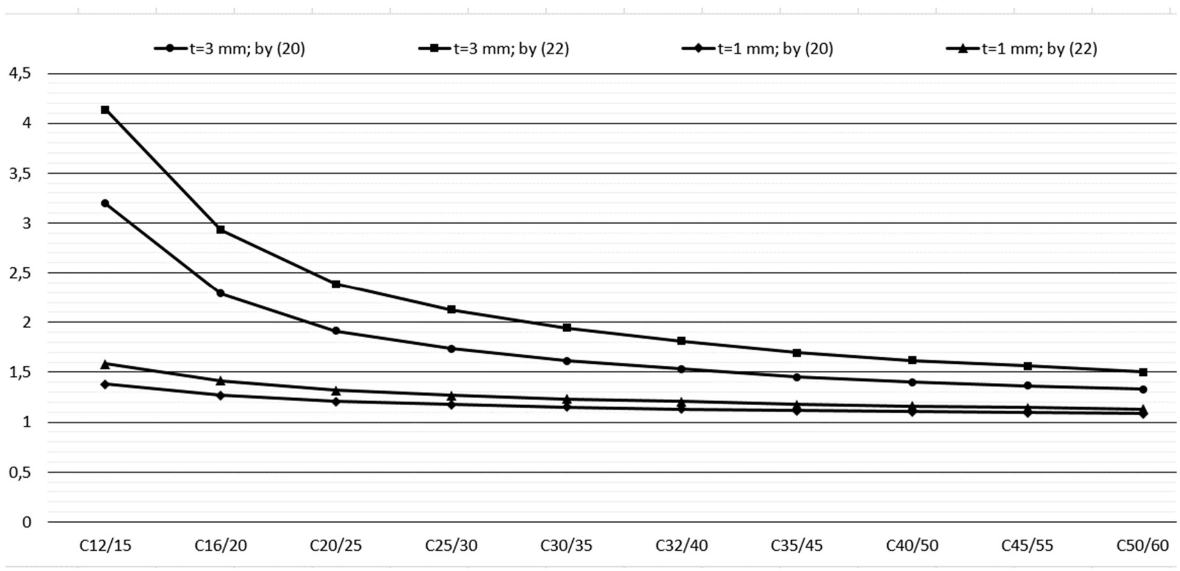

Figure 2 – Graphs of concrete strengthening coefficient values dependence on a concrete class for pipes with the set parameters

Table 2 − The bearing capacity values of the concrete-filled steel tubular elements with different pipe wall thickness of the pipe

No.	Concrete class	A wall thickness t = 3 mm			A wall thickness t = 1 mm		
		N_{Rd} by (23), kN	N_{Rd} by (24), kN	$\frac{N_{Rd}^{(24)} - N_{Rd}^{(23)}}{N_{Rd}^{(24)}}$, %	N_{Rd} by (23), kN	N_{Rd} by (24), kN	$\frac{N_{Rd}^{(24)} - N_{Rd}^{(23)}}{N_{Rd}^{(24)}}$, %
1	C12/15	464,6	522,5	11,08	352,6	365,4	3,49
2	C16/20	458,5	511,9	10,44	373,3	385,6	3,17
3	C20/25	468,9	518,5	9,57	394,6	406,4	2,92
4	C25/30	481,8	529,5	9,02	412,4	424,1	2,76
5	C30/35	496,0	542,0	8,49	430,3	441,8	2,62
6	C32/40	511,6	556,7	8,10	448,2	459,7	2,49
7	C35/45	530,7	574,5	7,62	469,7	481,1	2,37
8	C40/50	547,1	590,1	7,29	487,8	499,1	2,27
9	C45/55	564,2	606,8	7,01	505,8	517,1	2,18
10	C50/60	585,0	627,0	6,70	527,4	538,7	2,08

In general, the data in Table 2, firstly, is a clear proof of the obtaining the elements bearing capacity value possibility according to the above calculation method and, secondly, demonstrate that the bearing capacity values obtained from the same source data, i.e. calculated for one case, but given the different plasticity theories application, do not differ significantly. Due to the direct influence of the concrete strengthening coefficient value on the elements bearing capacity value, based on this table, it is concluded that the obtained values discrepancy according to different plasticity theories decreases with decreasing wall thickness.

Graphs of the bearing capacity value dependence on the prototype concrete class were compiled to better understand the calculations results. These graphs are shown in Figure 3.

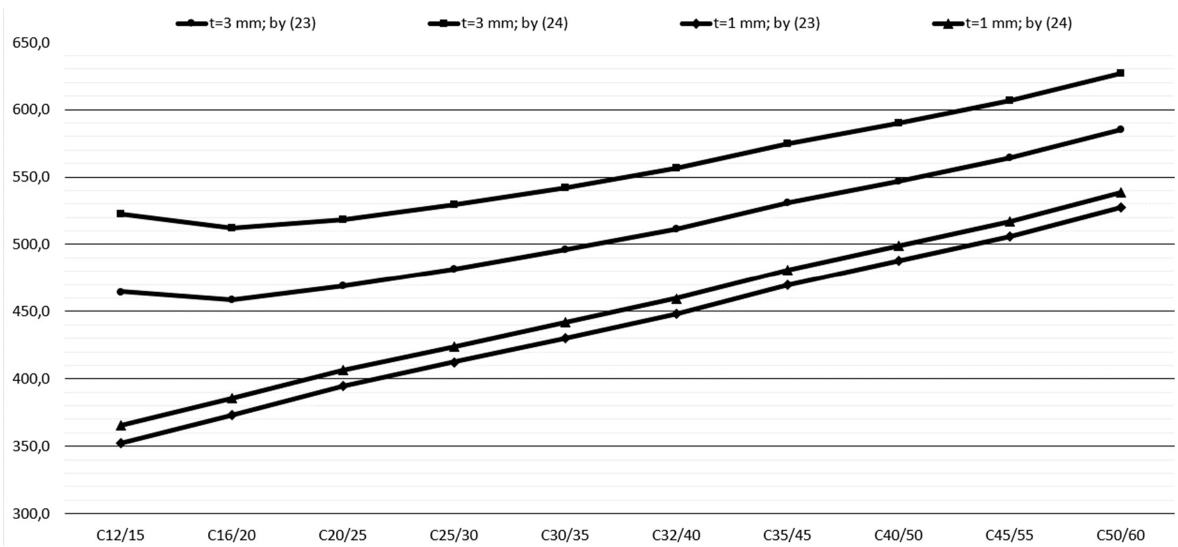

Figure 3 – Graphs of dependence of values of bearing capacity on a class of concrete

Conclusions

Based on the Saint-Venant and Huber-Mises-Genk plasticity theories application, analytical expressions are obtained to calculate the concrete reinforcement coefficient of a tubular concrete element core in the limit state and the strength equation of a tubular concrete element, which allows to use full strength. Calculations were made according to the given expressions and comparison of the coefficient strengthening values, obtained on the basis of the both presented plasticity theories application. To determine the proposed method accuracy, it is planned to compare the calculation results with experimental data.

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The durability of cryogenic structure materials

Medved Ivan[1], Tatarchenko Halyna[2*], Biloshytskyi Mykola[3], Poddubny Sergey[4]

[1] Volodymyr Dahl East Ukrainian National University https://orcid.org/0000-0001-9988-9900
[2] Volodymyr Dahl East Ukrainian National University https://orcid.org/0000-0003-4685-0337
[3] Volodymyr Dahl East Ukrainian National University https://orcid.org/0000-0002-9935-4161
[4] Volodymyr Dahl East Ukrainian National University https://orcid.org/0000-0003-1458-8784
*Corresponding author E-mail: tatarchenkogalina@gmail.com

According to world statistics, the main share of destruction in engineering practice occurs precisely because of fatigue, therefore, the fatigue problem is one of the most pressing scientific and technical problems of our time, which solution requires additional complex experimental and theoretical studies. The paper presents the experimental studies results of the effect of deep cooling on low-cycle fatigue and cyclic creep of stainless structural steel 03X20N16AG6 under conditions of a pulsating cycle of an external load change with a frequency of 0.033 s^{-1} (2 cycles/min) in air and environments of liquid refrigerants (nitrogen and helium) at temperatures of 293, 77 and 4.2 K, respectively

Keywords: structural alloys, deep cooling, low-cycle fatigue, cyclic creep

Довговічність матеріалів кріогенних конструкцій

Медвідь І.І.[1], Татарченко Г.О.[2*], Білошицький М.В.[3], Піддубний С.В.[4]

[1] Східноукраїнський національний університет імені Володимира Даля
[2] Східноукраїнський національний університет імені Володимира Даля
[3] Східноукраїнський національний університет імені Володимира Даля
[4] Східноукраїнський національний університет імені Володимира Даля
*Адреса для листування E-mail: tatarchenkogalina@gmail.com

Світова статистика свідчить, що основна частка руйнувань в інженерній практиці відбувається через утому конструкцій і матеріалів. Тому проблема втоми є однією з найбільш актуальних науково-технічних завдань сучасності, вирішення якої потребує додаткових комплексних експериментальних і теоретичних досліджень. Серед відповідальних конструкцій і об'єктів низькотемпературного призначення широкого поширення набули зварні великогабаритні ємності і резервуари, що перебувають під тиском при температурах від 293 до 4,2 К. Такі будівельні конструкції, працюють в умовах циклічного від нульового розтягування, що повторюється з низькою частотою. Такі умови навантаження виникають при експлуатації посудин для транспортування і зберігання зріджених газів (кисню, азоту, водню, гелію), кріогенних трубопроводів, посудин високого тиску, кріогенераторов і т.п. Тому завдання оцінки несучої здатності і довговічності в умовах впливу циклічних навантажень в широкому діапазоні температур має надзвичайно важливе значення. У роботі наводяться результати експериментальних досліджень впливу глибокого охолодження на малоциклічна втому і циклічну повзучість нержавіючої конструкційної сталі 03Х20Н16АГ6 в умовах пульсуючого циклу зміни зовнішнього навантаження з частотою 0,033 $с^{-1}$ (2 цикл/хв) на повітрі і в середовищах рідких холодоагентів (азоту і гелію) при температурах 293, 77 і 4,2 К відповідно. Результати експериментальних досліджень підтвердили той факт, що в інтервалі температур 293 – 77 К на кривих циклічної повзучості стадія прискореної повзучості вельми обмежена по довговічності, або взагалі відсутня, тому можна з упевненістю сказати, що число циклів до руйнування стали 03Х20Н16АГ6 в малоциклової області буде визначатися її здатністю чинити опір деформації на сталій стадії

Ключові слова: конструкційні сплави, глибоке охолодження, малоциклічна втома, циклічна повзучість

Introduction

Metallurgy, mechanical engineering, energy, agriculture, food industry, energy, electronics, rocket and space technology - this is far from the complete list of national economy areas in which liquid cryogenic products (cryoproducts). The production volumes of such products and their use scale are constantly increasing. This is due to the fact that cryogenic temperatures (below 120 K) provide unique opportunities for the implementation of such physical phenomena and processes that do not manifest themselves under normal conditions, but are used very effectively in science and technology.

The solution of fundamental scientific problems and applied problems of both promising and current importance is determined by the level of cryogenic technology development and its practical application degree.

The continuous expansion of the liquid cryoproducts production scale has led in recent years to a significant increase in the volume of systems production for their storage and transportation. These systems, as a rule, are welded shell structures in execution, they are operated in difficult conditions of temperature and force effects. The specific weight of their products in the total output of cryogenic engineering products is very significant, and the operating conditions in comparison with other types of cryogenic structures are the most stressful.

For the manufacture of cryogenic shell structures, expensive non-ferrous alloys and special steels are used, the consumption degree that, taking into account the sufficient struction material consumption and the expanding production scale, is constantly increasing. Therefore, one of the most urgent for cryogenic engineering at present is the problem of reducing the material consumption of shell structures and increasing their reliability and durability. It is obvious that a solution to this problem for cryogenic engineering products can be achieved by improving the methods of their strength calculations based on taking into account the specific hardening effect of low temperature on structural alloys. Low-cycle fatigue is one of the main factors determining the durability (operating life) of the main structural parts. According to world statistics, a large part of failures in engineering occurs precisely because of fatigue. Therefore, the fatigue problem is currently one of the most relevant scientific and technical problems. Its solution requires additional complex experimental and theoretical studies [1].

Assessing the bearing capacity and durability under cyclic loading conditions is extremely important.

Welded large tanks and containers being under pressure at a temperature from 293 to 4.2 K are widespread among critical structures and units for low-temperature purposes. Such building structures operate under conditions of zero-to-tension stress cycle repeated at a low frequency. Such loading conditions take place during the use of containers for the transportation and storage of liquefied gases (oxygen, nitrogen, hydrogen, helium), cryogenic pipelines, high-pressure vessels, cryogenerators, etc. As a result of cyclic changes in loading (especially in concentrators zones), significant stresses can arise in the material of these structures. Therefore, stresses can reach and exceed creep strengths, which leads to their failure after a small number of loading cycles N_p [2].

Review of research sources and publications

In the total rolled production volume, structural steel makes the largest amount.

During their service, various facilities and structures made of these materials bear complex external loading (tensile, compressive, bending, shock, alternating signs or their combinations), experiencing fluctuations in ambient temperature in the summer and winter months. They are also exposed to the atmosphere and corrosive environment (sea and river water, aqueous solutions of salts, alkalis, acids, etc.).

Sharp drops in temperature under conditions of structurally constrained deformation lead to great residual stresses, which, combining with the stresses from external forces, complicate the operating conditions of the material and can lead to accidents if its quality is unsatisfactory [3, 4].

There are high demands on steel as a structural material. This is explained by difficult operating conditions of mechanisms and structures, especially in the northern regions, reduced design sections when creating modern structures, machine units, and mechanisms aimed to reduce their mass and metal consumption and the need to ensure reliability, durability, and safety of their operation. Depending on the conditions of use and operation, the requirements for structural steel may somewhat change, but the most important of them can be generally distinguished.

The structural steel of building structures must have a combination of high strength and plastic properties. Creep strength is the main structural characteristic among the strength properties. The choice of this characteristic as the basis for strength calculations is explained by the fact that at higher stresses, irreversible linear changes take place in the structure, which can lead to its failure. Increased creep strength allows reducing the design sections, and, consequently, the steel structure mass, or bearing higher operating stresses at the same mass.

As the structure operation experience shows, metal should have the property of local plastic deformation for relaxation of stress peaks in the zone of various concentrators (holes, grooves, undercuts, dents, lack of penetration, welding cracks, etc.) leading to the three-dimensional stress state. The better this property is, the more resistant the metal is to the occurrence and propagation of cracks at local overstresses, i.e., ultimately, the metal reliability in structures increases.

Along with the strength and plasticity properties, a very important role in ensuring reliability and structure performance is given to indicators that determine metal entering the brittle state. Firstly, this is the operating temperature of the constructed structure, the presence of a notch (concentrator), the load application rate, and the three-dimensional stress state degree.

Currently, the problem of increasing the metal resistance to brittle failure is becoming one of the most important. This is explained by the need to ensure the reliability of structures and machines under harsh climatic conditions. Besides, an increase in the scale of engineering structures, the use of large welded assemblies and structures with greater rigidity and lower flexibility than riveted structures, as well as the material behavior under conditions of a combination of high stresses and corrosive environments create conditions leading to brittle failure [5, 6].

To assess the tendency of steel to brittle failure, there is a widely used method of impact testing of standard samples with the determination of impact strength and temperature of entering the brittle state. The prevalence of this type of testing is explained not only by the simplicity of making samples and a simple method of the series test but also by the fact that statistically reliable relations are sometimes observed between the impact strength properties and the steel behavior during operation.

However, in most cases, impact strength testing of standard samples does not give a complete picture of the material behavior in a structure.

Therefore, they are trying to find better methods for determining the tendency of steel to enter the brittle state, which would more fully correspond to the real conditions of the metal behavior in structures.

While manufacturing metal structures and specific types of rolled products (for example, railroad rails), which bear alternating loads during operation, an important role is given to increasing the endurance limit as one of the factors determining their service life. The endurance limit increases with increasing strength, metal purity as for non-metallic inclusions, improving the quality of its surface. It is especially important to increase the endurance limit in the presence of stress concentrators.

A prerequisite for durability and reliability of structures and facilities is sufficiently high corrosion resistance. It is especially important to increase the corrosion resistance for high-strength steel due to decreased design sections of structural elements when using this steel. At smaller structural sections, corrosion damage is more dangerous than in thicker sections made of steel with reduced strength.

To prevent corrosion, steel is subjected to special alloying (chromium, nickel, copper, phosphorus), careful and timely painting, galvanizing, and phosphating. Recently, it has been proposed to apply a vinyl chloride covering on the metal surface.

Finally, structural steel must have satisfactory processing properties. First of all, it must meet the requirements of weld ability ensuring the same strength of the basic metal and the welded joint and has a minimal tendency to deformation aging. It must be processed without any special difficulties in the hot and cold states (rolling, forging, bending, processing on metal cutting machines), and be relatively inexpensive to manufacture.

Definition of unsolved aspects of the problem

It has been experimentally established that under low-cycle loading, directional plastic deformation of structural alloys takes place, which is most clearly manifested in the asymmetric loading cycle. The process of one-sided accumulation of plastic deformation ε_p occurring as a result of variable loads is called cyclic creep [1].

In some national works, as well as in the works of foreign authors, experimental data are given indicating that at high-stress levels and low loading frequencies, cyclic creep is a factor determining the material durability not only at the high-temperature but also for the temperature range of 293 and 77 K [1, 5].

Problem statement

Experimental study of the operating temperature effect on low-cycle fatigue, cyclic creep, and durability of materials of cryogenic structures based on the example of 03H20N16AG6 structural steel.

Basic material and results

In this work, experimental studies have been conducted to identify the effect of deep freezing on low-cycle fatigue and cyclic creep based on the example of 03H20N16AG6 stainless steel. Loading has been carried out in a pulsating cycle with a frequency of 0.033 s^{-1} (2 cycles/min) in air and liquid refrigerants (nitrogen and helium) at temperatures of 293, 77 and 4.2 K, respectively.

Analysis of the obtained experimental data has shown that directional plastic deformation takes place at the test temperature of 293 K in the range of fatigue life of $0.5\text{-}10^4$ cycles (Fig. 1). For comparison, the curves of cyclic creep for steel 03Kh13AG19 and PT3V titanium alloy are shown (Fig. 2, Fig. 3).

In 03H20N16AG6 steel, cyclic creep takes place most intensively in the zone of stresses corresponding to quasi-static failure (Fig. 1). The curves characterizing the kinetics of changes in plastic deformation depending on the number of loading cycles in this stress zone have three characteristic sections: unsteady decaying, steady, and unsteady accelerated creep. At the same time, plastic deformation mainly takes place during the last two stages.

A decrease in the test temperature to 77 K does not qualitatively change the nature of deformation and failure of the studied materials. However, there is a sharp deceleration of directional plastic deformation characterized by a change in the inclination angle of the steady creep sections on curves built for the same values of reduced stresses at test temperatures of 293 and 77 K respectively.

Thus, taking into account that in the temperature range of 293 and 77 K the stage of accelerated creep is very limited in terms of durability, or is absent at all on the cyclic creep curves, it can be said with confidence that the number of cycles before failure of these materials in the low-cycle zone will be determined by their ability to resist deformation at the steady-state.

At the same time, the kinetics of directional plastic deformation of 03H20N16AG6 steel at temperatures of 293 and 77 K can be described from the standpoint of the theory of hardening with a sufficient degree of accuracy. Significant changes in the behavior of structural materials occur when they are tested under deep freezing conditions (T = 4.2 K). The deformation mechanism changes, plasticity decreases sharply [7]. Deformation accumulated before failure takes place in the first loading half-cycle as a result of intermittent creep acts, the number of which is uniquely determined by the level of maximum cycle stresses [8 – 11].

With further cyclic loading, there is no plastic deformation of the material. This indicates that directional plastic deformation at T = 4.2 K is completely suppressed, and failure of the samples takes place as a result of the formation and development of a fatigue crack to the critical value.

At the same time, it should be noted that intermittent creep has been experimentally recorded for some structural materials at the initial stage of cyclic loading.

Consequently, the absence of cyclic creep in structural steel and alloys at T = 4.2 K cannot be considered as an established fact. Additional experimental studies are required for a deeper research of this phenomenon.

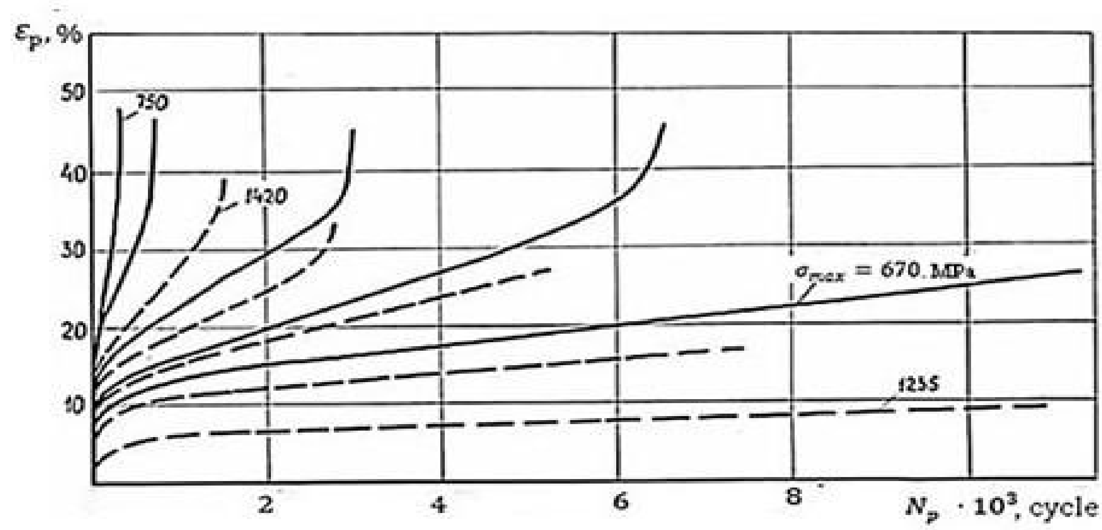

Figure 1 – Cyclic creep curves of 03H20N16AG6 steel
— - 293 К; ---- - 77 К

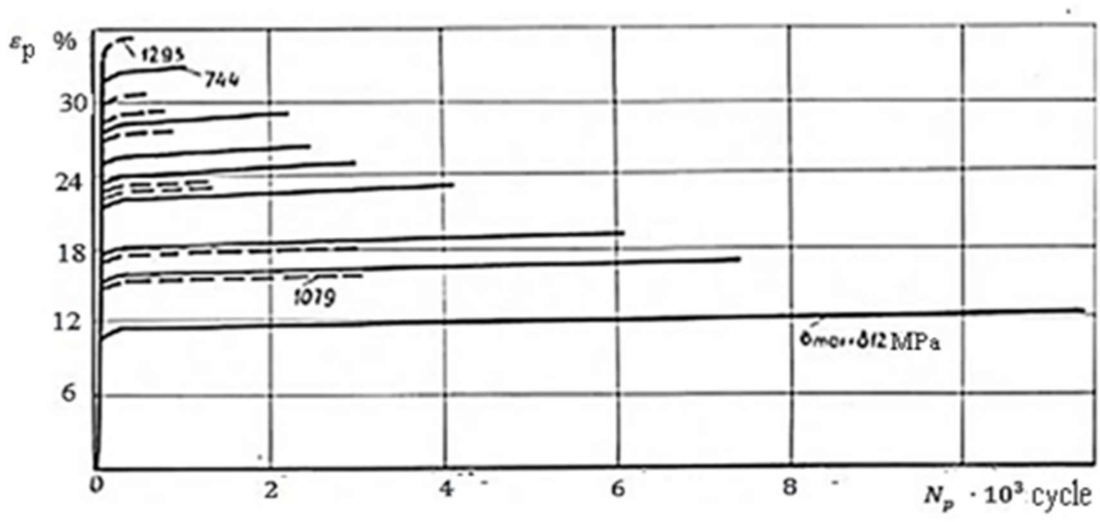

Figure 2 – Cyclic creep curves of 03Kh13AG19 steel
— - 293 K; ---- - 77 K

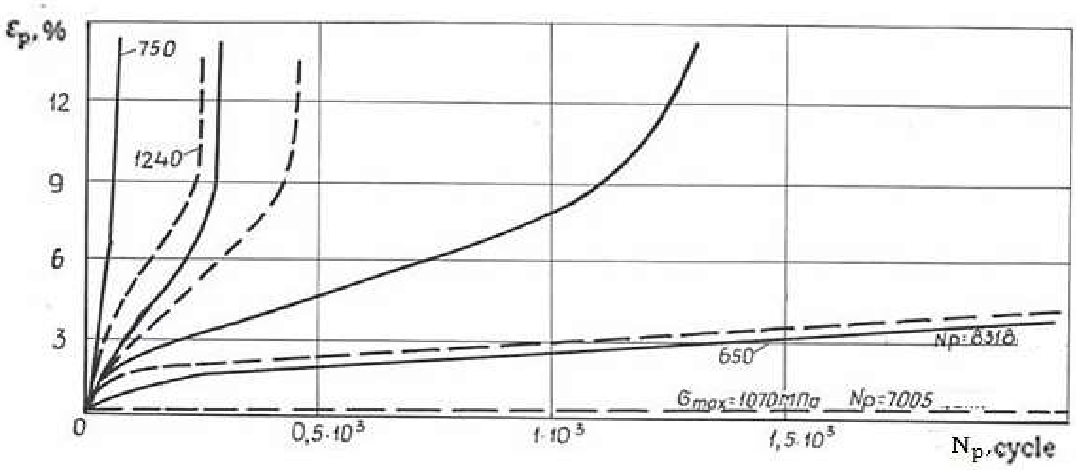

Figure 3 – Cyclic creep curves of PT3V titanium alloy
— - 293 K; ---- - 77 K

Conclusions

1. At the test temperature of 293 K in the range of durability of 0.5-10^4 cycles, 03H20N16AG6 steel has directional plastic deformation, which is a factor determining the durability of cryogenic structures.

2. For samples of 03H20N16AG6 steel, cyclic creep takes place most intensively in the stress range corresponding to quasi-static failure. They determine the service life of metal structures and specific types of rolled products.

3. Lowering the test temperature to 77 K does not make qualitative changes in the nature of the material deformation and failure. Therefore, the durability of cryogenic structures in this temperature range is completely controlled by the cyclic creep intensity.

4. At T = 4.2 K, directional plastic deformation of 03H20N16AG6 structural steel is completely suppressed, and failure takes place as a result of the formation and development of a fatigue crack to the critical value.

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Method and example of calculation of combined reinforced bending elements

Zhuravskyi Oleksandr[1*]

[1] Kyiv National University of Construction and Architecture https://orcid.org/0000-0001-7065-3312
*Corresponding author E-mail: zhuravskyi.od@knuba.edu.ua

A method for calculating rectangular cross-section bending elements, reinforced with ordinary and prestressed reinforcement, as well as steel fiber, based on the deformation method. This takes into account stress losses in the reinforcement from creep deformations and shrinkage of steel-fiber-concrete. The increase in compressive strength of reinforced concrete under biaxial compression conditions is also taken into account. The bearing capacity calculation results of a standard road plate P60.38 and a similar plate with metal fiber are given.

Keywords: bearing capacity, steel-fiber-concrete, bending moment, curvature, prestressed reinforcement, relative deformations, stresses in reinforcement, stresses in steel-fiber-concrete

Методика та приклад розрахунку комбіновано армованих згинальних елементів

Журавський О.Д.[1*]

[1] Київський національний університет будівництва і архітектури
*Адреса для листування E-mail: zhuravskyi.od@knuba.edu.ua

Сучасне будівництво неможливе без використання залізобетонних конструкцій з попередньо-напруженою арматурою. Для покращення міцністних та деформативних характеристик бетону використовують фіброве армування. Серед таких фібр найбільш широко використовується стальна фібра. Вона значно покращує міцність сталефібробетону на розтяг. Це дає можливість враховувати роботу сталефібробетону в розтягнутій зоні перерізу згинальних елементів. Діючі нормативні документи не дають не дають рекомендацій щодо розрахунку сталефібробетонних плоских елементів, які працюють у двох напрямках. Відсутні рекомендації щодо розрахунку сталефібробетонних елементів з попередньо-напруженою арматурою. Дослідження несучої здатності, тріщиностійкості та деформацій двохосно попередньо-напружених сталефібробетонних плит практично відсутній. Запропонована методика розрахунку згинальних елементів прямокутного перерізу, армованих звичайною та попередньо-напруженою арматурою, а також стальною фіброю, на основі деформаційного методу. При цьому враховуються втрати напружень в арматурі від деформацій повзучості та усадки сталефібробетону. Також враховується зростання міцності сталефібробетону на стиск в умовах двохосного обтиску. У результаті порівняльного розрахунку несучої здатності стандартної дорожньої плити П60.38 та аналогічної плити, у якій арматурні сітки були замінені стальною фіброю, встановлено, що несуча здатність плити зі стальною фіброю вища від стандартної на 24,4%. Ефективність плит зі стальною фіброю полягає в тому, що використанні стальної фібри дає можливість зменшення кількості високоміцної попередньо-напруженої арматури до 10…15%. Завдяки хорошим властивостям сталефібробетону протидії стиранню тривалість експлуатації плит аеродромних та дорожніх покриттів набагато більша від залізобетонних.

Ключові слова: плити для покриття доріг, несуча здатність, сталефібробетон, згинальний момент, кривизна, попередньо-напружена арматура, відносні деформації, напруження в арматурі, напруження в сталефібробетоні

Introduction

Modern construction is impossible without using reinforced concrete structures with prestressed reinforcement. Fiber reinforcement is used to improve the concrete strength and deformation characteristics. Among such fibers, steel fiber is the most widely used. It significantly improves the tensile strength of steel-fiber-concrete (SFC). This makes it possible to take into account the SFC operation in the bending elements stretched cross-sectional area. Current regulations do not provide recommendations for the SFC flat elements calculation that work in two directions. There are no recommendations for the SFC elements with prestressed reinforcement calculation. The bearing capacity study and crack biaxially prestressed SFC road slabs resistance is practically absent. The purpose of the work is a comparative calculation of standard road slabs and slabs with metal fiber.

Review of research sources and publications

In recent decades, extensive research on structures with inclusions of various fibers [10-13, 16, 17]. Fiber significantly improves the tensile strength of concrete. Steel fiber has proved itself very well. Recent studies of steel-fiber-concrete (SFC) have shown a positive effect on the work of bending elements. SFB is well resistant to abrasion. Therefore, it can be very effectively used in slabs of road and airfield surfaces.

Definition of unsolved aspects of the problem

The current regulations do not recommend the calculation of rectangular elements reinforced with ordinary and pre-stressed reinforcement, as well as steel fiber. There are also no recommendations for the calculation of the SFB of flat elements that work in two directions.

Problem statement

The purpose of this work is to develop a method for calculating the bending elements of rectangular cross section, reinforced with ordinary and pre-stressed reinforcement, as well as steel fiber, based on the deformation method. When calculating it is necessary to take into account stress losses in the reinforcement from creep deformations and shrinkage of steel-fiber-concrete. It is also necessary to take into account the increase in compressive strength of reinforced concrete under conditions of biaxial compression.

Basic material and results

Method of calculation of combined reinforced bending elements.

Consider a bending element of rectangular cross-section, reinforced with steel fiber and rod ordinary and prestressed reinforcement in compressed and stretched cross-sectional areas. The stress-strain state of a rectangular combined-reinforced section is shown in Fig. 1.

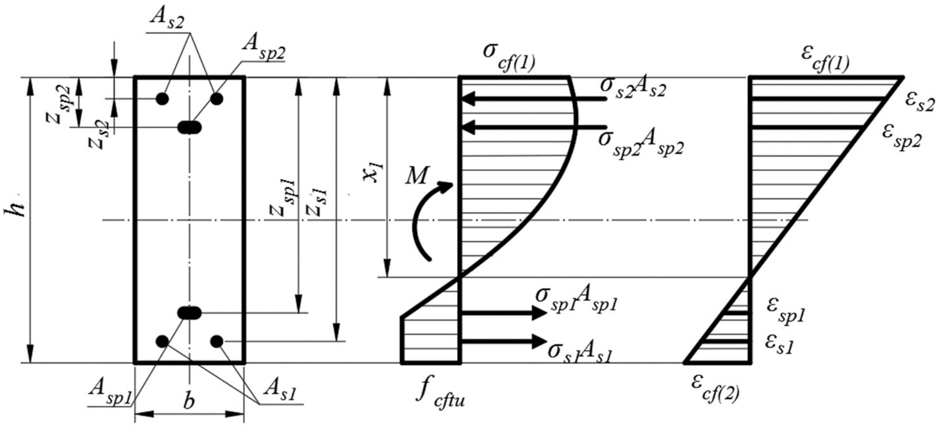

Figure 1 – Stress-strain state of a rectangular combined-reinforced section

Achieving fiber deformations of limit values is accepted as a criterion of bearing capacity exhaustion on a SFB normal section of an element $\varepsilon_{cftu} = -2\, f_{cftu} / E_{cf}$. The value of the ultimate bending moment for the SFB of bending elements of rectangular cross-section with pre-stressed reinforcement is recommended to be determined by the formulas (Fig. 1):

$$\frac{bf_{cf}k_c}{\aleph}\sum_{k=1}^{5}\frac{a_k}{k+1}\gamma^{k+1} - \frac{3}{4}bf_{cft}(h-x_1) + \sum_{i=1}^{n}\sigma_{si}A_{si} = 0; \tag{1}$$

$$\frac{bf_{cf}k_c}{\aleph^2}\sum_{k=1}^{5}\frac{a_k}{k+1}\gamma^{k+2} - \frac{11}{24}bf_{cft}(h-x_1)^2 + \sum_{i=1}^{n}\sigma_{si}A_{si}(x_1 - z_{si}) - M = 0. \tag{2}$$

In dependences (1), (2) according to [5] $\aleph$ – the curvature of the curved axis in cross-section (1/m):

$$\aleph = \left(\frac{1}{r}\right) = \frac{\varepsilon_{c(1)} - \varepsilon_{c(2)}}{h}; \tag{3}$$

$\varepsilon_{c(1)}$ – relative deformations of steel-fiber-concrete in the compressed cross-sectional area;

$\varepsilon_{c(2)}$ – relative deformations of steel-fiber-concrete in the stretched cross-sectional area;

γ – the ratio of relative compression strains $\varepsilon_{c(1)}$ to the limit ε_{cf1}:

$$\gamma = \frac{\varepsilon_{c(1)}}{\varepsilon_{cf1}}; \qquad (4)$$

x_1 – the height of the compressed zone (m):

$$x_1 = \frac{\varepsilon_{c(1)}}{\aleph}; \qquad (5)$$

$\overline{\aleph}$ – relative curvature:

$$\overline{\aleph} = \frac{\aleph}{\varepsilon_{cf1}}; \qquad (6)$$

σ_{si} – stresses in reinforceng rod;

z_{si} – distance from the reinforcement gravity center to the extreme verge compressed section;

a_k – the polynomial coefficients, which are determined depending on the value of the SFC compressive strength according to the method [8].

We present equations (1), (2) in the form

$$N_{cf} - N_{cft} + N_s = 0; \qquad (7)$$

$$M_{cf} + M_{cft} + M_s = M; \qquad (8)$$

where: N_{cf}, M_{cf} – efforts in the compressed zone of the SFC;

N_{cft}, M_{cft} – efforts in the stretched zone of the SFC;

N_s, M_s – total effort in reinforcing rods.

Let's describe the value of internal efforts

$$N_{cf} = \frac{bf_{cf}}{\overline{\aleph}} \sum_{k=1}^{5} \frac{a_k}{k+1} \gamma^{k+1}; \qquad (9)$$

$$N_{cft} = \frac{3}{4} bf_{cft}(h - x_1); \qquad (10)$$

$$N_s = \sigma_{s2}A_{s2} + \sigma_{sp2}A_{sp2} - \sigma_{s1}A_{s1} - \sigma_{sp1}A_{sp1} \qquad (11)$$

$$M_{cf} = \frac{bf_{cf}}{\overline{\aleph}^2} \sum_{k=1}^{5} \frac{a_k}{k+1} \gamma^{k+2}; \qquad (12)$$

$$M_{cft} = \frac{11}{24} bf_{cft}(h - x_1)^2; \qquad (13)$$

$$\begin{aligned} M_s &= A_{s1}E_{s1}\aleph(x_1 - z_{s1})^2 + \\ &+ A_{sp1}E_{sp1}(\aleph(x_1 - z_{sp1}) - \varepsilon_{p01})(x_1 - z_{sp1}) + \\ &+ A_{s2}E_{s2}\aleph(x_1 - z_{s2})^2 + \\ &+ A_{sp2}E_{sp2}(\aleph(x_1 - z_{sp2}) - \varepsilon_{p02})(x_1 - z_{sp2}) \end{aligned} \qquad (14)$$

where: ε_{p0i} – strain caused by prestressing reinforcement with all the losses.

Tension in normal and prestressing reinforcement:

$$\sigma_{si} = E_{si}\aleph(x_1 - z_{si}); \qquad (15)$$

$$\sigma_{spi} = E_{spi}(\aleph(x_1 - z_{spi}) - \varepsilon_{p0i}). \qquad (16)$$

Substituting expressions (5), (6), (15), (16) in equation (9) – (11), we obtain:

$$N_{cf} = \frac{bf_{cf}\varepsilon_{cf1}}{\aleph} \sum_{k=1}^{5} \frac{a_k}{k+1} \gamma^{k+1}; \qquad (17)$$

$$N_{cft} = \frac{3}{4} bf_{cft}\left(h - \frac{\varepsilon_{cf1}}{\aleph}\right); \qquad (18)$$

$$\begin{aligned} N_s &= A_{s2}E_{s2}\aleph(x_1 - z_{s2}) + \\ &+ A_{sp2}E_{sp2}(\aleph(x_1 - z_{sp2}) - \varepsilon_{p02}) - \\ &- A_{s1}E_{s1}\aleph(x_1 - z_{s1}) - \\ &- A_{sp1}E_{sp1}(\aleph(x_1 - z_{sp1}) - \varepsilon_{p01}). \end{aligned} \qquad (19)$$

Substituting equation (17) – (19) into (7) and after transformations, we obtain the dependence for curvature

$$\aleph = \frac{-b_\Sigma + \sqrt{b_\Sigma^2 - 4a_\Sigma c_\Sigma}}{2a_\Sigma}, \qquad (20)$$

where:

$$\begin{aligned} a_\Sigma &= A_{s1}E_{s1}z_{s1} + A_{s2}E_{s2}z_{s2} + \\ &+ A_{sp1}E_{sp1}z_{sp1} + A_{sp2}E_{sp2}z_{sp2}; \end{aligned} \qquad (21)$$

$$\begin{aligned} b_\Sigma &= \frac{3}{4} bhf_{cft} - \varepsilon_{cf(1)}(A_{s1}E_{s1} + A_{s2}E_{s2} + \\ &+ A_{sp1}E_{sp1} + A_{sp2}E_{sp2}) + \\ &+ A_{sp1}E_{sp1}\varepsilon_{p01} + A_{sp2}E_{sp2}\varepsilon_{p02}; \end{aligned} \qquad (22)$$

$$c_\Sigma = -\frac{3}{4} bf_{cft}\varepsilon_{cf(1)} - bf_{cf}\varepsilon_{cf1} \sum_{k=1}^{5} \frac{a_k}{k+1} \gamma^{k+1}. \qquad (23)$$

After determining the curvature $\aleph$, its values are substituted into formulas (12) – (14) to determine the moments M_{cf}, M_{cft}, M_s. After that, by formula (8) determine the bending moment M, which corresponds to the curvature $\aleph$. The calculation is performed step by step for each value of relative deformations in the compressed cross-sectional area $\varepsilon_{c(1)}$, which consistently increases in magnitude $\Delta\varepsilon_{c(1)}$.

At each step of the calculation necessary to control the tension in the prestressed reinforcement, which is located in a stretched zone section. To do this, use the diagram "σ - ε" for stressed steel (Fig. 2) [1]. Upon reaching the stress values $\sigma_{sp} \geq f_{pd}$ in the following steps, the stress in the prestressed reinforcement must be determined by the formula [8]

$$\sigma_{sp} = f_{pd} + \left(\frac{f_{pk}}{\gamma_s} - f_{pd}\right) \cdot \frac{\varepsilon_{sp} - \varepsilon_{p0}}{\varepsilon_{ud} - \varepsilon_{p0}} \qquad (24)$$

where:

$$f_{pd} = \frac{f_{p0,1k}}{\gamma_s}; \quad \varepsilon_{p0} = \frac{f_{pd}}{E_p};$$

$$\varepsilon_{sp} = \aleph(x_1 - z_{sp}) - 0.0021.$$

To determine the bearing capacity of SFC with prestressed reinforcement developed an algorithm, which is implemented in the program MathCAD.

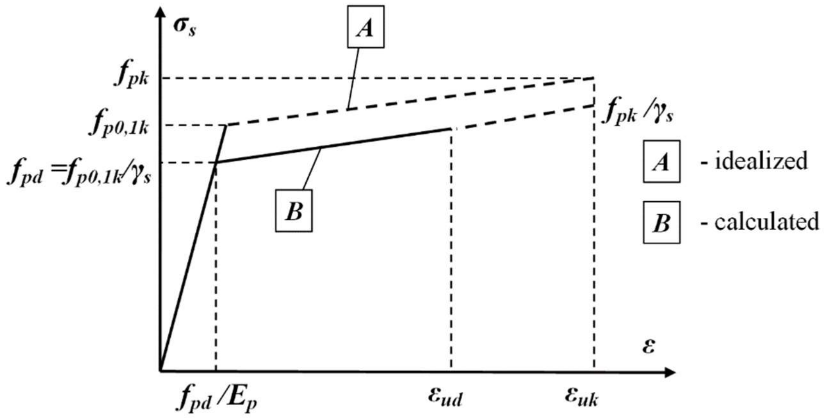

Figure 2 – Idealized and calculated diagram "σ-ε" for stressed steel

Comparative calculation of biaxially prestressed road slab.

Prefabricated reinforced concrete prestressed brands slabs are used for paving P60.38, P60.35, P60.30, which correspond to the current DSTU [4]. Plates of the brand P60.38 have the sizes in the plan 6,0×3,75 m, brand P60.35 – 6,0×3,5 m, brand P60.30 – 6,0×3,0 m and a thickness of 140 mm (Fig. 3).

Slabs are made of concrete class C25/30 and reinforced with pre-stressed reinforcement in two directions (Fig. 3). The plate P60.38 is reinforced with 24Ø10800 fittings located in the longitudinal direction of the plate in two levels and 18Ø12A800 located in the transverse direction of the plate in the center. The plate P60.35 is reinforced with 22Ø10A800, fittings located in the longitudinal direction of the plate in two levels and 18Ø12A800 located in the transverse direction of the plate in the center. The plate P60.30 is reinforced with 20Ø10A800, fittings located in the longitudinal direction of the plate in two levels and 18Ø12A800 located in the transverse direction of the plate in the center.

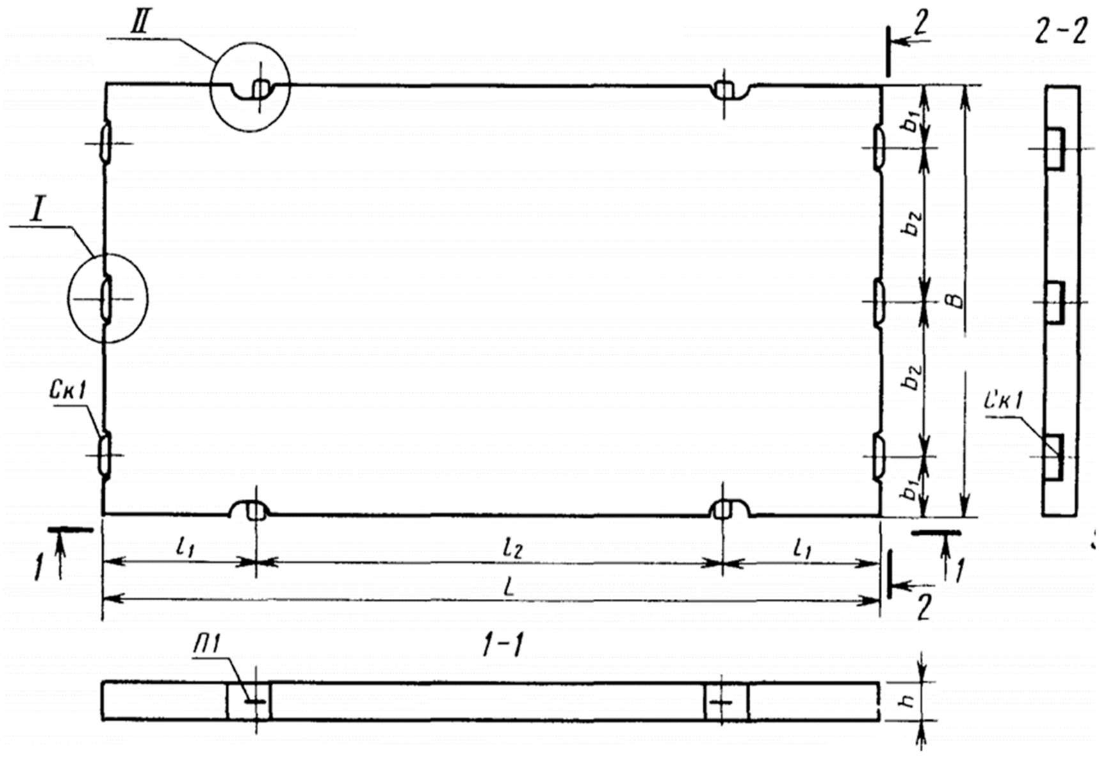

Figure 3 – Prefabricated reinforced concrete slabs of brands P60.38, P60.35, P60.30

Calculation of the bearing capacity of the plate P60.38 without metal fiber

Section dimensions b=3750 mm, h=140 mm (Fig. 4).

Prestressed reinforcement in two levels on 12Ø10A800: A_{sp1}=A_{sp2}=942 mm^2; E_{sp}=190000 MPa; f_{pk}=840 MPa; $f_{p0,1k}$=765 MPa; ε_{uk}=0,018. Elongation of reinforcement taking into account all losses makes ε_{sp0}= -0,002. distance from the center of gravity of reinforcement to the extreme verge compressed section z_{sp1}=105 mm, z_{sp2}=35 mm. Reinforcement pitch along the axis perpendicular to the calculated one S=350 mm.

Concrete class C25/30: f_{cd}=17,0 MPa, E_{cd}=25000 MPa, ε_{c1}=0,00169, ε_{cu}=0,00328.

Polynomial coefficients:

a_1	a_2	a_3	a_4	a_5
2,7404	-2,7649	1,3416	-0,35004	0,03295

The calculation results are summarized in the graph "moment-curvature", which is shown in Fig. 5. The plate bearing capacity is M_u=144,9 kNm.

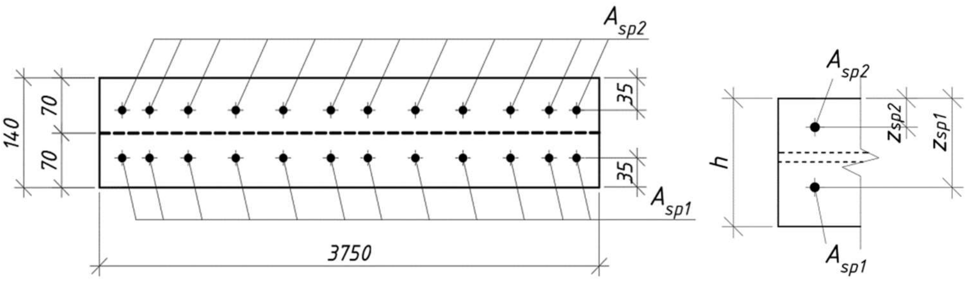

Figure 4 – The calculated cross-section of the plate P60.38 (a) and calculated parameters (b)

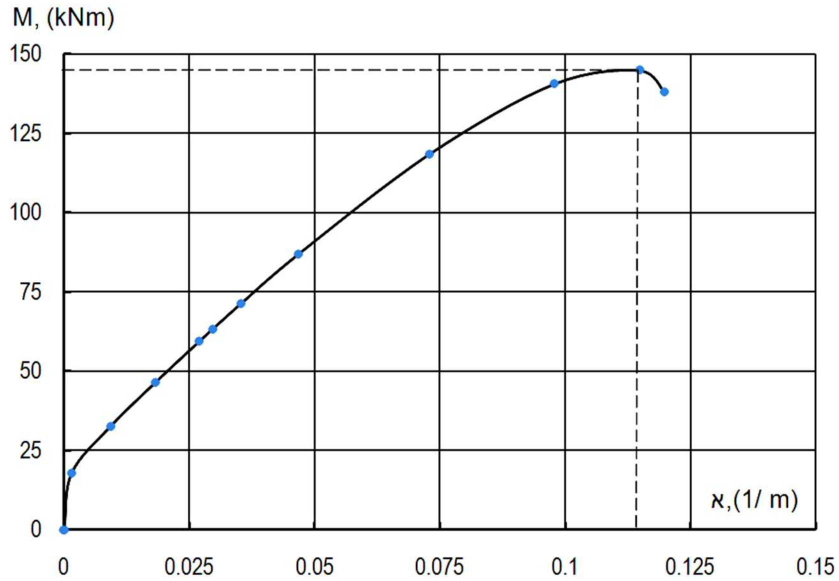

Figure 5 – Graph "moment-curvature" when calculating the plate P60.38 without metal fiber

Calculation of the bearing capacity of the plate P60.38 with metal fiber

Section dimensions b=3750 mm, h=140 mm (Fig. 4). Prestressed reinforcement in two levels on 8Ø10A800: A_{sp1}=A_{sp2}=628 mm^2; E_{sp}=190000 MPa; f_{pk}=840 MPa; $f_{p0,1k}$=765 MPa; ε_{uk}=0,018. Elongation of reinforcement taking into account all losses makes ε_{sp0}= -0,002. distance from the center of gravity of reinforcement to the extreme verge compressed section z_{sp1}=105 mm, z_{sp2}=35 mm. Reinforcement pitch along the axis perpendicular to the calculated one S=350 mm.

Concrete class C20/25: f_{cd}=14,5 MPa, E_{cd}=23000 MPa; ε_{c1}=0,00165, ε_{cu}=0,00344.

Metal fiber STAFIB 50/1.0: f_f=1000 MPa; l_f=50 mm; d_f=1 mm; μ_{fv}=0,01.

The calculated compressive strength of reinforced concrete is determined according to DSTU [6]: f_{cf}=22.36 MPa; f_{cft}=1.49 MPa.

The modulus of elasticity of SFC E_{cf}=24940 MPa.

Polynomial coefficients:

a_1	a_2	a_3	a_4	a_5
2,51816	-2,14804	0,71003	-0,04839	-0,03169

Relative SFC deformations at compression ε_{cf1}=0,00176; ε_{cftu} =0.00293.

Relative SFC strains at tension ε_{cf1}=0,00018; ε_{cftu} =0.00035.

The calculation results are summarized in the graph "moment-curvature", which is shown in Fig. 6. The plate bearing capacity is M_u=180,3 kNm.

As a comparative bearing capacity calculation result of the standard road slab П60.38 and a similar plate with metal fiber, it was found that the bearing capacity of the plate with steel fiber is higher than the standard by 24.4%.

The steel fiber plate efficiency is that the steel fiber makes it possible to reduce the number of high-strength reinforcement from 24Ø10A800 to 16Ø10A800. At the same time, the plate bearing capacity with steel fiber is much higher. The number of high-strength reinforcement in the transverse direction is also reduced by 15…20%.

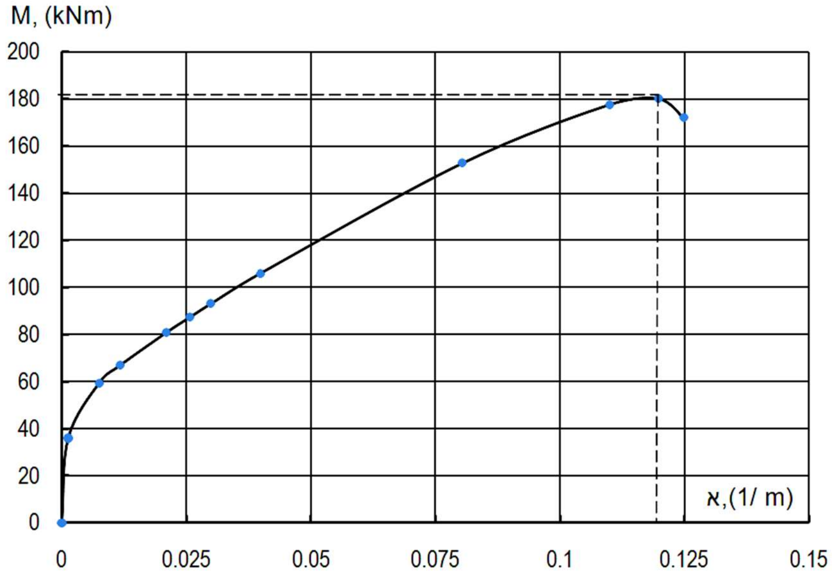

Figure 6 – Graph "moment-curvature" when calculating the plate P60.38 with metal fiber

Conclusions

The general algorithm of bending elements calculation of rectangular section reinforced by usual and prestressed reinforcing rod, and also steel fiber is offered.

The calculation method is based on the deformation theory of reinforced concrete structures calculation taking into account the complete diagram "σ-ε" for concrete and reinforced concrete under compression.

The method makes it possible to calculate biaxially prestressed plates. This takes into account the increase in strength of concrete and steel-fiber-concrete in the biaxial compression conditions.

As a result of comparative bearing capacity calculation of the standard road slab P60.38 and a similar plate with metal fiber, it was found that the plate bearing capacity with steel fiber is higher than the standard by 24.4%.

The plates' efficiency with steel fiber is that steel fiber makes it possible to reduce the number of high-strength prestressed reinforcement to 10…15%.

Due to the good anti-abrasion properties of reinforced concrete, the service life of aerodrome and road surface slabs is much longer than reinforced concrete.

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Civil building frame-struts steel carcass optimization by efforts regulation

Semko Oleksandr [1], Hasenko Anton[2]*, Filonenko Olena[3], Mahas Nataliia[4]

[1] National University «Yuri Kondratyuk Poltava Polytechnic» https://orcid.org/0000-0002-2455-752X
[2] National University «Yuri Kondratyuk Poltava Polytechnic» https://orcid.org/0000-0003-1045-8077
[3] National University «Yuri Kondratyuk Poltava Polytechnic» https://orcid.org/0000-0001-8571-9089
[4] National University «Yuri Kondratyuk Poltava Polytechnic» https://orcid.org/0000-0002-4459-3704
*Corresponding author E-mail: gasentk@gmail.com

With steel structures help it is possible to construct buildings with individual dimensions and different functions, using typical design solutions. The increase in the load-bearing building structures unification level is facilitated by the use of the same transverse frames, which are installed with an equal step. It is possible to ensure the frame stiffness in its own plane by installing struts between the column and the beam. In this case, the crossbar must be calculated as a beam on the hinged supports on the frame columns and on the intermediate elastic supports with a given predetermined stiffness on the struts. By adjusting the struts stiffness and their installation scheme, it is possible to adjust and optimize the stress along the length of the crossbar

Keywords: civil building, frame-struts steel carcass, design scheme optimization, internal forces regulation

Оптимізація регулюванням зусиль рамно-підкісного сталевого каркасу цивільної будівлі

Семко О.В.[1], Гасенко А.В.[2*], Філоненко О.І.[3], Магас Н.М.[4]

[1, 2, 3, 4] Національний університет «Полтавська політехніка імені Юрія Кондратюка»
*Адреса для листування E-mail: gasentk@gmail.com

Сталеві конструкції дозволяють споруджувати будівлі з індивідуальними розмірами та різного функціонального призначення, використовуючи при цьому типові архітектурно-конструктивні рішення. Підвищенню рівня уніфікації несучих будівельних конструкцій сприяє використання однакових поперечних рам, що встановлюються з рівним кроком. Жорсткість рам у власній площині забезпечується або встановленням системи вертикальних і горизонтальних в'язей (при цьому вузли спирання балок на колони виконуються шарнірними), або влаштуванням жорстких вузлів між балками та колонами. Ще одним ресурсоефективним способом забезпечення жорсткості рами у власній площині є встановлення підкосів між колоною й балкою. Завдяки такому рішенню підвищується жорсткість кожної рами та збільшується робочий внутрішній простір будівлі за рахунок відсутності вертикальних в'язей між колонами у поперечному напрямку будівлі. Нерозрізний ригель перекриття у цьому випадку необхідно розраховувати як балку на шарнірних опорах на стійках рами та на проміжних пружних опорах із заданою попередньо визначеною жорсткістю на підкосах. Напруження по довжині ригеля перекриття можна регулювати та оптимізувати за рахунок зміни жорсткості підкосів і схеми їх установлення,. Показано, що нехтування різною жорсткістю стійок і підкосів завищує в декілька разів опорні моменти на підкосах. Це зі свого боку зменшує прольотні моменти, що загалом приводить до отримання хибних результатів статичного розрахунку. У цілому, використання рамно-підкісної схеми каркаса цивільної будівлі дало можливість ресурсоефективно відрегулювати внутрішні зусилля по довжині основних елементів поперечних рам будівлі – стійок і ригелів. Навіть із врахуванням додаткових витрат сталі на влаштування підкосів витрати металу на несучі рами будівлі зменшено на 6% (0,85 кг/м^2) в основному за рахунок зменшення перерізу на один номер прокатного двотавра ригелів перекриття

Ключові слова: цивільна будівля, рамно-підкісний сталевий каркас, оптимізація розрахункової схеми, регулювання внутрішніх зусиль

Introduction

Reducing typical constructions volume in Ukraine in recent decades requires reducing the cost of individual (single for one object) basic load-bearing buildings elements production or increasing the level of their formative and qualitative parameters unification (for example, structures load-bearing capacity, levels of design loads on buildings floors and roofs, etc.) [1].

Steel structures meet the above requirements as they can be used to construct buildings with individual dimensions and purposes, using standard design solutions [2]. Also, steel load-bearing structures have less weight compared to a relatively heavier reinforced concrete carcass, which reduces the laboriousness and terms of building elements production and installation both due to the lack of "wet" processes and by reducing the required load capacity of assembly and transport equipment and machines [3].

Review of the research sources and publications

The increase in the load-bearing building structures unification level is facilitated by the use of the same transverse frames, which are installed with an equal step [4]. The load-bearing capacity of steel elements in load-bearing building structures thus can be effectively used by enough free adjustment of these sections geometrical parameters, namely a wide range of rolled profiles range and ability to create profitable welded sections [5; 6].

The frames' rigidity in its own plane is provided either by the installation of vertical and horizontal knits systems (in this case, the nodes of bearing the beams on the columns are hinged), or the arrangement of rigid nodes between the beams and columns. Another solution to ensure the transverse rigidity of the frame using hinges is to install struts between the column and the beam [7]. Hinged nodes are easy to manufacture; they transmit only longitudinal and transverse forces. Rigid joints form a frame system that, in addition to linear forces, redistributes angular forces (moments) [8]. Rigid nodes also make it possible to create statically indeterminate frames, with which it is possible to redistribute internal forces between the frame elements [9].

With the computer equipment spread with high technical capabilities availability, investigating the stress-strain state of the building frame elements is advantageous with computer programs of finite element analysis using [10; 11].

Definition of unsolved aspects of the problem

When developing architectural and structural design solutions for any building, the designer can use many possible standard solutions for the installation of both load-bearing and enclosing structures. [12]. Architectural and structural solutions of civil buildings elements can be classified by material, manufacturing and installation technology, etc. Therefore, when designing a building, the designer must necessarily determine the technical and economic indicators and optimize the made decisions. During the design of load-bearing structures, particular attention should be paid to the point loads location [13]. In addition, as mentioned above, the type of node arrangement affects the forces redistribution in the carcass linear elements [7].

Problem statement

The purpose of this work is to determine the feasibility of building frame-struts steel carcass placing, which allows due to the struts arrangement between the columns and beams in the building transverse frames to regulate their internal efforts.

Basic material and results

The analyzed building consists of two parts, which from a structural point of view have the same solution. The building two parts designs were developed independently of each other, so each of the parts has a symmetrical relative to the longitudinal axis transverse section with its own gable roof. During two project coordination in one, there was a necessity of a drainage arrangement longitudinal end (see fig. 1). It was decided to abandon the option of installing one common gable roof with one ridge between the two parts, so as not to rework the finished individual projects.

The left part is four-, and the right – three-span building with a carcass constructive scheme (see fig. 1). The spans' width is 6 meters. The exception is the span of the two parts connected in the axes E-F, the width of which is 1.5 m; in this span, there is end drainage. The step of transverse frames is 6 m. The grid of columns are designed with a step 6×6 m.

The height of the external columns along the axes A, E, F, and K is 3 m. The roof slope is equal to $i = 0{,}1$ (5,70).

The ridge of the left four-span part of the building is arranged on the middle column along the C axis.

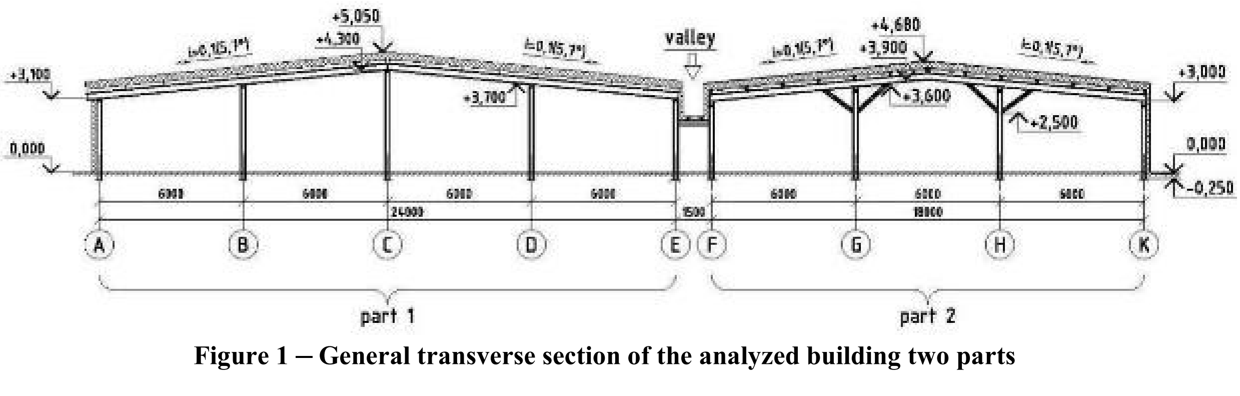


Figure 1 – General transverse section of the analyzed building two parts

The beams’ bottom mark in the ridge in this building part is 4.2 m. The ridge of the building's right three-span part is arranged in the middle of the span G-H. The mark of the bottom of the beams in the ridge in this part of the building is 3.9 m. The wall and roof fencing is made of three-layer light sandwich panels. Light mineral wool insulation is provided between the two profiled flooring sheets. The vertical load-bearing structures of the building frames are steel columns with rectangular transverse sections, made of two rolled channels welded into a "box". Foundations under columns – monolithic reinforced concrete separate shallow foundations of hemp step type are arranged on crushed stone-sand preparation.

Rolled beams of the I-beam cross-section serve as beams. In the left part of the building, the beams are arranged according to a four-span split scheme. In the right part of the building, the beams are arranged according to a three-way continuous scheme.

Nodes leaning on the columns' bases and foundations and nodes contiguity beams to the columns taken hinge (see fig. 2).

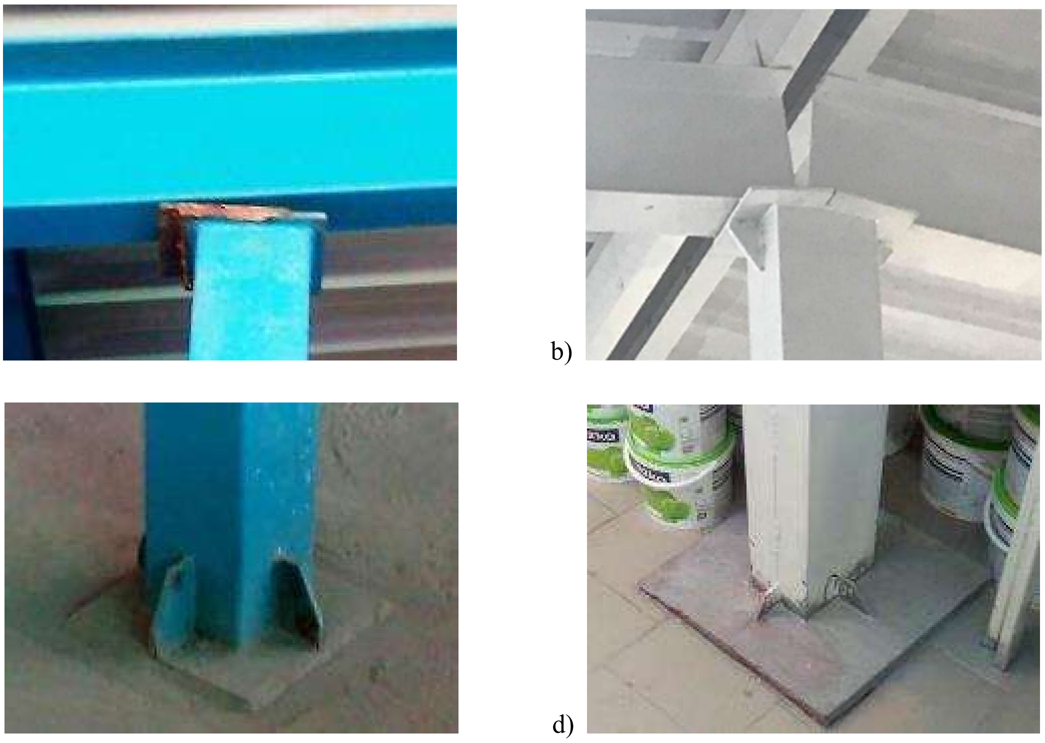
The building rigidity in vertical and horizontal planes is ensured by the system of cross or portal ties installation in one step of the frames. In addition, the rigidity in the frames transverse direction is provided by partial concreting of the bases of the columns and for the second part of the building in the F-K axes by installing struts between the columns and the beams (see fig. 4).

It is the study of the beams' continuity and struts installation influence on the internal efforts in the optimization of the beam, the subject of research in this paper.

The figures 3 and 4 show separate transverse building sections in two parts. Under each transverse section of buildings, the design schemes for which static calculation and definition of internal efforts in frame elements were carried out are given. In this case, the concentrated load on the crossbar from the girders to simplify the calculations is replaced by evenly distributed. This simplification will not make significant changes to the following materials.

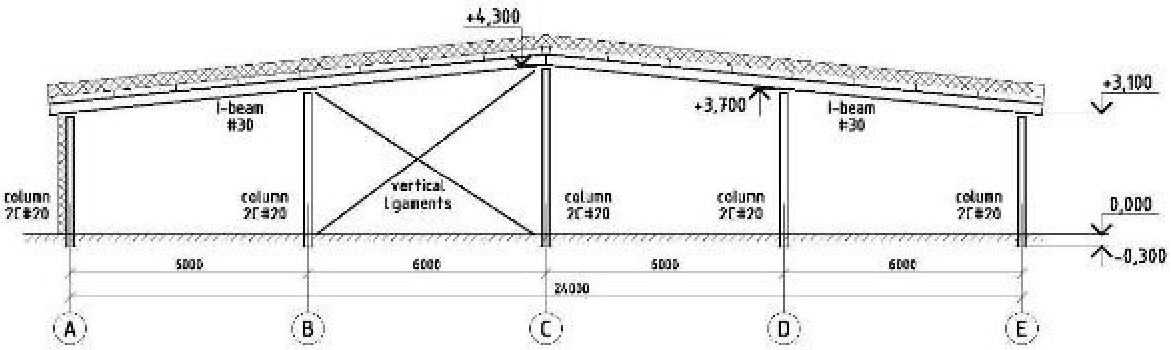
The building covering is made of steel profiled flooring sheets concluded on the girders from rolled channels. Due to the attachment of each wave of the profiled flooring to the upper shelf of the girders with self-tapping screws is provided additional rigidity of the coating disk and performed the perception of the pitched roof horizontal component by a hard disk of profiled flooring and girders, which allowed to reduce the cross-section of the latter.

For the two analyzed variants of building transverse section design schemes, which are shown in figures 2b and 3b depending on the variant of rigidity ensuring for the transverse frame (with vertical ties or struts), it is possible to sketch the "game of internal efforts" - the bending moment M and the longitudinal force N - along the beam length [14]. Herewith for factors of influence on internal efforts value in beam reduction, in both variants, continuous schemes of only 18-meter beams work are accepted. The change in the transverse force V is neglected since the stresses from it are many times less than from M and N.

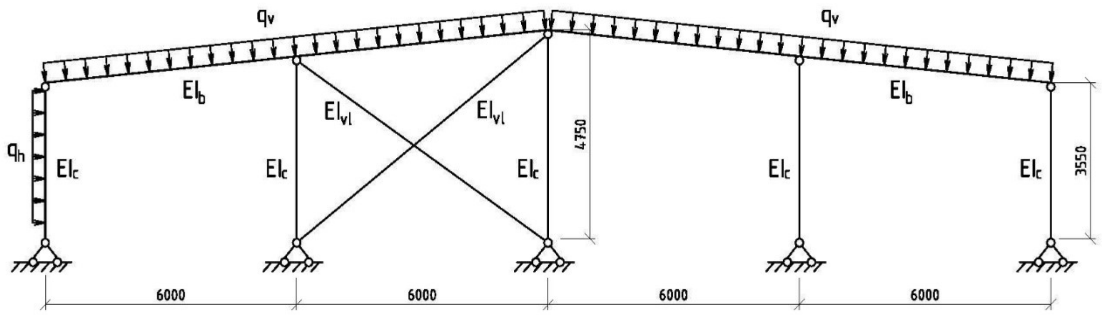


a) b) c) d)

Figure 2 ─ Placing hinged nodes of beams of connection to heads of external columns (a), middle columns (b) and the hinged column bases (c; d)

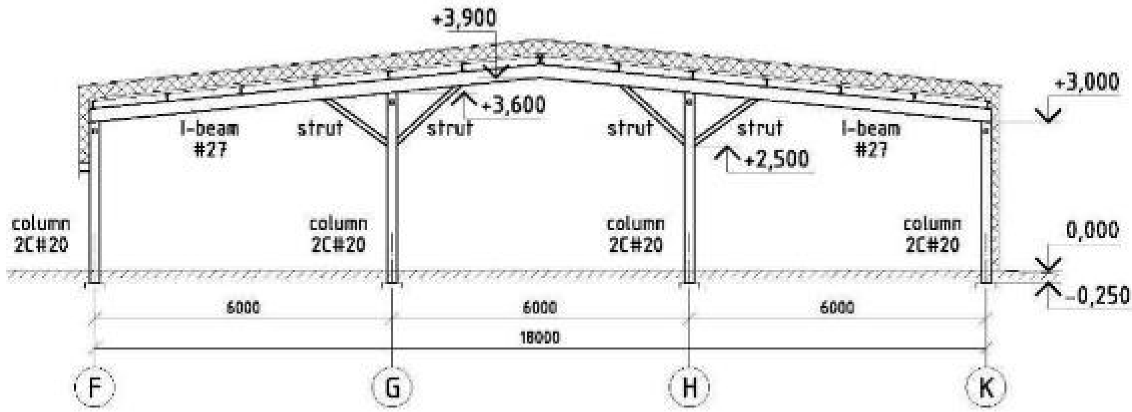


a)

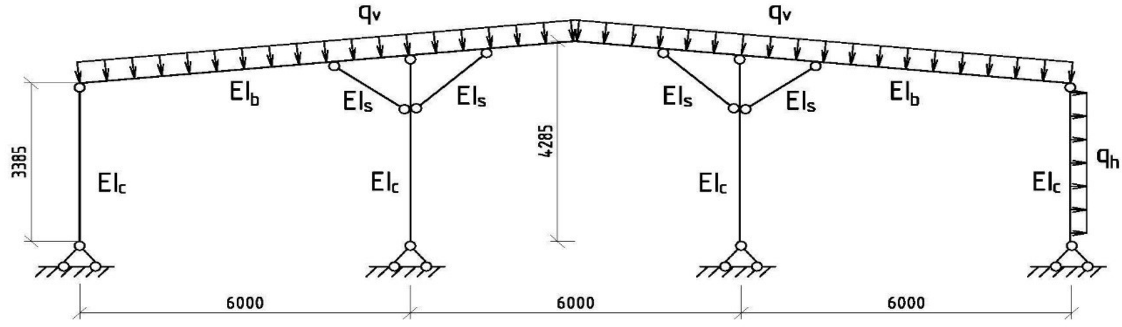


b)

Figure 3 – Transverse section (a) and design scheme (b)
of the left part of the building in the axes A-E



a)



b)

Figure 4 – Transverse section (a) and design scheme (b)
of the right part of the building in the axes F-K

It is known [2] that the normal stresses in the cross-sections beams, working on compression with bending, are determined by the formula

$$\sigma_{\max}=\left|-\frac{N}{\varphi \cdot A_n} \pm \frac{M_x}{c_x \cdot W_{xn.\min}}\right| \leq \gamma_c \cdot R_y . \qquad (1)$$

In the case of providing frame transverse rigidity with vertical ties (see fig. 3), the crossbar occurs predominantly at a bending moment, and the longitudinal force is negligible and occurs only due to the beam slope (see fig. 5). The moments along the crossbar length diagram are two-extreme. The first extreme M_{sp} occurs in the extreme spans in the lower fibers of the beam (in this area the lower fibers of the beam are stretched). The second extreme M_{sup} occurs in the upper fibers of the beam on the middle columns of the frame (in this area the upper fibers of the beam are stretched). Extreme bending moment values in extreme spans M_{sp} and on the middle support M_{sup} are different, the largest of which is M_{sup}. That is, we have two design cross-sections along the continuous 18-meter crossbar length only on the middle supports.

For the first design scheme, in addition to the design cross-sections occurring on the middle supports, we will have another problematic issue from constructive considerations. Since the floor beams are arranged on top of the beams, we will have on the middle supports not unfastened from the frame plane the lower compressed fibers of the floor beams [15]. To solve this problem, it is necessary to either arrange additional ties between the frames in the specified places or weld additional steel plates on the lower shelves of the I-beams.

The design moments M_{sup} on the middle supports of the floor beam can be reduced by using a frame-struts scheme of the transverse building frame (see fig. 4). In this case, it is possible to achieve a diagram of M along the beam length with four calculated sections instead of two. That is to align the values of the span and support moments by aligning the steps of the crossbar supports "rack-strut-rack" installation. In [7] it is proved that the optimal angle of the struts inclination relative to the vertical in terms of internal forces in the strut is equal to 40…50°. This angle of the struts inclination to the uniform supports step "rack-strut-rack" lowers the point of the attachment of the struts to the rack too low in the working space of the room, which is impractical for internal space of premises free planning. Therefore, the point of the attachment of the struts to the rack is performed at a mark close to the mark of the extreme columns head (usually not less than 2.5 m – see fig. 4, a). With such a design scheme, the value of the bending moment M_{sup} on the middle support decreases by 2-3 times, and the design cross-section becomes the span moment M_{sp}, as shown in fig. 6. That is, in this case, we will also have two design cross-sections along the continuous18-meter crossbar length, but no longer on the middle supports but in the extreme spans. The design values of the span moments will be lower than in the first case (see fig. 5) by ~ 42%.

When using the frame-struts design scheme of the transverse frame in cross-sections of crossbar between points of struts fastening to middle columns we will have sites of a local increase in longitudinal force (see fig. 6). That is, with a significant decrease in the value of M on the middle supports, we will have a jump on the diagram N. The stresses in the beam cross-sections, determined by the formula (1), will change not only in proportion to the change in bending moment but abruptly in places of abrupt change in the diagram N.

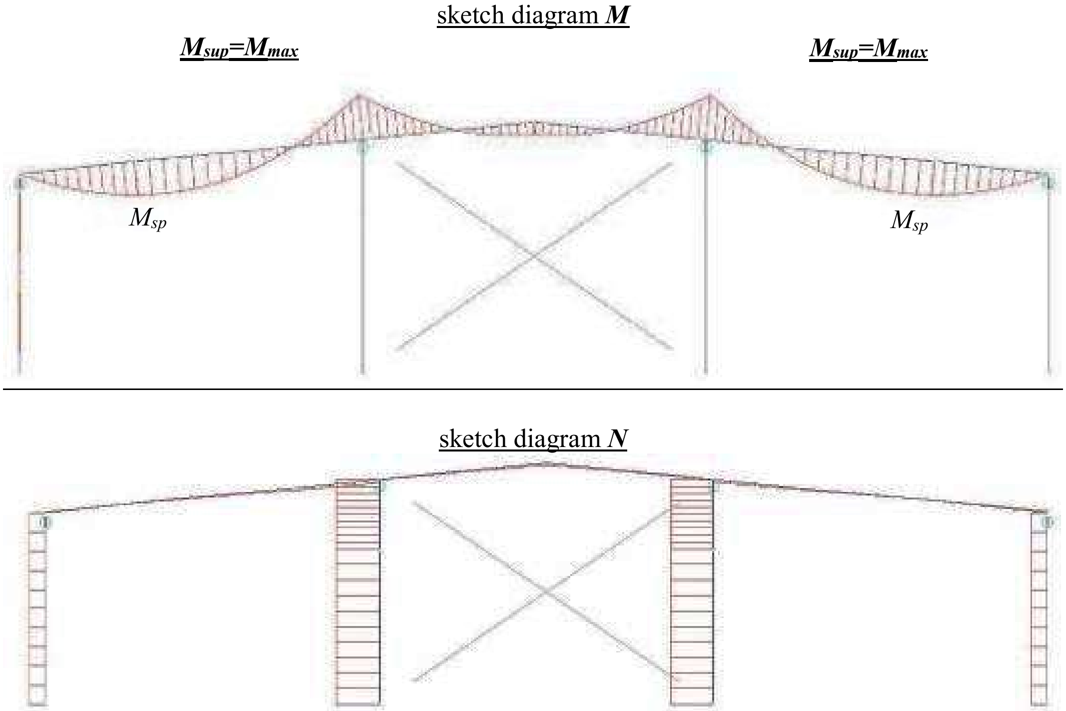


Figure 5 – Sketch diagrams of internal efforts for the transverse frame with vertical ties

Also, when using struts, you need to keep in mind the concentration of bending moment in the frame columns at the points of struts attachment (see fig. 6), as well as the growth of the longitudinal force in the middle columns by increasing the load width of the distribution over the area weight. It is possible to provide the local bearing capacity of a rack in a place of struts adjunction either by the device of a cross-cutting gusset or (see fig. 7, a) or by the device of additional vertical plates (see fig. 7, b).

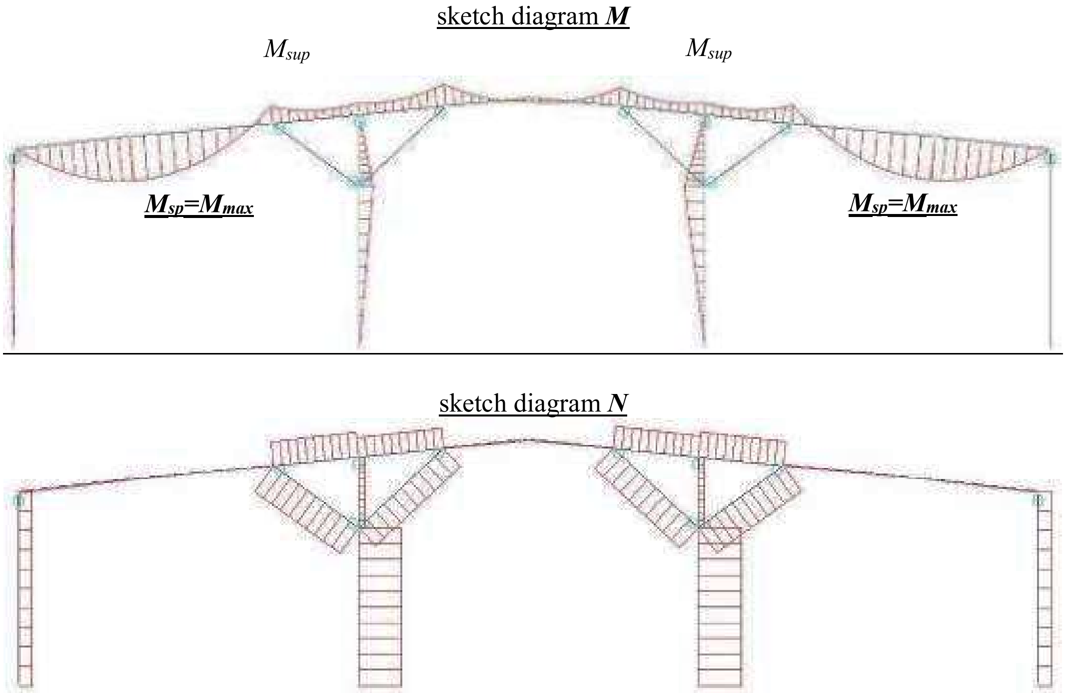


Figure 6 – Sketch diagrams of internal efforts for the transverse frame with struts

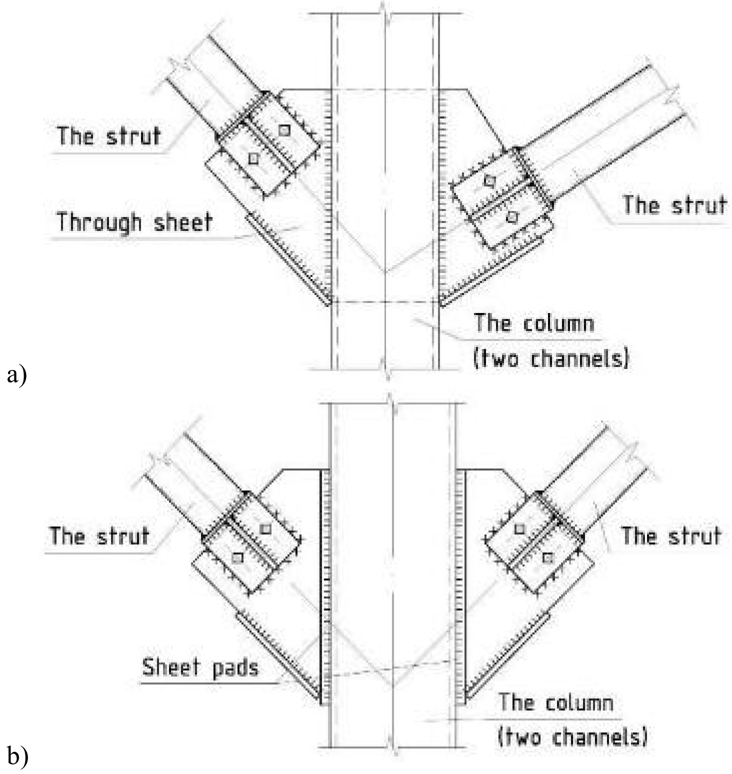


Figure 7 – Nodes of struts to columns connection:
a) using cross-cutting gusset; b) using additional vertical plates

Also on change of internal efforts in beam cross-sections will be influenced not only by the geometrical scheme of struts installation but also struts rigidity (cross-sectional dimensions). Usually, the struts have a smaller cross-section than the frame columns. Therefore subsidence (vertical displacements) of beam supports in places of frame columns and struts will not be identical, which will affect the distribution of the bending moment along the beam length. In this case, the design scheme of the beam will be shown more correctly as proved in [4] in the form of a beam on hinged supports on the frame struts and on intermediate elastic supports with a given predetermined stiffness on the struts (see fig. 8).

Figure 9 shows a comparison of the determining the bending moment values results for the 18-meter beam on the hinged supports on the frame columns and the intermediate elastic supports on the struts and all hinged supports. From the presented diagrams it is clear that the type of supports significantly affects the distribution of internal efforts along the beam length. As expected, the replacement of elastic supports with hinged ones up to twice the bearing moments in these places. This in turn reduces the span moments that generally lead to obtaining false results of the static calculation. Therefore, when determining the internal efforts in the continuous crossbar, it is necessary to take into account the actual stiffness of all supports.

Thus, by adjusting the struts stiffness and their installation scheme, it is possible to adjust the stress in the beam. This is the skill of obtaining a cost-effective construction. For this variant of the transverse frame, the use of the frame-struts scheme allowed to reduce by one number the cross-section of the beam from the rolled I-beam #30 to the I-beam #27.

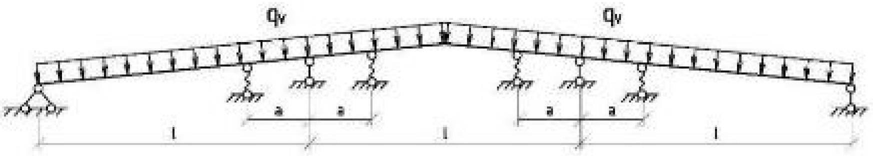


Figure 8 – The design scheme of the crossbar with intermediate spring support (struts)

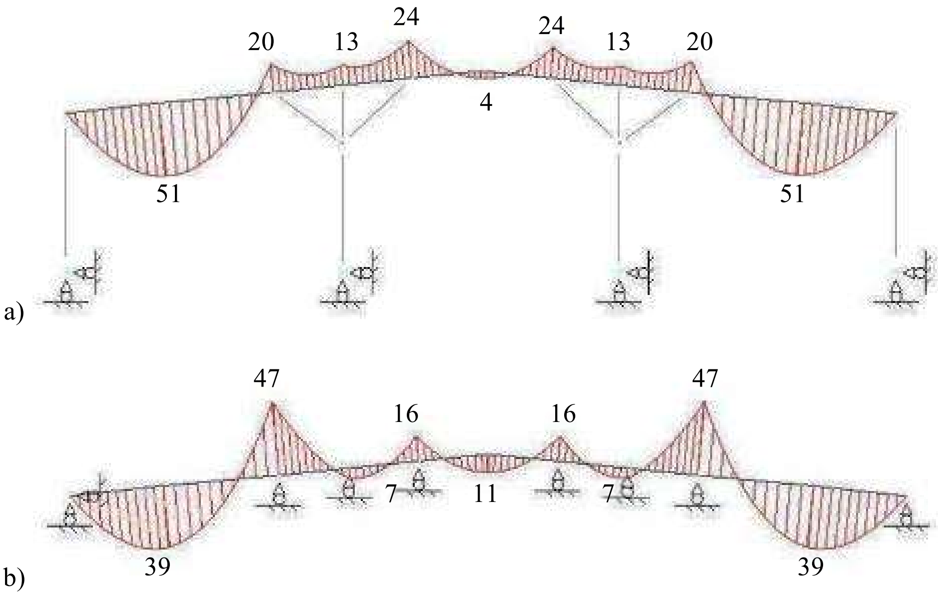


Figure 9 – Diagrams of bending moments in the 18-meter crossbar, kNm:
a) on hinged supports on frame columns and intermediate elastic supports on struts;
b) on all hinged supports

Conclusions

The use of a civil building frame-struts carcass scheme made it possible to efficiently adjust the internal efforts along with columns and beams, namely:

– even taking into account the additional costs of steel for struts installation, are reduced metal costs for the building load-bearing frames by 6% (0.85 kg/m^2) mainly by reducing the cross-section of the beam of rolled I-beam by one number;

– the rigidity of the frame in the transverse direction is increased due to the struts installation;

– is increased working space of the building due to the absence of vertical ties between the columns in the transverse direction of the building.

In further research, it is planned to carry out the mathematical description of beam work on hinged support on columns and on intermediate elastic supports with the set predetermined rigidity on struts.

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Transformation of the retaining wall external geometry with rationalizing of system parameters

Kalmykov Oleg[1]*, Khalife Rabih[2], Demianenko Ivan[3]

[1] O.M. Beketov National University of Urban Economy in Kharkiv https://orcid.org/0000-0001-7294-4279
[2] O.M. Beketov National University of Urban Economy in Kharkiv https://orcid.org/0000-0002-8800-0325
[3] O.M. Beketov National University of Urban Economy in Kharkiv https://orcid.org/0000-0001-9511-7663
*Corresponding author E-mail: olegkalmikov010@gmail.com

The article presents the variable task formulation and implementation of finding a rational outline of the waiting wall back face. In the Coulomb theory framework, an analysis is made of a system consisting of a retaining structure and soil pressing on it for the formulating a rational design problem possibility. The possibility of formulating the problem of finding the rational rear face geometry of the retaining wall within a given horizontal projection is shown. The substantiation of the energy rationalization method operation in solving the problem under consideration is given. The proposed approach allows a variable method to determine the surface configuration of the retaining wall, rational from the standpoint of the accepted criterion. The example given in the work clearly proves the correctness of the problem and its solution statement.

Keywords: retaining wall, curved surface, approximation, algorithm construction, variation approach

Трансформація зовнішньої геометрії конструкції підпірної стіни при раціоналізації параметрів системи

Калмиков О.О.[1]*, Халіфе Р.[2], Дем'яненко І.В.[3]

[1, 2, 3] Харківський національний університет міського господарства ім. О.М. Бекетова
*Адреса для листування E-mail: olegkalmikov010@gmail.com

Подано постановку і реалізацію варіативної задачі пошуку оптимального обрису задньої грані підпірної стіни. У рамках теорії Кулона проаналізовано систему, що складається з підпірної конструкції й ґрунту, що давить на неї, щодо можливості постановки завдання раціонального проектування. На простому прикладі показана можливість постановки задачі пошуку раціональної геометрії задньої грані підпірної стіни в рамках заданої горизонтальної проекції. Наведено обґрунтування експлуатації енергетичного методу раціоналізації при розв’язанні наведеної задачі. Суть пропонованого методу пошуку раціональної геометрії задньої грані підпірної стіни полягає в апроксимації поверхні підпірної стіни ламаною лінією. Для кожної ділянки ламаної лінії виведені ключові залежності по впливу на характер напружено-деформованого стану конструкції, зокрема в цій постановці, на величину згинального моменту в затисканні. Виведено ключові залежності й описано алгоритм розв’язання задачі. Показано, що при заданих характеристиках засипки величина моменту в затисканні фактично може бути описана через комбінацію кутів нахилу кожної з ділянок, а в загальному вигляді таких комбінацій безліч. Задача зведена до пошуку такої комбінації кута α_i, при якій введений критерій (у розглянутій постановці момент у затисканні) займе своє значення внизу. Реалізація підходу продемонстрована на чисельному прикладі. Запропонований підхід дозволяє варіативним методом визначати конфігурацію поверхні підпірної стіни, раціональну з позиції прийнятого критерію. Наведений у роботі приклад наочно доводить коректність постановки задачі та її розв’язання. Використання цього методу доцільне в інформаційному середовищі обчислення. Зокрема, практичне застосування наведеного підходу можливе шляхом постановки і розв’язання завдання лінійного програмування симплекс-методом.

Ключові слова: підпірна стіна, криволінійна поверхня, апроксимація, побудова алгоритму, варіаційний підхід

Introduction

Considering the search for optimal forms of load-bearing elements, it can be recognized as useful "excursions" for specialists who solve complex problems of generating constructive and architectural forms to botany, biology and even physiology. Technical decisions can be effectively parallel with the evolutionary nature of "experience" for a period of tens of millions of years. Construction of optimal forms that are fundamentally close in their constructive geometry to natural bearing (self-bearing) objects of living nature. At the same time, faced with significant cumbersomeness and complexity of mathematical operations generated by the nonlinearity of physical and geometric relationships between the parameters that determine the solution. Considering the rapid development of the IT sphere in recent years, in particular, in the field of building structures design, there is a wide opportunity for creating algorithmic, software methods for finding the optimal forms of load-bearing elements.

Review of research sources and publications

Structural elements of buildings and structures that perceive the sideload from a bulk material refer to systems at which the magnitude and nature of the load directly depend on the configuration of the element that receives this load. The generally accepted theory of the pressure of an incoherent loose material on a side surface, in particular, soil on retaining walls, is the Coulomb theory. [1]. According to this theory, the pressure of the bulk material on the lateral surface depends on the lateral pressure coefficient λ, which is trigonometric depending on the angular parameters of the system (the internal bulk material angle, the angle of wall inclination to the vertical, the angle of filling inclination) [8-12].

In the field of structures research that perceive the side pressure of a bulk material, an exact solution is known in the form of the 4th-degree equation [4, 5]. This equation describes the mutual configuration influence of the bulk material side pressure plot and the curvature of the surface receiving this pressure. This solution is obtained for a static statement. According to the solution, the slope angles of the tangent curve are set, which provides a given pressure profile. Thus, the magnitude and nature of the pressure depend on the surface curvature, and vice versa.

Definition of unsolved aspects of the problem

The described solution demonstrates the mathematical relationship between the curvature of the surface that perceives lateral pressure from a granular medium and the distribution nature of this pressure. At the same time, this solution assumes only a numerical relationship between the parameters mentioned. There is no analytical relationship between the functional describing the configuration of the retaining wall and the lateral pressure plot. At the same time, each individual retaining wall configuration has a unique lateral pressure distribution and hence a unique distribution of internal forces. If the system "retaining wall - backfill soil" is constrained, for example, in the form of the wall horizontal projection constancy or maximum displacements limitation, it is possible to find its outline, which will predetermine the minimum of the accepted criterion [6,7]. The criterion here can be the volume of material, cost, the maximum principal stresses value, the value of the system deformation potential energy, and others.

Problem statement

Taking into account the presented information, proposed a variable method for finding a rational surface of a retaining wall. The method essence consists etc.

A cantilever retaining wall of arbitrary shape has predetermined external dimensions – horizontal B and vertical H projection of the system. The wall is divided into n linear sections (Fig. 1). Each section has an inherent only slope angle α_i, which lies in the range $\alpha_i \in [\varphi; 90°]$. The horizontal projection of the system defined as:

$$B = \sum_{1}^{n} \frac{h}{tg(\alpha_i)} . \qquad (1)$$

The distributed pressure of the bulk material at the base and top of each of the sections $q_{1,2}$ is respectively determined by the expressions:

$$q_{i,1} = (H - h \times i)\, \gamma\, \lambda_I ; \qquad (2)$$

$$q_{i,2} = (H - h\,(i - 1)\, \gamma\, \lambda_i ; \qquad (2)$$

where: γ – the bulk density;

λ_i – the lateral pressure coefficient of the bulk material.

The distributed pressure of the bulk in the area of the site is reduced to a concentrated force acting normal to the site (Fig. 2), which is defined as

$$Q_i = \frac{q_{i,1} + q_{i,2}}{2} \times \sin(\alpha_i) \times h . \qquad (4)$$

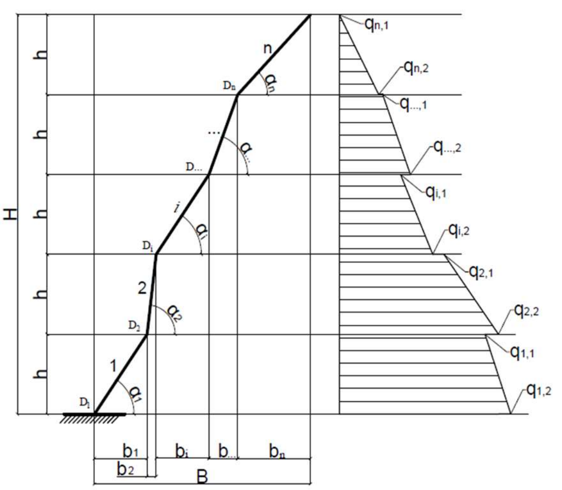

Figure 1 – Curvilinear generatrix approximation

The point coordinates of the concentrated force application relative to the section base are determined by the expressions:

$$h_{0,i} = h \times m \;; \tag{5}$$

$$b_{0,i} = \frac{h_{0,i}}{tg(\alpha_i)} \;; \tag{6}$$

$$l_{0,i} = \frac{h_{0,i}}{\sin(\alpha_i)} \;; \tag{7}$$

where:

$$m = \frac{2q_{i,1} + q_{i,2}}{2(q_{i,1} + q_{i,2})} \;. \tag{8}$$

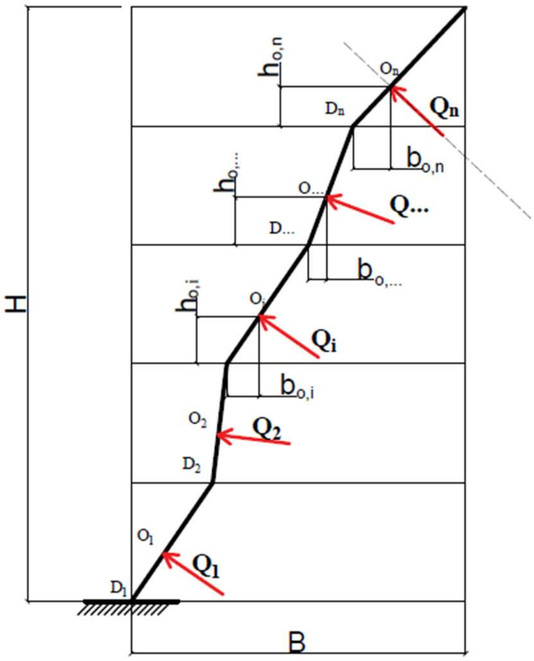

Figure 2 – The pressure of the bulk in the form of concentrated forces

The diagram of the bending moment from the action of the *j*-th force will have the form shown in Fig. 3. The bending moment magnitude $M_{i,j}$ at the base of the *i*-th section (point D_i) from the action of the *j*-th force is determined as

$$M_{i,j} = Q_j \times D_i C_{i,j} \;; \tag{9}$$

where $D_iC_{i,j}$ – normal from the base of the *i*-th section (D_i) to the vector of the *j*-th force (Fig. 3), determined by the expression:

$$D_i C_{i,j} = \left[\frac{b_{o,j} + \sum_i^{j-1} b_i}{tg(\alpha_i)} + h \cdot (j - i) + h_{o,j} \right] \sin(\alpha_i) \,. \tag{10}$$

The general diagram of moments is the sum of all diagrams M_j, from the force Q_1 to the force Q_n.

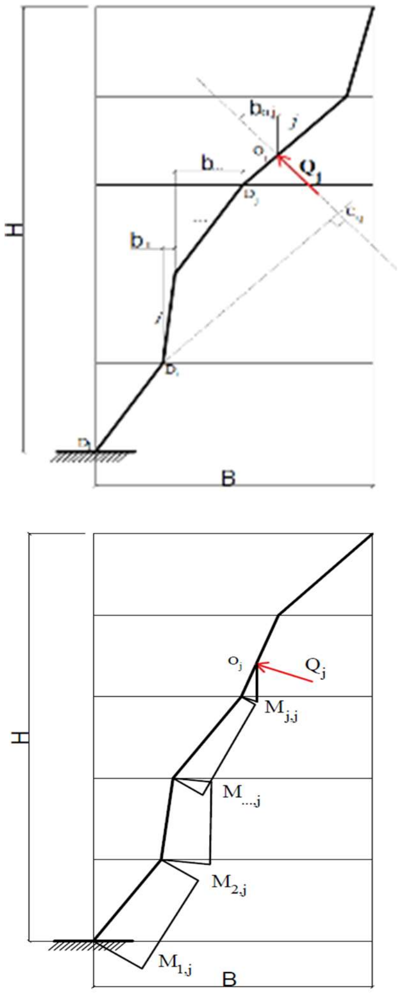

Figure 3 – Plotting bending moments

The energy criterion of rationalization is introduced into consideration. As applied to the problem, its interpretation is as follows: within the given constraints, it is necessary to find a retaining wall configuration, in which the potential energy of deformation (PED) will take a minimum value [2, 3].

Analytically, the potential energy of deformation can be represented as the sum of the partial (PED) $U_{j,tot}$ from the each of forces action in the section from 1 to n of the force

$$U = \sum_1^n U_{j,tot} \;. \tag{11}$$

The partial PEDs $U_{i,tot}$, from the action of the *j*-th force are the sum of the volumes of the truncated pyramids (Fig. 4) in the sections from $i = 1$ to $i = j$. On the sections from $i = 1$ to $i = j-1$ at the base of the truncated pyramid (the volume of the truncated pyramids - U_i) there is a square with sides $M_{i,j}$, at the top - a square with sides $M_{i+1,j}$, the height of the pyramid - l_i. The upper pyramid is a special case - a non-truncated pyramid (the volume of a non-truncated pyramid - U_j) of height lo, j, at the base of which there is a square with an edge $M_{j,j}$. The particular PED values for each of the sections are determined by the expression:

$$U_{j,tot} = U_j + \sum_1^{j-1} U_i \ , \quad (12)$$

where:

$$U_i = \frac{M_{i,j}^2 + \sqrt{M_{i,j} \times M_{i+1,j}} + M_{i+1,j}^2}{3} l_i \ , \quad (13)$$

$$U_i = \frac{M_{j,j}^2 \times l_{o,j}}{3} \ . \quad (14)$$

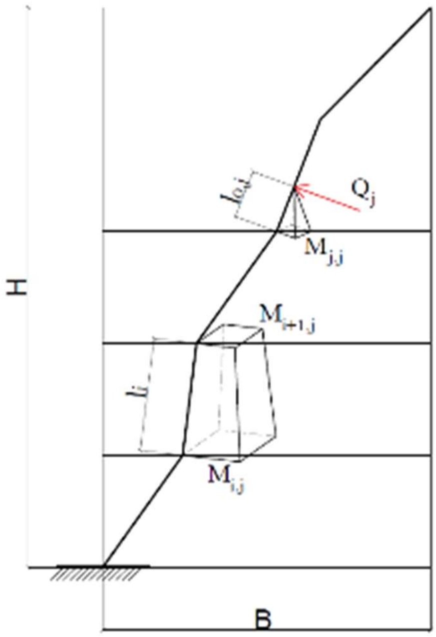

Figure 4 – To the definition of PED from the action of force in the *i*-th section

The arbitrary curvature of the rear face curved generatrix has its unique combination of the inclination angles α_i of each of the sections, which it can be described by. With an increase in the number of sections n in the limit, the broken line tends to a curved outline. With the given characteristics of the bulk (φ, γ), the PED value, in fact, can be described through a combination of the inclination angles αi of each of the sections, and in general, there is an infinite number of such combinations. Varying the configuration of the retaining wall lateral surface, it is possible to find such a geometry at which the PED value will take a lower value. Then the problem is reduced to the search for such a combination of αi, for which the introduced criterion (in the considered formulation, PED) will take its lower value.

Basic material and results

The stated formulation of the problem presupposes an iterative method of finding the rational outline of the retaining wall. With an increase in the steps of dividing the system in the horizontal and vertical directions, the number of virtual variants that form structures increases significantly. To determine the number of variants of the structure generators, depending on the magnitude of the horizontal and vertical steps of the partition, we reduce the problem under consideration to the classical problem of combinatorics.

From each row, you need to select any one number mi so that the sum of all mi equals n -1:

$$\sum_{1}^{i=m} m_i = n - 1 \ .$$

In the table 1 n – the number of columns, m – rows. Each line looks like 0,1,2…*i*.

Table 1 - Determination of the number of variable options

		n				
		1	2	3	…	*j*
m	1	0	1	2	…	*i* - 1
	2	0	1	2	…	*i* - 1
	3	0	1	2	…	*i* - 1
	…	0	1	2	…	*i* - 1
	i	0	1	2	…	*i* - 1

One variation of *C* is a combination of *m*-values from 1 to m_i, which adds up to *n*-1. The problem is formed as follows: for a given *n* and *m*, how many variations of *C* can be constructed.

The solution is determined by dependency

$$C_{n+m-1}^{m-1} = \frac{(n+m-2)!}{(m-1)! \cdot (n-1)!} \ .$$

Using the obtained expression, it is possible to determine the number of variants of generators depending on the number of vertical and horizontal sections of the system partition (Table 2). From the given data it follows that when the system is divided into 10 vertical and 10 horizontal sections, the number of variants forming the system under study is close to 100 thousand. The foregoing leads to the need to search for informational ways to solve the problem.

To implement the solution, a script was written in the Dynamo visual programming environment (Fig. 5).

For a clear demonstration of the operation of the proposed approach, we present the simplest example of its applications. Let us set the initial data of the retaining wall and the soil acting on it: vertical projection H = 10m; horizontal projection B = 5 *m*; the volumetric weight of soil γ = 1.5 t/m^3; angle of internal friction of the soil φ = 30°; the number of subdivisions n = 10.

In accordance with the formulation of the problem, such a combination of the angles of inclination of each of the sections is determined, which will predetermine the minimum PED of the system. To generate the assigned angle values, the Refinery add-in was used, which is aimed at solving such problems. Here you set such parameters as a limitation for the magnitude of the angles, the magnitude of the horizontal projection, the criterion for finding a rational solution, the accuracy of the solution and the number of options under consideration. Out of the calculated 1000 variants, such a configuration was found in which the PED of the system took the minimum value (Fig. 6).

Table 2 – The number of variants of the generating structure

		Number of sections horizontally, *n*									
		1	2	3	4	5	6	7	8	9	10
Number of sections vertically, *m*	2	2	3	4	5	6	7	8	9	10	11
	3	3	6	10	15	21	28	36	45	55	66
	4	4	10	20	35	56	84	120	165	220	286
	5	5	15	35	70	126	210	330	495	715	1001
	6	6	21	56	126	252	462	792	1287	2002	3003
	7	7	28	84	210	462	924	1716	3003	5005	8008
	8	8	36	120	330	792	1716	3432	6435	11440	19448
	9	9	45	165	495	1287	3003	6435	12870	24310	43758
	10	10	55	220	715	2002	5005	11440	24310	48620	92378

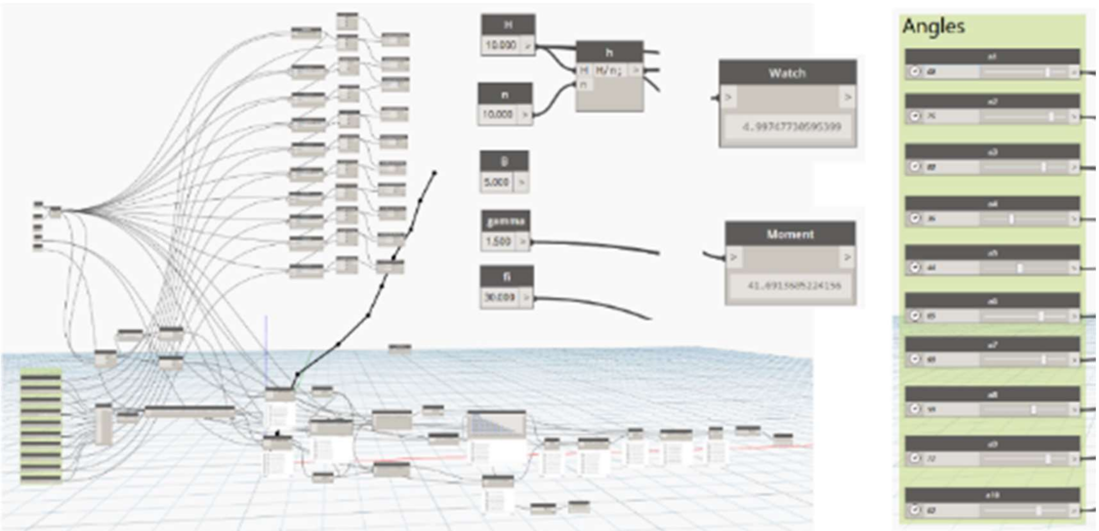

Figure 5 – General view of the Dynamo script

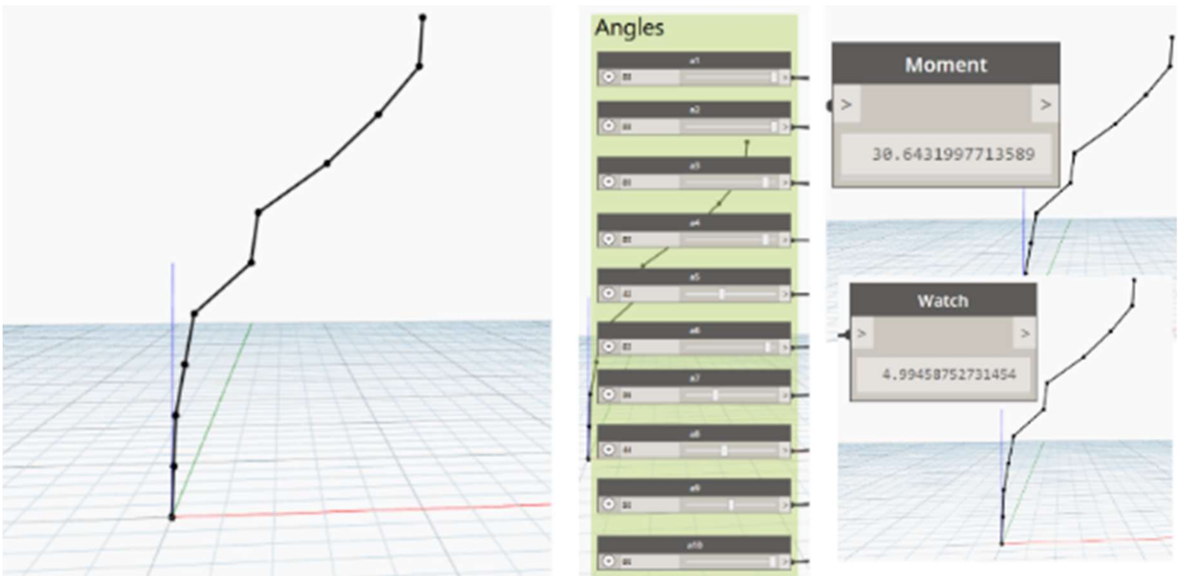

Figure 6 – Solution result

Conclusions

The proposed approach allows using the variational method to determine the configuration of the retaining wall surface that is rational from the standpoint of the accepted criterion. The example given in the work clearly proves the correctness of the problem statement and its solution. In general, with a much larger number of sections n, the broken generatrix of the retaining wall rear face is smoothed and tends to a curvilinear outline. In further research, it is of interest to formulate and solve this problem using linear programming methods.

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Stress strained state change in the «deformed building – pile foundation – base» system resulting from supplying the slab under the grilles

Vynnykov Yuriy[1*], Kharchenko Maksym[2], Manzhalii Sergii[3]

[1] National University «Yuri Kondratyuk Poltava polytechnic» https://orcid.org/0000-0003-2164-9936
[2] National University «Yuri Kondratyuk Poltava polytechnic» https://orcid.org/0000-0002-1621-2601
[3] National University «Yuri Kondratyuk Poltava polytechnic» https://orcid.org/0000-0001-8481-9547
*Corresponding author E-mail: vynnykov@ukr.net

Presented features of a new analytical model of the "deformed building - driven prismatic piles as a part of a continuous grille - soil base with a weak underlying layer" system before and after supplying the monolithic reinforced concrete slab under the existing foundation grilles. Also, this system's stress strained state (SSS) simulation results by the finite element method (FEM) for evaluation of the combined action features of the system's components were presented. It has been published the new empirical data on the SSS change in the "deformed building - driven prismatic piles as a part of a continuous grille - soil base with a weak underlying layer" system resulting from supplying the monolithic reinforced concrete slab under the existing foundation grilles.

Keywords: soil base, poor-bearing soil, driven prismatic pile, monolithic reinforced concrete grille, settlement, crack, stress-strained state, monolithic reinforced concrete slab

Зміни напружено-деформованого стану системи «деформована будівля – пальовий фундамент – основа» внаслідок підведення під ростверки плити

Винников Ю.Л.[1*], Харченко М.О.[2], Манжалій С.М.[3]

[1, 2, 3] Національний університет «Полтавська політехніка імені Юрія Кондратюка»
*Адреса для листування E-mail: vynnykov@ukr.net

Викладено особливості нової розрахункової схеми системи «деформована будівля – забивні призматичні палі у складі стрічкового ростверку – ґрунтова основа зі слабким підстильним шаром» до та після підведення під існуючі ростверки монолітної залізобетонної плити й результати математичного моделювання з використанням методу скінченних елементів напружено-деформованого стану (НДС) цієї системи для оцінювання особливостей спільної роботи її складових. Розрахунок виконувався методом скінченних елементів у просторовій (3D) розрахунковій схемі з урахуванням спільної роботи надземних і підземних конструкцій, пальового фундаменту та основи під ним. При оцінюванні НДС будівлі ґрунтова основа існуючих фундаментів умовно замінювалася відповідними коефіцієнтами. Посилення полягало в підведенні під ростверки залізобетонних балок L-подібного обрису, об'єднаних поперечними балками, а зверху – монолітною плитою товщиною 200 мм. Отримано ребристу плиту підсилення з ребрами до низу, основою якої є пісок намивний, дрібний, середньої щільності. Ця конструкція добре перерозподіляє напруження від нерівномірних деформацій основ і має значну жорсткість за мінімального об'єму земляних робіт. Оприлюднено нові дослідні дані про зміну НДС системи «деформована будівля – забивні призматичні палі у складі стрічкового ростверку – ґрунтова основа зі слабким підстильним шаром» унаслідок підведення під існуючі ростверки монолітної залізобетонної плити. Моделювання НДС системи після посилення фундаменту показало, що фактичне створення плитно-пальового фундаменту значною мірою прибрало нерівномірний характер розподілу напружень і наблизило його до початкового стану. Доведено достатньо високу ефективність та надійність способу посилення пальових фундаментів у складі стрічкового ростверку підведенням плити.

Ключові слова: ґрунтова основа, слабкий ґрунт, забивна призматична паля, монолітний залізобетонний ростверк, осідання, тріщина, напружено-деформований стан, монолітна залізобетонна плита

Introduction

The choice of specific constructive-technological decisions of strengthening (reconstruction) of each deformed building on the pile foundation is carried out only after careful estimation of the technical condition of load-bearing building structures and investigation their bases and foundations parameters, by engineering inspection and establishing the causes of excessive deformation of the foundations' base [1 - 4].

Previously tested constructive-technological decisions of the increasing the load-bearing capacity of pile foundations of deformed buildings and structures by pushing piles to strong soil and installation of offset piles with sufficiently high reliability of their use results have a very high hand labor-intensity and require a long period of work. Therefore, it is advisable to improve adequately efficiency for conditions of sizable uneven deformations of buildings, and, at the same time, less labor-intensive and more prompt decisions to increase the bearing capacity of pile foundations with their or soil base SSS change, such as strengthening of driven prismatic piles foundations in a continuous grille by supplying a monolithic reinforced-concrete slab under the existing grilles [3 – 6].

Review of the research sources and publications

The correctness of soil state models and geomechanical models of 2D and 3D versions of FEM regarding calculations of combined action of piles as a part of continuous and plate grilles with the base is justified [5 – 14]. Thus, the 3D PLAXIS version of the complex has a whole library of nonlinear models of soil mechanical behavior (ideal elastic-plastic with the Mohr-Coulomb strength criterion; isotropic compaction (strengthening); weak creeping soil, etc.), makes it possible to vary the geometry of foundations, track the SSS stages of the base at different paths of loading, has a convenient interface for outputting results. It is well tested for SSS estimation of pile-slab foundations, pile groups combined by a grille of strip pile foundations [7 – 16].

In particular, Professor I. Boyko and his school [13] solved several problems of modeling SSS of the "plate-pile-base" system in the VESNA complex using an elastic-plastic soil model based on the dilatancy theory of V. Nikolaevsky, with the criterion of the plastic flow condition of Misesap-Schleicher-Botkin in Boyko's modification. It has been proved that taking into account the mutual influence of neighboring pile foundations, the deformation behavior of the base significantly changes, increasing the settlement of the foundations of the sectional high-rise building up to 30%, and the values of bending moments in the slabs joint area of the sections increase by 1.5-2 times.

I. Mayevska and N. Blaschuk [14] in the 3D version of PLAXIS investigated the influence of several factors on the bearing capacity and deformability of the system "continuous grille – driven piles of constant cross-section – soil" before and after strengthening by piles jacking: the piles spacing; distances between pile rows; pile lengths; type of piles (with and without removal of soil); soils. It was defined as the fractions of the load perceived by a new grille - from 5 to 65% and the grille, installed in strengthening by piles of the continuous footing - from 30 to 72%. The bearing capacity of pile foundations was estimated by the criterion of the limit value of a settlement.

Specialists of the Poltava geotechnical school [15, 16] by modeling in the 2D and 3D versions of PLAXIS the system "continuous grille – cast-in-place piles in drilled wells – wet loess base" using an elastic-plastic soil model and step-iteration procedures obtained a relative error of 15% compared to long-term natural object leveling data. For both tasks, the possibility of correct accounting for the soil's heterogeneity in the "zone of influence" of piles has been proved. Both simulated and experimental "load - settlement" graphs are curvilinear, that is, with the achievement of a second critical force on the system, the soil around the piles, its broadenings, and under the grille works in the plastic stage. It has also been established that the deformation modulus by the compression tests of wet loess soils should be taken into account during simulation without increasing factors.

Definition of unsolved aspects of the problem

The FEM has the greatest value in solving the so-called "complex geotechnical problems" – with the complex geometry of the design configuration [2, 5, 7, 9, 13], in particular, for estimating the SSS of the "soil mass - existing surrounding buildings - new structures" system in dense urban development conditions and taking into account the stages and technology of operations (fragments of the pit, fixing of its walls, building of the underground part of the structure, loading of it or its components during the erection of surface structures, etc.) as well as complex engineering and geological conditions and adverse effects (overwhelmingly use the software products PLAXIS, DIANA, FLAC, VESNA, FEM models and several others in the 3D version of FEM). Therefore, according to Professor V. Illichov, a new direction of soil mechanics has already been developed - "technological soil mechanics" [5].

Therefore, it makes sense to test modeling with the 3D version of the FEM and the elastic-plastic soil model (Cam-Clay, Soft soil creep model, etc.) for the SSS estimation of the "deformed building – pile foundation (before and after reinforcement) – soil-based" systems.

In addition, when estimating the SSS of the "deformed building – pile foundation – base" systems, significant systemic discrepancies were established in the values of the pile`s bearing capacity calculated by the standard method, according to which soil resistance under the toe and along the side surface is obtained depending on the depth of their immersion, the liquidity index of clay soil or sand`s grading, and by the results of static tests of piles [16].

Problem statement

Therefore, the purpose of the actual work was: to develop a calculation scheme of the "deformed building – driven prismatic piles as part of a continuous grille – a soil base with a weak underlying layer" system before and after supplying a monolithic reinforced concrete slab under the existing grilles and to perform this system's SSS mathematical modeling using the FEM to estimate the features of combined action of its components.

Basic material and results

Put in commission in 1977, the end block of the five-story residential building with a basement in Horishni Plavni of Poltava oblast is resting on driven prismatic piles (9 m length, 350x350 mm section), joined by a continuous grille. The end block has sizable fractures in bearing masonry walls because a part of the piles is driven higher than the designed depth, and its toes are found in fluid sandy loam with silt layers [3].

Aside from that, due to the negative skin friction effect caused by self-compacting and mechanical suffusion in the upper layers of the bulk sands after rupture of the main thermal pipeline, which was intensified by inertial forces from the explosions in the quarry, the design load on pile dropped to $N = 268$ kN, which is less than the stress from the building 404.5 kN. Therefore, the pile foundation base's settlement was already in the nonlinear stage, which led to the emerging and development of respective non-uniform limit-exceeding deformations in it [4].

In structural regard, the building is a structure with longitudinal load-bearing brick-built walls. The height of the floors is 2.8 m, and the basement is 2.2 m. Its spatial rigidity is provided by transverse walls of a stairwell and floor slab disks (fig. 1).

The building's structural scheme cannot be considered rigid, and the technical condition of its pile foundation is qualified as insufficient [6].

The reinforcement consisted of the supply under the grille monolithic reinforced concrete L-shaped beams (900 mm height), joined by transverse reinforced concrete beams, and on top – by a monolithic 200 mm thick slab. The concrete for grille elements strengthening is of C20/25 strength grade.

A ribbed strengthening plate was obtained, which was based on the alluvial fine sand of medium density. Its ribs were directed to the bottom [6]. This design effectively redistributes stress from uneven deformations of the base and has considerable rigidity at the minimum volume of groundworks.

The design was simulated by FEM in spatial (3D) configuration with consideration of combined action of underground and superstructure, pile foundation, and base beneath it. In the evaluation of the building's SSS, the soil base of the existing foundation was conventionally replaced by the respective coefficients.

Spatial FEM configuration of the building block's foundation before its strengthening is shown in fig. 2, and after the strengthening – in fig. 3 and fig. 4 (schematic view fragment).

A simulation was carried out for the following phases:

1) the building's completion moment;

2) after the base wetting (flooding) from the pipeline malfunction;

3) after the foundation strengthening.

As initial conditions, the settlement factor was set to be k=7000 kN/m (fig. 5), for the flooding state the reduction of its load-bearing capacity was considered in certain areas [3], where k=5000 kN/m (fig. 6).

The respective analytical 3D model of the building is shown in Fig. 7 and fig. 8. It includes the building's load and influences data and its physical model (3D system of walls, slabs, beams, its joints, foundation, and base, as well as data on the physical and mechanical properties of materials).

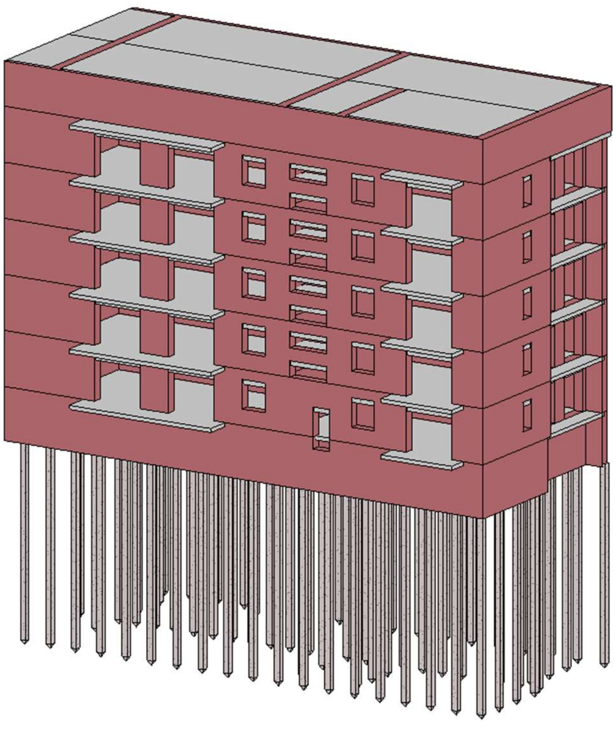

Figure 1 – Spatial (3D) configuration of the building

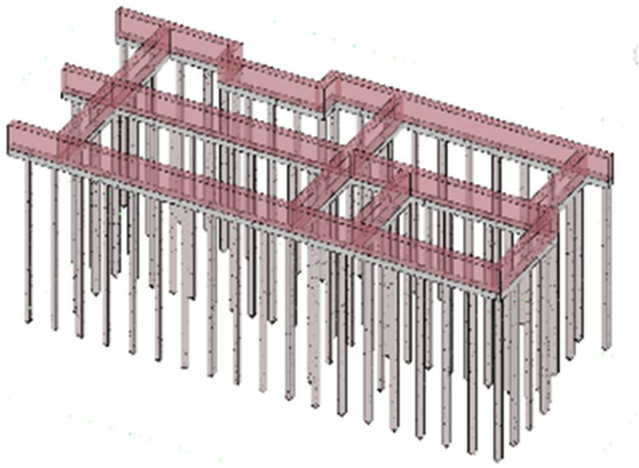

Figure 2 – 3D scheme of foundation before the strengthening

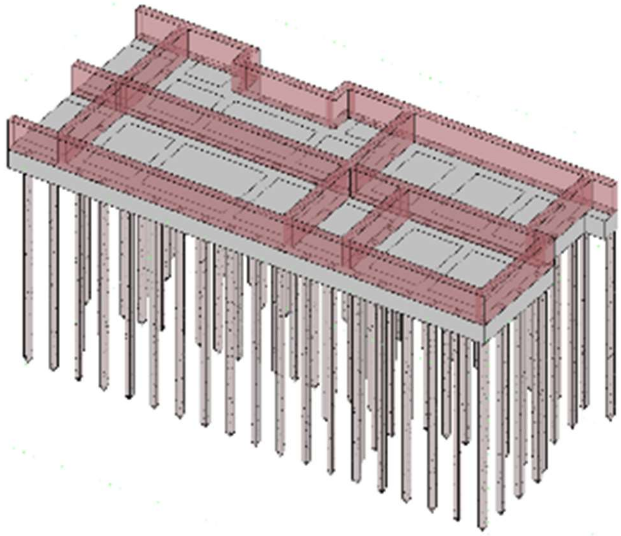

Figure 3 – 3D scheme of the foundation after the strengthening

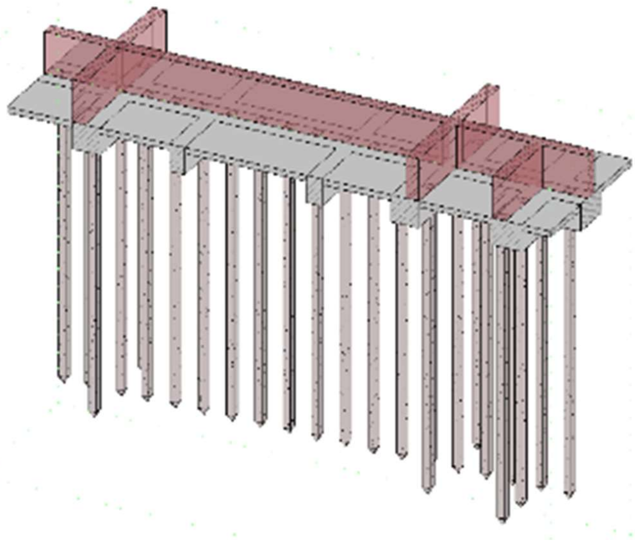

Figure 4 – View fragment of 3D foundation scheme after strengthening (along the inner longitudinal bearing wall)

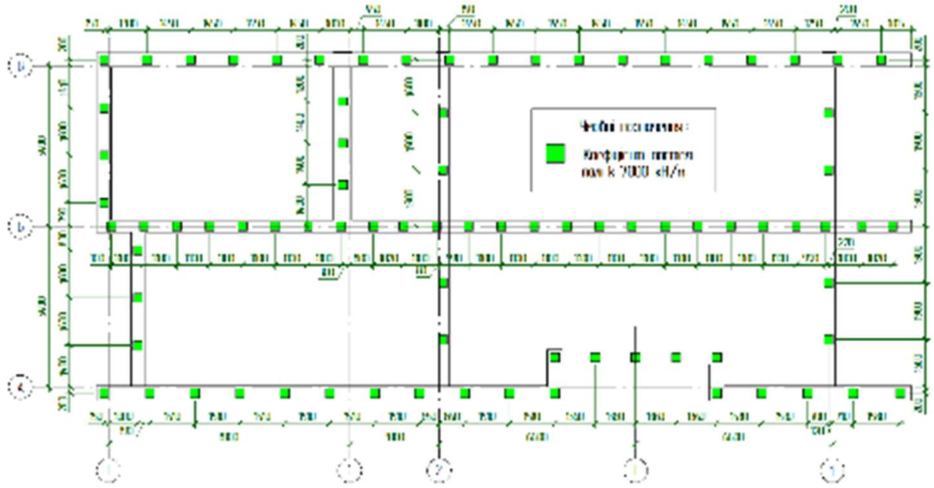

Figure 5 – Piles' initial rigidity, adopted in the simulation

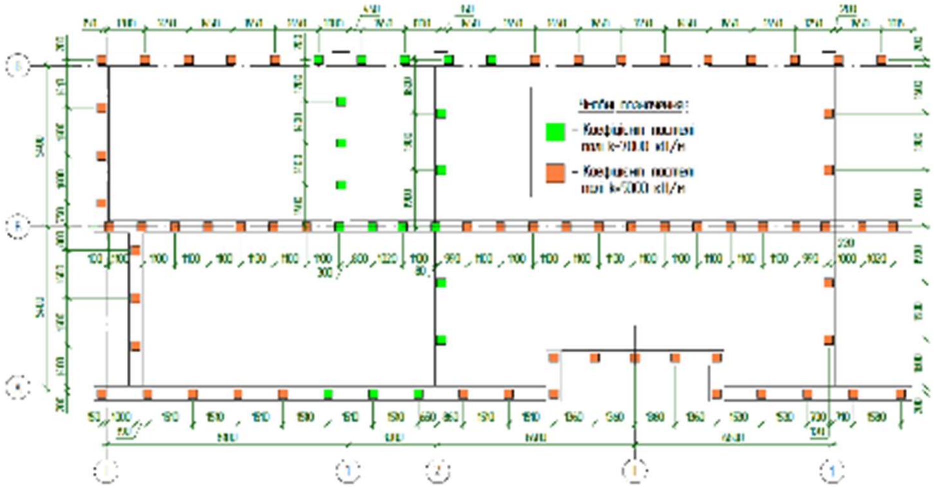

Figure 6 – Piles' rigidity, adopted in the simulation, after the base flooding

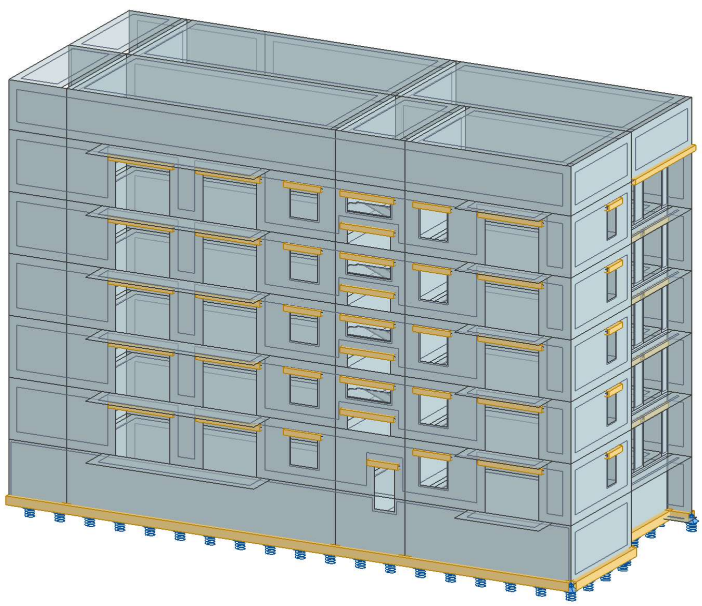

Figure 7 – 3D building's analytical model (pile foundation is set as elastic bearing)

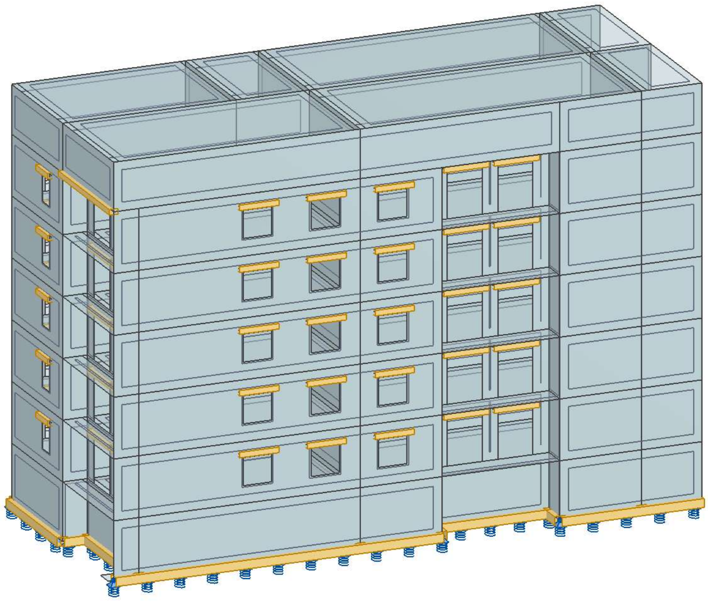

Figure 8 – 3D building's analytical model (pile foundation is set as elastic bearing)

The model of the building foundations before and after their strengthening is given in Fig. 9 and fig. 10.

The scheme of loads application for this building is shown in fig. 11. In this model for each floor, the characteristic values of the permanent load were set to be 4.9 kPa (floor weight, floor slabs, and parting walls), the characteristic values of variable sustained load – 1.5 kPa. The characteristic values of permanent load on the fifth-story flooring are set to be 4.84 kPa (floor weight, floor slabs), the characteristic values of variable sustained load – 0.7 kPa. The characteristic values of permanent load on the covering are 3.94 kPa (ruberoid sheet, roofing slabs, etc.), characteristic values of snow load – 1.28 kPa.

3D building's finite element model is shown in figure 12.

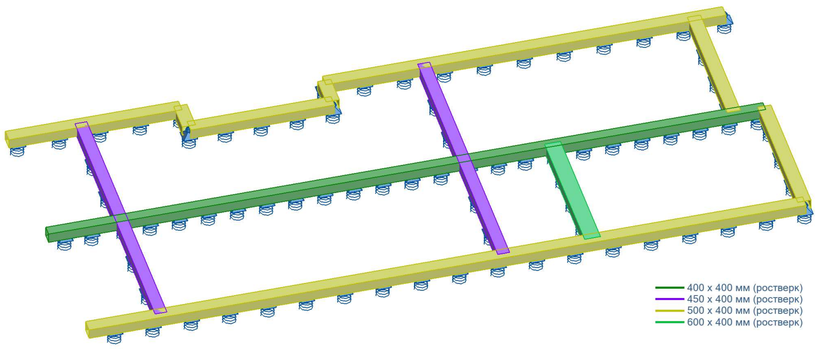

Figure 9 – Building's foundation model before the strengthening

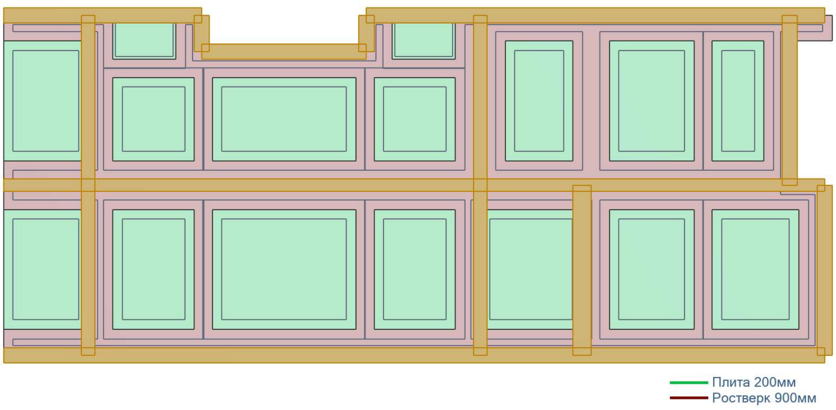

Figure 10 – Building's foundation model after its strengthening

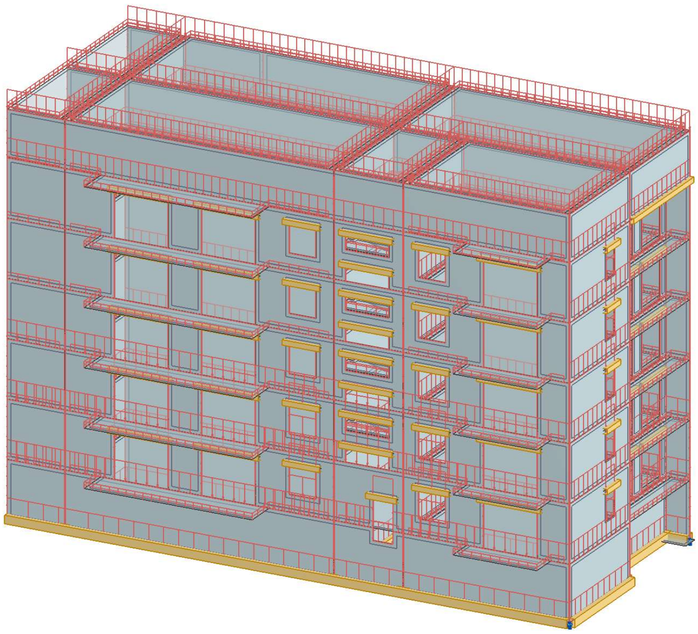

Figure 11 – 3D load application configuration for the building

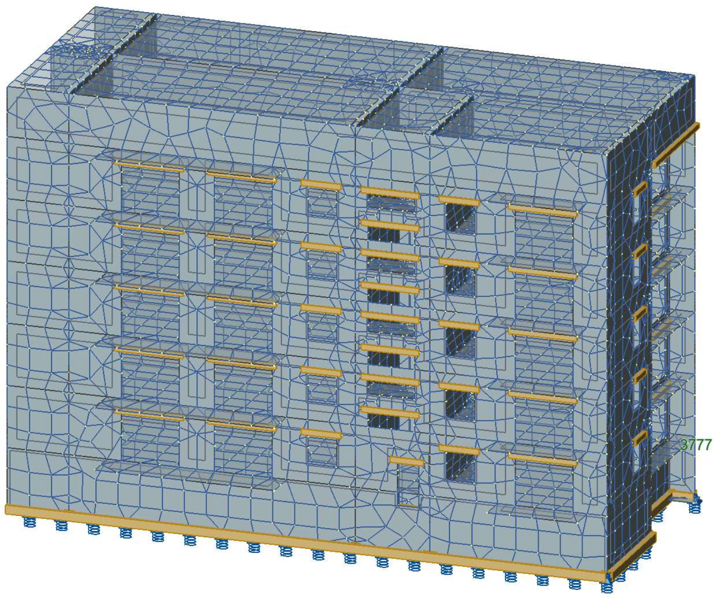

Figure 12 – 3D building's finite element model

Thus complex spatial geometrical schemes are simplified by replacement of a real design by the conventional scheme, for example, beams were approximated by bars, reduced to the axis, and slabs with walls were replaced with disks, reduced to the middle plane.

Calculation of structural configuration was carried out in spatial structural design with account for combined action of underground and superstructure, foundation and its base. In the building's SSS evaluation, the soil base of the existing foundation was replaced by elasticity factors, obtained in the spatial simulation of the building's structures. In structural configuration calculation, the actual reinforcement of reinforced concrete units and thus its non-linear operation was disregarded.

In the result of a structural configuration of the building calculation it's been defined the following: in the flat floor, covering, and foundation slabs – values of torsional and bending forces, shear and axial forces; in walls – values of normal and tangential axial forces, torsional and bending forces, and shear forces, etc.

In the building's numerical model, there were the following preconditions adopted: walls and floor slabs had hinge joints, as well as walls and grille joints. In the frame's numerical model, there were design parameters of strength, rigidity, and geometrical parameters of structures used. Floor slabs, stairwell, walls, and foundation reinforcement were simulated by disk elements, continuous grilles, and beams – by bar elements with respective axial and bending rigidity. The remaining structures, that had no impact on the spatial rigidity of the building (partying walls, floor, ceiling, roof, etc.), were represented by the equivalent load, applied in respective areas of the numerical model.

Using the evaluation results of the SSS of the "deformed building – driven prismatic piles as a part of a continuous grille – soil base with a weak underlying layer" system before and after supplying the monolithic reinforced concrete slab under the existing foundation grilles by FEM, let's analyze mainly the distribution of normal stress (to the horizontal plane) in-wall elements against the combination of existing loads (that include permanent and temporary with the limit design values).

The obtained stress values were compared to the design strain strength of brickwork in the traced section and compressive strength. Aside from that, it was considered that the brickwork was made of silicate brick M75 and grout M25.

According to table 9 (DBN V.2.6-162:2010, Appendix R) design strain strength of brickwork in the traced section is f_{bk2}=0.11 MPa. According to table 1 (DBN V.2.6-162:2010, Appendix R) design, the compressive strength of brickwork is f_d=1.10 MPa.

At the first stage of the simulation, the initial nature of the strains was determined in the brickwork at the time of the building`s construction before defects occurred for the faces 2-I and I-2 (respectively Fig. 13–14).

The compression strain does not exceed the maximum (f_d=1.10 MPa), and the tensile stress in the horizontal plane in some areas (for example, the places where the balconies are installed) exceeds the maximum values (f_{bk2}=0.11 MPa), which may be the result of a mismatch between the design model and between the real stress distribution scheme in the brickwork. But also in the walls around the stairwell, the tension of brickwork stressing is traced, that in some areas reaching the point of brickwork`s break by tensioning along a bandaged section, which may be the result of unsuccessful volumetric planning decisions of the building or errors in the design of the building foundation.

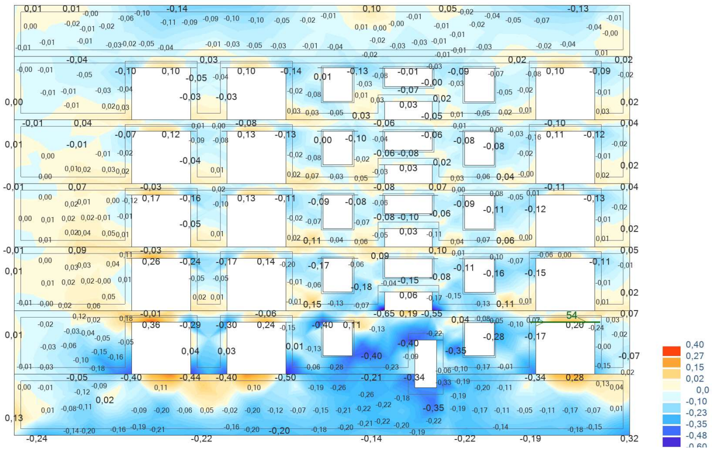

Figure 13 – Normal stress distribution view in the brickwork of outer walls on the 2-I face before the defects occurring

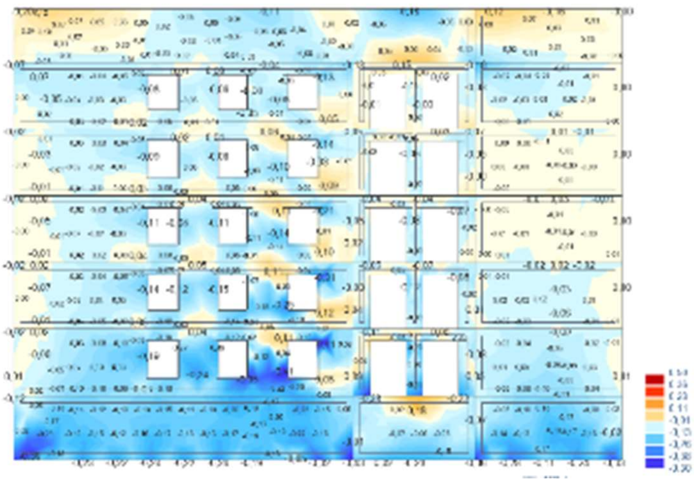

Figure 14 – Normal stress distribution view in the brickwork of outer walls on the I-2 face before the defects occurring

At the second stage of the calculation, the changes in the pile`s bearing capacity as a result of heating pipeline malfunction were considered, and the new behavior of the brickwork`s stress distribution behind the main faces was determined. By the nature of the stress distribution, it is possible to generalize that due to the flooding of the base, as a result of a pipeline accident, there was an uneven settlement of the left and right parts of the building around the stairwell (which because of the actual location of the pile as part of pile field has high rigidity). Therefore, most cracks are concentrated around the stairwell. On the stress distribution views (Fig. 15 and Fig. 16), the lines emphasized the places where the strength of masonry by tension is exceeded (f_{bk2}=0.11 MPa), and therefore the most possible occurrence of vertical cracks along a traced section. Values of compression strains do not exceed permissible values (f_d=1,1 MPa).

Figure 17 and Figure 18 show the layout of defects (cracks) of external bearing walls by the geotechnical monitoring data [4]. As can be seen, the locations and behavior of the theoretical (modeled by FEM) vertical cracks along the traced section are practically the same as the results of the surveys.

The third stage of the FEM modeling additionally takes into account the work of foundation reinforcement elements. The relevant stress distribution schemes (Figures 19 and 20) show that the foundation`s reinforcement has significantly helped to remove the uneven behavior of the stress distribution and bring it closer to the initial state. At the same time, the features of the tension stress distribution in the brickwork around the stairwell are saved, and their values either do not exceed the ultimate tensile strength of masonry (f_{bk2}=0.11 MPa) or exceed them in places similar to the initial state of the building.

The foundation reinforcement project was carried out [6]. A ribbed reinforcement plate is obtained, the base of which is the wet sand, fine, medium density. The ribs of the foundation slab are directed to the bottom. It efficiently redistributes stress from uneven deformations of the foundations and has significant rigidity with a minimum amount of groundworks, that is, in the process of strengthening a slab-pile foundation was arranged.

As geotechnical monitoring showed, new cracks in the building`s walls at the time of strengthening and the building's subsequent operation did not occur [4].

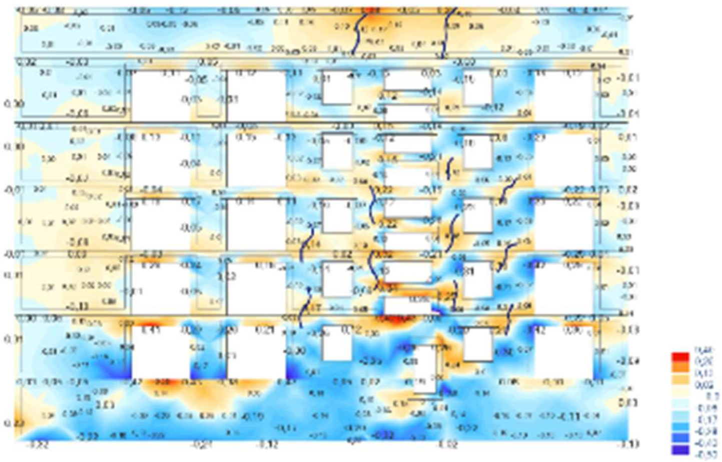

Figure 15 – Normal stress distribution view and respective fractures in the brickwork of outer walls on the 2-I face after the base flooding

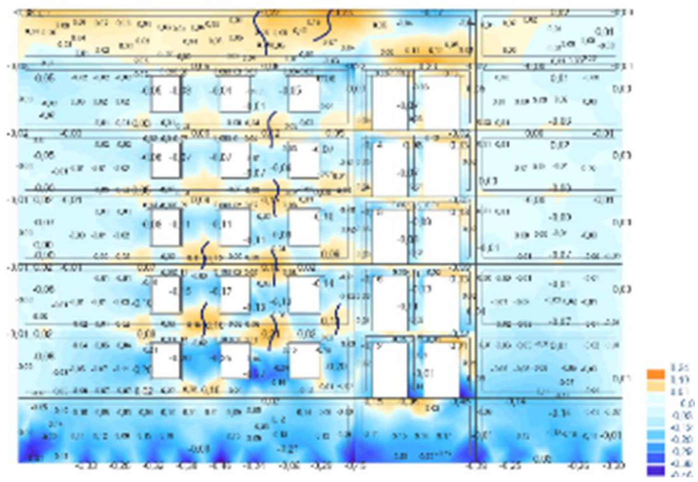

Figure 16 – Normal stress distribution view and respective fractures in the brickwork of outer walls on the I-2 face after the base flooding

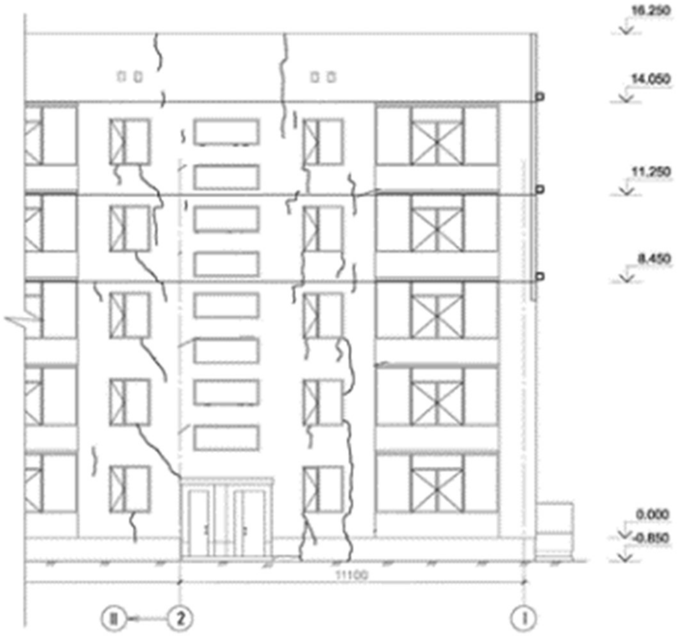

Figure 17 – Scheme of defects and damages in the brickwork of the outer walls on the 2-I face, along B axis (left end block section I-II) by the geotechnical monitoring data

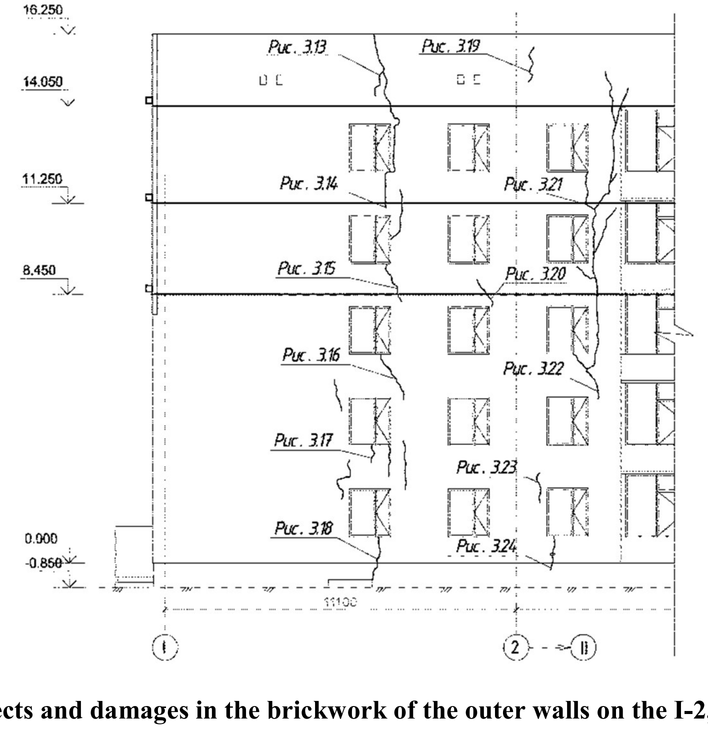

Figure 18 – Scheme of defects and damages in the brickwork of the outer walls on the I-2, face, along A axis (left end block section I-II) by the geotechnical monitoring data

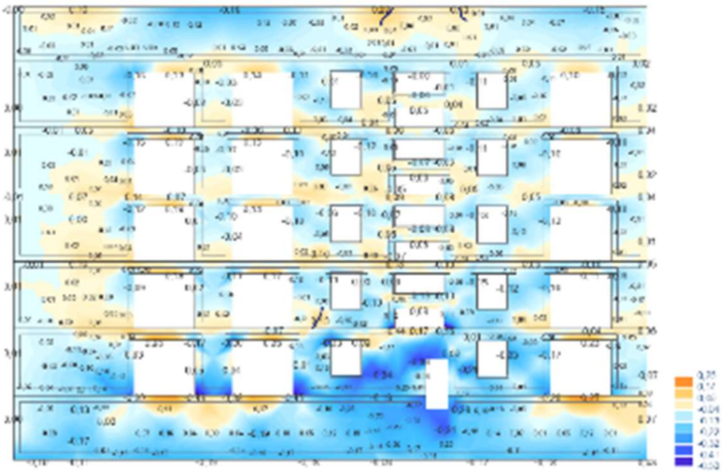

Figure 19 – Normal stress distribution view in the brickwork of the outer walls on the 2-I after the foundation strengthening

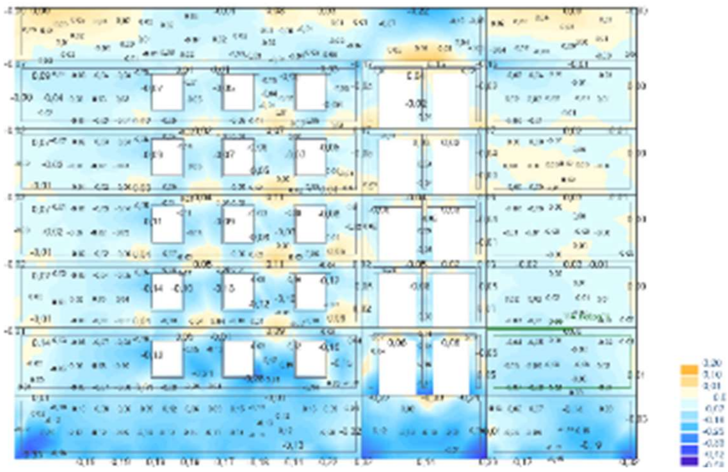

Figure 20 – Normal stress distribution view in the brickwork of the outer walls on the I-2 face after the foundation strengthening

Conclusions

Thus, a new 3D design scheme of the "deformed building – driven prismatic piles as part of continuous grille – a soil base with a weak underlying layer" system was developed before and after supplying monolithic reinforced concrete slab under the existing grilles, and mathematical modeling was carried out using the FEM of this system's SSS to estimate the features of combined action of its components.

New experimental data were obtained about the SSS change of the "deformed building – driven prismatic piles as part of a continuous grille – soil base with a weak underlying layer" system due to the supplying of a monolithic reinforced concrete slab under the existing grilles. The simulation of the SSS system after strengthening the foundation showed that the actual creation of the slab-pile foundation significantly mitigated the uneven behavior of the stress distribution and brought it closer to the original state. Sufficiently high efficiency and reliability of the method of the pile foundations strengthening as a part of the continuous grille by supplying the slab has been proved.

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Change of stress-deformed mode of the slope masses during developing and operation of excavations in it

Vynnykov Yuriy[1], Kharchenko Maksym[2], Yaholnyk Andrii[3], Lystopad Serhii[4*]

[1]National University «Yuri Kondratyuk Poltava polytechnic» https://orcid.org/0000-0003-2164-9936
[2]National University «Yuri Kondratyuk Poltava polytechnic» https://orcid.org/0000-0002-1621-2601
[3]National University «Yuri Kondratyuk Poltava polytechnic» https://orcid.org/0000-0003-4792-1934
[4]National University «Yuri Kondratyuk Poltava polytechnic» https://orcid.org/0000-0002-6743-1257
*Corresponding author E-mail: serzhlistopad@gmail.com

Peculiarities of structure geomorphological and engineering-geological features on the site with artificial excavation in the water basin form, the surface water valley runoff into the reservoir, and the soils properties are determined. Negative engineering-geological processes on the site and the reasons for the activation of landslide processes are considered. A slope spatial model is compiled. The slope stability was assessed taking into account the peculiarities of geological and hydrogeological structure using the structural soil strength. Sliding planes and shear pressures on possible landslide protection structures are determined. A "reverse" calculation of the slope stability was also performed to clarify the characteristics of soil strength. It is established that when arranging excavations in the slope array, its SDM changes, which activates landslides.

Keywords: slope, artificial excavation, landslide, comb, soil strength, cohesion, stress-deformed mode, finite element method, ultimate equilibrium method, one plane shear testing, «plate-by-plate» method

Зміна напружено-деформованого стану масиву схилу при влаштуванні та експлуатації у ньому виїмок

Винников Ю.Л.[1], Харченко М.О.[2], Ягольник А.М.[3], Листопад С.М.[4*]

[1, 2, 3,4] Національний університет «Полтавська політехніка імені Юрія Кондратюка»
*Адреса для листування E-mail: serzhlistopad@gmail.com

Проаналізовано геоморфологічні та інженерно-геологічні особливості будови ділянки розміщення штучної виїмки у вигляді ставка-накопичувача. Виявлено улоговину стоку поверхневих вод до водойми та визначено фізико-механічні властивості ґрунтів. Розглянуто негативні інженерно-геологічні процеси на ділянці й причини активізації зсувних процесів. Складено просторову інформаційну модель масиву схилу, до якої результати лабораторних досліджень ґрунтів внесено шляхом присвоєння її елементам відповідних властивостей. Виконано оцінювання стійкості схилу з урахуванням особливостей його інженерно-геологічної та гідрогеологічної будови і з використанням структурної міцності ґрунтів. Визначено можливі площини ковзання та зсувні тиски на потенційні протизсувні споруди. Проведено «зворотній» розрахунок стійкості схилу для уточнення значень характеристик міцності ґрунтів. Для оцінювання напружено-деформованого стану (НДС) масиву дослідного схилу використано математичне моделювання методом скінченних елементів з використанням пружно-пластичної моделі ґрунту за критерієм міцності Мора – Кулона. При цьому розрахунок методом скінченних елементів виконано шляхом ітераційного зменшення міцності ґрунтів до моменту настання граничної рівноваги. Встановлено, що при влаштуванні виїмок в масиві схилу відбувається зміна його НДС, що в свою чергу активізує зсувні процеси. Доведено, що для комплексного оцінювання впливу виїмки на НДС схилу доцільно використовувати технологію геотехнічного інформаційного моделювання. За результатами моделювання розроблено заходи щодо подальшої безпечної експлуатації схилу з виїмкою шляхом зменшення навантаження на верхню частину масиву та влаштування протизсувної споруди.

Ключові слова: схил, штучна виїмка, зсув, улоговина, міцність ґрунту, зчеплення, напружено-деформований стан, метод скінченних елементів, метод граничної рівноваги, одноплощинне зрушення, методика «плашка за плашкою»

Introduction

The task's urgency related to ensuring the slopes' soil masses stability is due to the need to operate building structures and MEP on landslide hazard areas.

Insufficient study of such areas engineering geological conditions features within the river valleys slopes leads to errors in the design, construction and operation of various building structures and MEP. Such consequences, in particular, are natural landscape disturbance characteristics, which occur during the installation of various artificial excavations, embankments, building pits, cutting slopes, etc. [1 - 4].

Using generally accepted slopes classifications, methods for determining the soils' mechanical characteristics and methods for assessing the soil masses stability and deformability does not make it possible to predict the landslides occurrence and determine the factors that cause them. The landslides occurrence is most often associated with the engineering-geological structure peculiarities and with changes in the slope masses stress-deformed mode (SDM). During the site development of such areas, and especially during the artificial excavations construction, there are additional influences, changes in soil properties, hydrogeological regime and relief [2, 4, 5].

The slopes soil masses SDM study, in particular significant anthropogenic impact, is a complex scientific and practical task. Therefore, to assess the slope stability, it is advisable to build spatial information models (BIM), as well as apply the finite element method (FEM) using elastic-plastic model and appropriate strength criteria, which allows to automate calculations, discretize the calculation area, take into account more factors, reduce time on cyclic operations taking into account changes in engineering-geological and hydrogeological conditions, etc. [6, 7].

This approach allows the development of effective landslide prevention works set at the stages of design, construction and operation of landslide hazard areas.

Review of the research sources and publications

Issues related to the landslides occurrence and dynamics peculiarities within the river valleys slopes are considered in the works of the Poltava geotechnical school, in particular, Yu. Velykodny and M. Zotsenko. The landslide processes formation conditions on the slopes in the presence of groundwater runoff basins are investigated. Using data from laboratory and field tests, landslides monitoring, of geological structure and slope stability assessment analysis, changes in the physical and soils mechanical properties within the basins were studied [2, 6, 7].

To improve the slope stability calculations, the normative standards [8, 9] provide for calculations taking into account the slopes typification. Existing typification is scale-related to geomorphological, geological, hydrogeological conditions, etc.

However, the typification takes into account only the slope destruction nature and the soil masses movement peculiarities, which incompletely allows assessing the landslides causes, which depend on the groundwater regime.

In turn, the groundwater regime is due to the waterproof layer nature, which roof they move in streams with different pressure gradients.

In addition, the existing landslides models do not take into account the spatial effects of soil layers and groundwater movement, which can be taken into account when using information modeling. Similar trends were noted in [12 - 15].

Therefore, based on the existing landslides typification and landslide processes dynamics F. Savarensky, A. Pavlov, I. Popov, Z. Ter-Martirosyan and others developed a classification of depressions and landslides on the slopes, which takes into account the soil mass geological structure.

The slope stability in many cases depends on the characteristics of soil strength and their physical condition. There are a number of methods for determining the mechanical parameters, which are included in the normative standards. Recommended processing test results methods often give unreasonably high strength characteristics values, because the experimental conditions do not correspond to the actual soil state in the slope masses [10, 11].

The landslides causes and development studies, soil strength reliable characteristics determination, the choice of correct calculation schemes for calculation, slope stability assessment and their modeling are exposed in the works of I. Boyko, L. Ginzburg, M. Goldstein, A. Gotman, N. Gotman, M. Demchyshyn, V. Kazarnovsky, M. Kornienko, V. Krayev, M. Maslov, S. Meschyan, D. Shapiro, V. Shvets, O. Schkola A. Bishop, A. Casagrande, J. Duncan and others.

Currently, it is proved the necessity to take into account the violation of soil structure, especially for loessial soil. Because of such violations, the soil strength characteristics are reduced, they should be determined by the «plate-by-plate» method. The characteristics obtained by this method are less compared to the standard method of one plane shear testing, as only the soils structural strength is taken into account, and the calculated slope stability coefficients showed that the use of such parameters allows more reliable slope stability assessment.

In some applications, the reducing procedure the strength properties values are implemented to find such critical values at which loses the slope stability, which helps to establish the actual experimental slope stability coefficient [12].

Definition of unsolved aspects of the problem

The normative standards [8, 9] regulate the slope stability calculation at the first limit state, taking into account both complete and structural soils adhesion. Appropriate techniques for determining complete and structural adhesion give inflated results. As a result, slopes with active displacements are defined as stable when assessing stability. Therefore, obtaining reliable initial data requires improvement of methods for their determination, adapted directly to the soil masses SDM features and slopes engineering-geological structure.

In addition, traditional methods for slope stability assessment often do not take into account the presence of depressions in waterproof layers. As a soil feature result in the geological structure is constantly under the more intense movement influence of groundwater flow. This negatively affects their mechanical properties and requires a specific approach to research methods. These factors and the forecast of their changes should be taken into account when assessing the soil mass SDM and the slopes stability.

Problem statement

Therefore, the purpose of the work is: to study the soil slope SDM with an artificial excavation (water basin); slope stability assessment of this array, taking into account the engineering-geological structure peculiarities, changes in the soils physical and mechanical properties in the depression; development of recommendations to protect the array from landslides.

To achieve it, the following tasks: engineering-geodetic and engineering-geological studies of the territory; laboratory studies of soil characteristics to assess the stability of the slope; slope information model construction, slope engineering-geological profiles and calculation schemes; possible lines (planes) determination of sliding and estimation of all slope and its separate sites stability; development of recommendations to stabilize landslide processes during operation on the structures slope.

Basic material and results

The site is situated in Azov-Pridniprovska geomorphological region of Kozyatyn structural-denudational watershed and is dated to the slope of the right tributary of the Kamyanka River. The region's river valley characteristic feature structure is formation in wide water-glacier descents and valleys with wide wetlanded bottoms.

By the analysis of toposurface, which was plotted before the embankment construction for the territory, it can be argued that the territory had general surface slope in the southwest direction–to the river (the right slope of valley of unnamed river, which is the right branch of Kamyanka river).

The slope surface was cut by the valley-erosion net. The most characteristic example of the network landforms is the depression (fig. 1), which was covered up in the construction process and constructed a water basin (fig. 2).

In the natural state (prior to the territory construction), the depression was the flow path of surface water into the river (the basin area from which surface water was collected is about 110,000 m2). At the top of the depression is an abandoned loam pit 2,5 - 6,5 m deep. On the bottom of the coombe, there are occasional deposits of dusty sand 1 – 5 cm.

The water ground level is about 9,7 – 13,1 m from the ground surface. Sands and sandy loams are water-bearing soils.

The geological structure of the site up to a depth of 30 m involves the thickness of Quaternary sediments: silty sands, small and medium size, as well as sandy loam and loam, which are covered with loose soil, soil-vegetative layer and humidified loam with total thickness 2,5–6,3 m (fig. 3, 4).

According to geological surveys, the depression geological structure presents soils that have genetic differences from the surrounding area soils (deluvial and alluvial soils). This feature is due to the processes associated with surface and groundwater runoff (flushing, erosion, filtration), which contributed to the formation of deluvial and alluvial deposits on the slopes and in the lower part of the basin.

Note, that the atmospheric water discharge to vertical planning occurred along this buried depression in the southwest direction. Because of such a long water movement impact in the depression, the soil has significantly lost its mechanical properties. From a hydrogeological point of view, the "buried" by embankment coomb continues to be a path of groundwater flow in the direction of the slope.

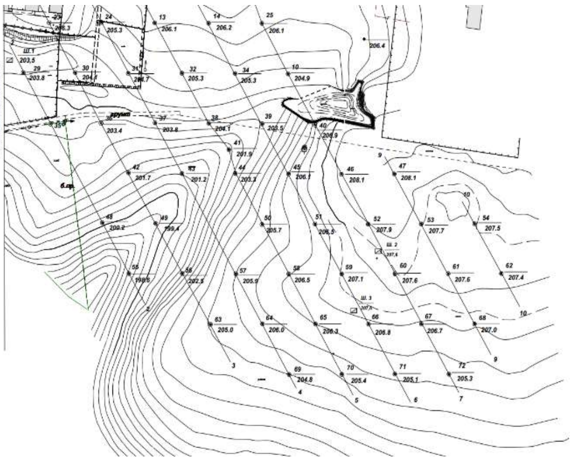

Figure 1 – Depression layout scheme (before construction in the territory)

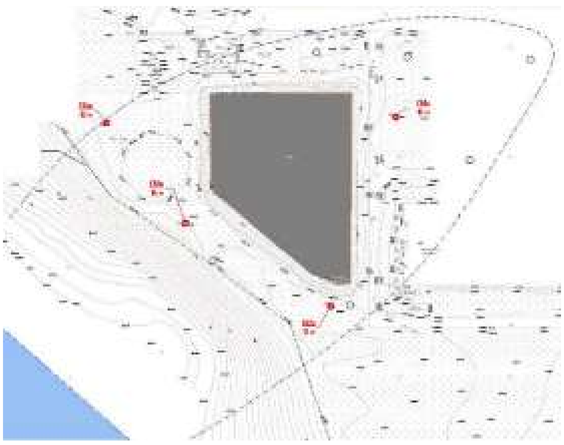

Figure 2 – Slope layout scheme with water basin

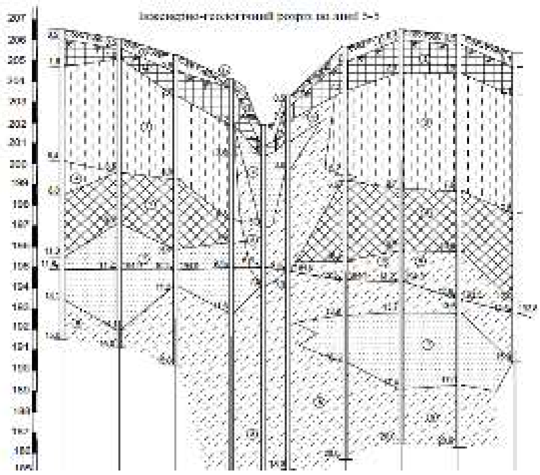

Figure 3 – Engineering-geological section before the slope construction

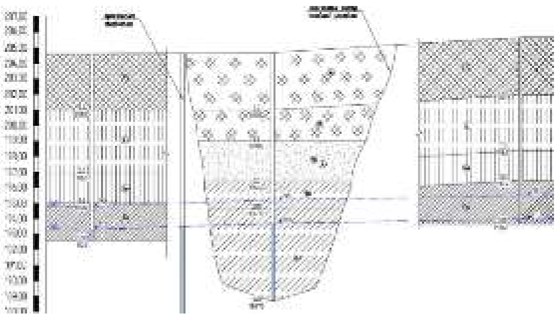

Figure 4 – Engineering-geological section after the slope construction

Unfavorable physical-geological processes within the site include:

– significant soil array heterogeneity both in area and depth: the presence of numerous layers, lenses, wedges, etc.; separate layers of soil encountered only by one or two wells; such array structure features indicate, among other things, the different soil sediments origin;

– presence soils with specific properties (collapsing, soft, backfills) particularly IGE-4 (loess sandy loam, silty, stiff, in saturated state – liquid, macroporous) and IGE-4 (loess loam, silty, macroporous, stiff, in saturated state – soft-plastic, macroporous) with collapsible properties due to loess strata saturation from surface mainly by atmospheric water infiltration;

– silty sand (IGE-5b) and silty sandy loams (IGE-6 and IGE-6a) with liquefied properties;

– site surface water erosion;

– presence of buried depression on the water basin site;

– presence of abandoned loam pit at the depression top;

– presence of landslide processes on the slope.

These negative processes led to the partial water basin slopes destruction, which were confirmed by the monitoring results. This deformation type can indicate the soil mass displacement and its pressure on the north side of the pond, resulting in a slight lifting of its sides - up to 30 mm.

On the southwest side of the basin, due to the soil mass displacement down the slope, the sides of its basin the slab lowering occurs (up to 250 mm), as well as the basin slope subsidence in the middle part (fig. 5). These deformations are the result of the landslide processes activation. The local transverse (latitudinal) fissures in the embankment massive from the riverside, water basin banks destruction should be highlighted among them.

Landslide processes activation is influenced by:

– depression territory usage, through which water flow to river was, for water basin construction; after water basin construction this water flow was partially blocked leading the slope soil massive additional saturation;

– territory releveling in the water basin construction process (excavation / mound) of soil mass contribute for atmospheric water accumulation in this manmade massif;

– active releveling of territory relief on the slope without layer by layer compaction and control of this process, turf cutting, activated landslides, erosion processes, mechanical suffusion, subsidence, etc;

– collapsible loess soils saturation (as a result of drainless areas, not enough organized surface drainage) contribute to soil mechanical properties decreasing;

– natural slope massif additional loading by soil embankment and constructions and water basin content.

Figure 5 – Soil slope subsidence of south-west side of water basin

Therefore, the visual surveys, topographic survey of the territory, as well as based on the archival data generalization in the existing water basin area can summarize the following.

1) Soil massif displacement processes occur at the investigated site.

2) The soil masses displacement affects the structures and the technical condition of the water basin and other surrounding buildings and structures.

3) As the territory made up of artificial embankment, and the water basin territory is in the place buried by the natural depression embankment, two variants of the landslides are possible:

- soil masses displacement occurs only within the bulk soils limits on the natural depression surface;
- soil displacement occurs on sliding surfaces within the natural soils of alluvial and delluvial genesis.

To assess the slope stability, engineering-geological profiles (sections) are constructed both in the longitudinal and transverse directions.

This importance is given to the origin (genesis) of each soil layer (IGE).

To evaluate soil properties, the characteristics of several groups are used: classification; basic; derivatives. For this purpose, soil samples of the undisturbed (natural) structure selected from the boreholes were used.

For strata above or at the level of the sliding surface, an additional test was performed using the method «plate-by-plate».

To analyze the groundwater level, a groundwater surface map in hydroisogips was drawn up (fig. 6).

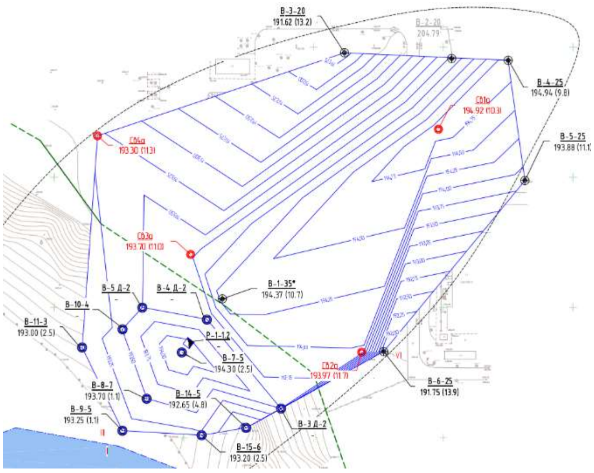

Figure 6 – Hidroisohypsis scheme on a site survey

As a result, the groundwater movement intensity increases, the hydraulic gradient increases, which leads to the soils weakening in the basin, their partial destruction

As a result of the natural place overlapping of ground water unloading by filling soil and the water basin construction, the groundwater level rose locally.

It is enhanced by leakage from the water basin and storm water drainage system.

The rise of groundwater around the water basin is confirmed by the hydroisogip scheme (fig. 6). It is marked by a "dome" and a change in the place of unloading groundwater on the slope.

Changing the hydrogeological regime and increasing the hydraulic gradient in the soils massif results in their considerable weakening, suppositional destruction and decrease of slope stability.

Generally, slope stability breach is associated with overcoming soil resistance forces by shear stresses acting on some planes.

Shear stresses in slope massif appear under the self-weight influence of soil mass and additional loading to slope from structures and filtration water pressure.

Soil shear resistance exists due to forces of internal friction and cohesion acted in soil massif. If the friction is present ($\varphi>0°$) forces of friction arise under influence of soil self-weight and additional load from structures.

After generating the initial data to assess the slope stability, calculations by combined slip surface, which position was chosen in most weak soil stratum along contact surfaces based on creation of maximal acting to retaining structures, were used and those calculations were checked by numerical modeling by FEM.

In such a case, the rate of slope stability is evaluated by value of safety factor (SF or k_{st}). In the case of $k_{st} > 1$, the slope is considered stable. In the case of $k_{st} < 1$, the slope stability loss is occurs. In the case of $k_{st} \approx 1$ the equilibrium limit state comes which leads to landslide.

Slope stability assessment consists in equilibrium condition consideration of soil massif 1 m width (two dimensional problem) with vertical lateral faces, conditionally cut from slope massif in the landslide direction (forces acted to lateral faces are not taken into account).

Its position is influenced by the peculiarities of the engineering-geological structure and hydrogeological regime on the site.

The sliding surfaces position was specified on the basis of a stability calculations series and the most probable sliding surface selection, where the stability coefficient value is minimal.

To clarify the soil strength characteristics (internal friction angle φst and structural specific cohesion c_{st}), a "reverse" calculation is performed.

It is performed with stability factor is equal to one ($k_{st} = 1$), because the slope is in a landslide condition (cracks, slopes, trees inclination towards the slope fall, etc.).

Inverse calculations for slope stability assessment are based on the analysis of their actual and forecast state, taking into account the action of all possible adverse factors and changes in engineering and geological conditions. This takes into account the change in hydrogeological conditions (surface and underground flown) by increasing the actual groundwater level, as well as changing the massive soil strength characteristics (using the smallest values obtained in the "reverse" calculation).

The aquifers influence that drain on the slopes' stability is taken into account in the conditions of rocks wetting, weighing, filtration pressure, removal by piping.

Water causes a weighing effect on the deposits that make up the slope. By saturating the soils, the water changes their physical and mechanical characteristics, and especially reduces the shear resistance value.

In addition, groundwater, wetting possible sliding surfaces, reduces friction. In doing so, the water, by weighing the soil skeleton, reduces the normal stresses in the displacement plane due to the pore pressure and can lead to almost complete removal of the internal friction in the soil.

According to the above methods and prerequisites, the calculation of the water basin dam slope stability were realised (table 2).

The slope calculation diagram is shown in fig. 7.

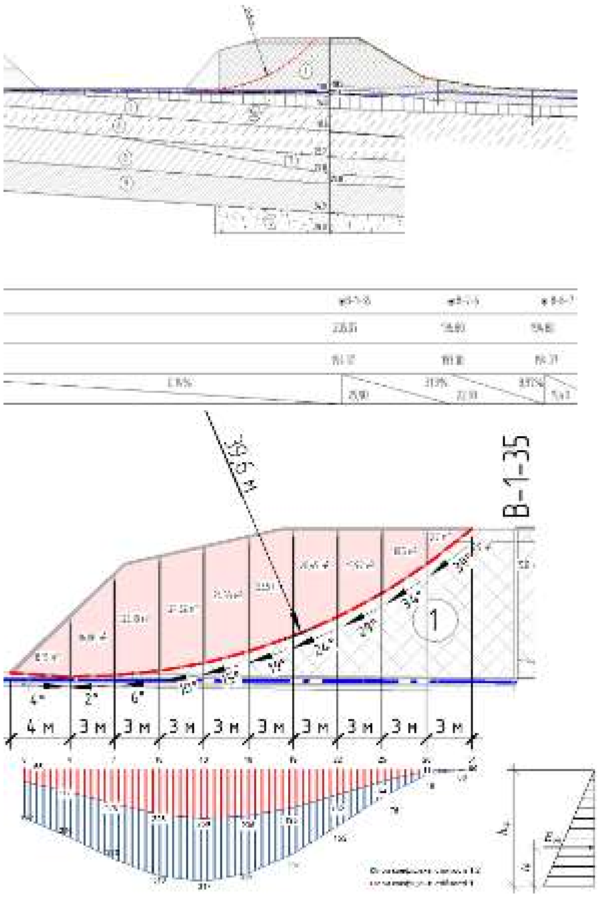

Figure 7 – Calculation schemes:
from above - placement of the sliding line
on the engineering-geological section
of the slope massif;
bottom - distribution diagram of equivalent
shear pressure (kN / m.p.) at stability
coefficient 1 and 1.2

FEM modeling was used to estimate the SDM of soils on the slope.

Slope stability modeling was performed using an elastic-plastic model with the Moore-Coulomb (MK) strength criterion. The following preconditions are accepted in the elastic-plastic problem formulation:

- the considered nonlinearity manifestations include plastic deformation of forming under a complex stress state, unimpeded deformation under tension;

- in a complex stress state (compression with shear), the total deformation contains a linear (elastic) and plastic parts, and the plastic deformation component occurs after the stress state reaches the tensile strength in accordance with the condition MK for the plane problem

$$\frac{1}{2}(\sigma_1-\sigma_2)+\frac{1}{2}(\sigma_1+\sigma_2)\sin\varphi-c\cdot\cos\varphi=0\,. \qquad (1)$$

The computational domain discretization while solving a nonlinear problem is performed by FEM.

The modeling was performed according to the method of soil strength reduction. It consists in reducing the strength characteristics until the onset of ultimate equilibrium.

This approach determines the slope stability coefficient as the specified characteristics ratio to their limit values

$$k_{st}=\frac{c+\sigma\cdot\tan\varphi}{c_r+\sigma\cdot\tan\varphi_r}\;, \qquad (2)$$

where c and φ are the initial strength characteristics;
r – normal voltage component;
c_r and φ_r are the limit values of strength characteristics

Accepted in the physical and mechanical properties calculations of engineering-geological elements and the structures properties are summarized in table. 1.

The geometric model with a finite elements grid for calculation and simulation results are shown in fig. 8.

It is used to determine possible sliding lines (planes) and to assess the entire slope stability and its individual sections, to develop measures to stabilize landslide processes during operation on the structures' slope, and so on.

In particular, according to the calculations results, the soil mass stability coefficient was 0.91. The artificial recess and the slope are in unsatisfactory technical condition. The actual soil mass state is approaching the limit, it is possible to intensify landslides.

Table 1 – Calculated Values of physical and mechanical properties of soils

Parameter	Unit	Number of soil strata				
		1	1a	1б	2	3
γ_I/γ_{II}	kN/m^3	16,8 17,0	18,4 18,6	16,5 16,7	16,7 16,9	17,5 17,6
The angle of internal friction φ_I/φ_{II}	deg.			11/ 13	24/25	17/18
Unit cohesion c_I/c_{II}	kPa			10,2/ 11,6	0,0/ 0,0	15,3/ 16,7
Unit cohesion structural	kPa			1,0	0,0	5,2

Table 2 – Shear forces calculation of the water basin dam soil massif

Block No.	Block length	Soil layer No.	Block Area, m^2	Block Volume, m^3	Unit weight of soil, kN/m^3	Block weight, kN	Slope angle of block	Internal friction angle φ_1, °	Unit effective cohesion c, kPa	Shear stress, kN	Shear resistance, kN		Shear stress (pressure), kN	Diagram of shear pressure by safety factor 1, kN	Diagram of shear pressure by safety factor 1.2, kN
											Fiction resistance, kN	Cohesion resistance, kN			
1	3	1	3,70	3,70	17	62,9	39,00	11,0	5	39,6	9,5	19	10,8	10,8	15,6
2	3	1	10,50	10,5	17	178,5	34,00	11,0	5	99,8	28,8	18	53,0	63,7	76,3
3	3	1	15,97	15,97	17	271,5	29,00	11,0	5	131,6	46,2	17	68,3	132,1	155,2
4	3	1	20,45	20,45	17	347,7	24,00	11,0	5	141,4	61,7	16	63,2	195,3	231,5
5	3	1	23,57	23,57	17	400,7	19,00	11,0	5	130,5	73,6	16	40,9	236,2	287,4
6	3	1	24,38	24,38	17	414,5	15,00	11,0	5	107,3	77,8	16	13,9	250,2	316,8
7	3	1	24,32	24,32	17	413,4	10,00	11,0	5	71,8	79,1	15	-22,6	227,6	310,0
8	3	1	23,18	23,18	17	394,1	6,00	11,0	5	41,2	76,2	15	-50,1	177,5	275,1
9	3	1	16,88	16,88	17	287,0	2,00	11,0	5	10,0	55,7	15	-60,7	116,8	226,2
10	2	1	8,15	8,15	17	138,6	-4,00	11,0	5	-9,7	26,9	15	-51,6	65,2	181,6
									Total	763,5	535,5	163			
										763,5	698,3				

Safety factor 0,91

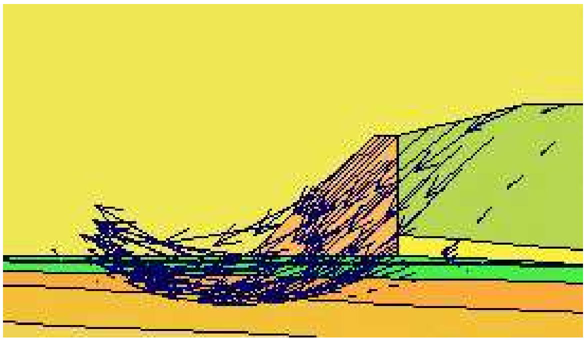

Figure 8 – Geometric model and results of SDM slope modeling

Modeling of the FEM using a comprehensive approach to determining the initial data confirms the calculations results and the actual slope soil massif state. Based on the results of assessing the slope stability with an artificial excavation for its accident-free operation, it is advisable reducing load on the slope (embankment) by cutting loose soil on the slope (minimum 2 m).

It is expedient to plan the territory for organized drainage of atmospheric water from the slope towards the river, as well as the landslide protection structures arrangement.

Conclusions

1. As a result of engineering-geodetic and engineering-geological survey of the territory there is a possibility of slope spatial information model geometry construction. The soils laboratory studies results are included in this model by assigning its elements the appropriate properties.
2. As a result of slope stability assessment analytically and by modeling the SDM according to the methods described above, the possible sliding planes and shear pressures on potential landslides were determined.
3. It is established that when arranging excavations in the slope array, its stress-strain state changes. This in turn affects the landslides activation. For a comprehensive assessment of the excavations' impact on the stress-strain state, it is advisable to use the technology of geotechnical information modeling.
4. Procedures have been developed for further slope operation with a recess by reducing the load on the upper slope part and the landslide protection structure installation.

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Creating urban spaces and medium-sized cities

Biloshytska Nataliia [1]*, Tatarchenko Halyna [2], Biloshytskyi Mykola [3], Tatarchenko Zakhar [4]

[1] Volodymyr Dahl East Ukrainian National University; https://orcid.org/0000-0002-8840-2885
[2] Volodymyr Dahl East Ukrainian National University; https://orcid.org/0000-0003-4685-0337
[3] Volodymyr Dahl East Ukrainian National University; https://orcid.org/0000-0002-9935-4161
[4] Volodymyr Dahl East Ukrainian National University; https://orcid.org/0000-0003-4632-1363
*Corresponding author E-mail: beloshitska@ukr.net

The research relevance is determined by the need to draw up a specific reference, regulatory and procedural documents that take into account regional features in designing public spaces. The analysis of urban spaces is carried out based on the example of the city of Severodonetsk to identify the spatial planning pattern of these elements and issues of their creation. It is found out that the main problem in creating public spaces is uncertainty in the regulations on urban development, namely, their number in the city, functionality, spatial planning pattern, design. There is revealed the unstructuredness of some elements of public spaces; lack of clear ranking and functional zoning in some cases; detachment of urban public spaces from the natural landscape.

Key words: public space, community space, square, city.

Формування міських просторів малих та середніх міст

Білошицька Н.І. [1]*, Татарченко Г.О. [2], Білошицький М.В. [3], Татарченко З.С. [4]

[1] Східноукраїнський національний університет імені Володимира Даля
[2] Східноукраїнський національний університет імені Володимира Даля
[3] Східноукраїнський національний університет імені Володимира Даля
[4] Східноукраїнський національний університет імені Володимира Даля
Адреса для листування E-mail: beloshitska@ukr.net

Організація міських просторів має важливий містобудівний статус, оскільки вони призначені для соціального, політичного, економічного спілкування городян. Актуальність роботи визначається необхідністю формування окремих довідкових, нормативно-методичних документів, які враховують регіональні особливості в проєктуванні публічних просторів малих та середніх міст. Існують тільки нормативні документи у галузі будівництва, які встановлюють розміри громадських центрів залежно від чисельності населення. роведено аналіз міських просторів на прикладі м. Сєвєродонецька з метою виявлення просторово-планувальної структури таких елементів і проблем у їх формуванні. Визначено, що основною проблемою у формуванні публічних просторів є невизначеність їх у нормативних документах з містобудування, а саме: кількості, функціональності, просторово-планувальної структури, формування. Виникає необхідність створення умов подальшої взаємодії містобудівних, еколого-ландшафтних, соціальних пріоритетів у розвитку середовища міських просторів як територій з підвищеною концентрацією активності городян, з урахуванням необхідності створення сприятливих умов проживання й індивідуалізації міського середовища та створення нормативних документів. Виявлено неструктурованість окремих елементів суспільних просторів; відсутність у ряді випадків чітко вираженого ранжирування і функціонального зонування території; відірваність міських публічних просторів від природного ландшафту. Складові елементи міських просторів розглянуто як окремо взяті містобудівні об'єкти (громадські центри, міські вулиці та площі, озеленення), що відірвані від ландшафтної підоснови і загальної екологічної ситуації. Громадські й публічні простори можуть виникати та успішно функціонувати тільки при більш уважному ставленні до них з боку міської адміністрації, соціально відповідального бізнесу і самих городян, тоді як ефективність роботи може бути досягнута лише завдяки їхнім спільним зусиллям. Зроблено прогноз точок появи нових публічних і громадських просторів.

Ключові слова: громадський простір, публічний простір, площа, місто

Introduction

A city is a complex dynamic and multifunctional organism, where urban spaces as centers of social, cultural, and social life are changing together with the environment. Therefore, the issues of creating urban spaces are becoming increasingly important. These elements of the urban environment have an urban-development significance and are made for social, political, economic communication of citizens. The peculiarity of these spaces is liveliness, crowds, high attendance, friendly social atmosphere, which is explained by their main functional content – to be a center of activity and exchange of information. However, in the theory and practice of urban development, there is no systematic approach to the design of these urban spaces from the standpoint of sustainable development of their environment, especially for small and medium-sized cities. The constituent elements of urban spaces are considered as separate urban-development facilities (public centers, city streets, squares, landscaping) detached from the landscape foundation and the general environmental state. The result is environmental and landscape degradation; in compliance with the created planning pattern of functional use with the requirements of creating favorable conditions for a person; the impossibility of constant use by different groups of the population at different times of the year and day. All this reduces the urban-development and social efficiency of territories and requires active environmental and landscape intervention as well as the development of methods of spatial planning, functional, socio-aesthetic change of environmental characteristics to create sustainable, self-regulatory natural and anthropogenic systems of urban spaces.

It should be noted that many authors provide different definitions of urban spaces in their research: some call them public, others – community. However, they have similar definitions.

Review of the research sources and publications

The development and transformation of the urban environment are studied in the works of Lynch K. [1], Posatskyi B.S. [2], Tyshchenko I.M. [3], Ladniuk M.I. [4], and others.

The author [1] defines that public space as a city cultural component expressed through the creation and recreation of urban culture, lifestyle, and the highest standards of cultural activity. Its main features are pedestrian accessibility and excess of possibilities.

According to [5, 6], a public space is a place that is open and accessible to all people (parks, squares, sidewalks, streets, etc.).

In the 1970s, "Placemaking" (development of public spaces) became popular. This term first formulated by J. Jacobs and W. White [7, 8, 9] refers to a multifaceted approach to planning, design, and management of public spaces. The idea of "Placemaking" is the opportunity to create new places that will fit harmoniously into the existing buildings based on knowledge, opportunities, and ideas of the local community.

Great importance is given to the short-term rest, which means recreation and comprehensive harmonious development of a person with clear territoriality. Short-term rest accounts for up to 90% of the recreational time during a five-day working week. However, it is mostly disorganized, semi-spontaneous, so it necessitates the search for management opportunities.

Public space is currently becoming the so-called "third place" – a place, where a person spends time between home and communication with family members ("first place") and the workspace ("second place"). This place is a "neutral territory", where different age and social groups of the city population spend their free time.

Public space is a place for exercising political and public human rights [4, 10], and is characterized by the following functions:

– political – creating space for political discourse;
– social – interaction of people between themselves and the government;
– recreational – places for leisure;
– educational – a tool for preserving historical and cultural monuments.

The typology of urban spaces is determined by the urban function [11]:

– linear – the city's street network that solves the issues of communication, transport;
– local – open spaces, where accumulation or redistribution of transport or human masses, the concentration of trade, communication take place;
– dispersed – closed spaces that form "work fields" of dispersed processes of work and life (recreation, living environment, production). This type of spatial pattern of the urban environment is the alternation of residential yards and areas of public buildings in the residential area.

In their turn, linear urban spaces are divided into:

– frontal – the field of view is "filled" with a planar image of a facility;
– three-dimensional – a facility is perceived in the perspective;
– deep – a facility goes into the view depths forming a "spatial body".

The architectural features of public spaces are accessibility; convenience, safety; multifunctionality (ability to transform according to the needs of different social groups). These are city squares, streets, sidewalks, parks, cafes, etc. In this case, the form is secondary, because it is a process taking place there that is an important criterion.

The meanings of community space and public space in some cases coincide, but there is a significant difference between them: community space is associated with a common and accessible place for public activity, and public – with something happening in front of everyone, but deeply and personally significant at the same time [12].

Urban geosystems of small and medium-sized cities have a much lower dynamism compared to the large ones making urban planning more relevant along with their most important role in the social division of work and the whole life of society. It is important to understand that community and public spaces can appear and function successfully only with a more attentive attitude to them on the part of the city administration, socially responsible business, and the citizens themselves, while efficiency can be achieved only through their joint efforts.

Definition of unsolved aspects of the problem

The research relevance is determined by the need to draw up the currently absent reference, regulatory and procedural documents that take into account regional features in designing public spaces of small and medium-sized cities. There are only regulations in the field of construction, which determine the size of community centers depending on the city population [13, 14]. There are many features in creating spaces in such cities: climatic, natural, historical, etc. For example, the change in the city status (Severodonetsk has the status of the oblast center since 2014), the consequences of hostilities, unsatisfactory environmental state as a result of chemical industry activities, and unsatisfactory level of landscaping [15] require a special approach to urban spaces from the standpoint of sustainable development.

As a result of the above, there is a need to create conditions for further interaction of urban-development, environmental, landscape, social priorities in the development of the urban environment as spaces with a high concentration of citizens taking into account the need to create favorable living conditions and individualization of the urban environment, and to draw up the regulations.

Problem statement

To analyze urban spaces of small and medium-sized cities based on the example of Severodonetsk, to identify the spatial planning pattern of these elements and issues of their creation.

Basic material and results

In modern architectural and urban-development practice, there is a problem of lack of developed, citizen-oriented comfortable, and accessible public spaces. According to world standards, public places should be primarily safe, free, comfortable for all groups of the population. In modern Ukrainian cities, most public spaces do not meet these criteria. This is especially true for small and medium-sized cities. Severodonetsk is no exception.

The generally accepted typology of urban public spaces [11] can be used to categorize the public spaces of Severodonetsk. However, it is subjected to clarification and adjustment due to the specific urban situation. Thus, according to the spatial planning conditions of Severodonetsk, except for configuration and size, the typology also includes the connection of the space (local, linear, dispersed) with elements of the planning pattern "framework" of the city, i.e. highways of different types.

According to the analysis, it has been found out that 45% of the public spaces of Severodonetsk are the local ones (Fig. 1). In their turn, local spaces are divided into the following subtypes:

- squares with buildings at the corners – 43% of the total number of spaces of this subtype (Victory Square, the square near the City Council – Peace Square).

- squares with a decorative fence (the square near the Ukrainian Music and Drama Theater, the square near the city's Palace of Culture of Builders).

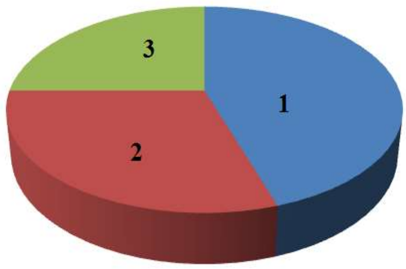

Figure 1 – Typology of Severodonetsk public places:

1 – local, 45%; 2 – linear, 30%; 3 – dispersed, 25%

All these squares are multi-purpose with a predominance of administrative and business, cultural and entertainment establishments, trade functions. These are areas of the city or district significance.

The second type of space is linear - 30%. Among them the following subtypes are distinguished:

– designed between two high-traffic highways – residential streets formed by a chain of segments and sites of local significance - 33% (Tsentralnyi Prospect);

– spaces intersected by one or several highways of the city or district significance – 57% (Hvardiiskyi Prospect, Khimikiv Prospect);

– spaces intersected by neighboring local spaces - 10% (Khimikiv Prospect – Peoples' Friendship Boulevard).

Such spaces are created by the sequential connection of facilities of visual perception. Spaces of this type are multifunctional and most often belong to areas of district significance depending on the rank of their elements.

The third type - dispersed spaces – comprise 25% of the total number of public spaces in the city and are made up of a set of local and linear elements. The constituent parts of these spaces are often separated by buildings, and their visual connections are difficult (e.g. the ensemble of public spaces of the Holy Christmas Cathedral is compositionally connected with the main axis of Hvardiiskyi Prospect). These systems belong to areas of the city significance.

It is necessary to distinguish public spaces based on their interaction with the natural landscape (spaces adjacent to the natural landscape make up 15% of the total number of urban public spaces):

– local spaces adjacent to the landscape - 10% of the total number of spaces of this subtype (Lake Chyste, Lake Parkove, city park);

– linear spaces adjacent to the landscape - 66% (Tsentralnyi Prospect – "Jazz" shopping and entertainment complex).

According to their functional purpose, all public spaces of the city of Severodonetsk can be divided into political, social, and recreational functions.

According to the political function, it is possible to name such spaces as the square near the City Council – Peace Square (Fig. 2, a). It was the main square for various political events. In 2014, Victory Square (Fig. 2, b) acquired the same status due to the location near the Civil and Military Administration of Luhansk oblast. In addition, during election campaigns, this function is acquired by the places of concentration of the city's residents (the intersections of Hvardiiskyi Prospect and Kurchatova Street, Hvardiiskyi Prospect and Donetska Street, Khimikiv and Tsentralnyi Prospects).

At the same time, these squares perform both social and recreational functions, because there are held not only political, social but also cultural events. First of all, this applies to Victory Square. The square near the Ukrainian Music and Drama Theater performs a social function.

Public spaces that perform a recreational function may also include Lake Chyste (Fig. 2, c), the city park with Lake Parkove (Fig. 2, d), and the territory of "Jazz" shopping and entertainment complex.

a b c d

Figure 2 – Severodonetsk Public Spaces:
a – Peace Square; b – Victory Square; c – Lake Chyste; d –Lake Parkove and the city park

A distinctive feature of creating public spaces of small and medium-sized cities is their small number, compactness, and multifunctionality.

The analysis of the typology of public spaces in Severodonetsk has been conducted to identify the spatial planning patterns of these elements of urban spaces. It has shown numerous problems with these spaces, the main of which is the following:

– unstructured elements of public spaces;

– the absence of a clear ranking and functional zoning in some cases;

– detachment of urban public spaces from the natural landscape (only 15% of public spaces in Severodonetsk somehow interact with the natural environment, and the established connection of public spaces exists only in the central (second) and partly in the first planning areas of the city).

The problem of creating public spaces in cities, particularly small and medium-sized, is that in the current DBN B.2.2-12:2019 "Planning and development of territories", a "public center" is an independent object of urban development, and public or community space is its component. In the same DBN B.2.2-12:2019, the consolidated indicators of the area of a multipurpose city center are provided: for medium-sized cities – 5-10 m^2/person; for small – 10-20 m^2/person. When making new master plans of cities, there is a need to clearly define the concept of a "public space" and its size.

The historical formation of the city of Severodonetsk began with the first planning area in the 30-50s of the last century, the second was formed in the 70-80s. The third planning area began to be built at the end of the 20th - beginning of the 21st century. It is found out that most urban spaces are located in the first planning area

- points 1-4 in Fig. 3. This is mainly the squares of the Prospect, where mass events take place, e.g. flash mobs, rallies, concerts, fairs. In the second area, there are few such places. First of all, it is a leisure area of Lake Chyste (Fig. 3, p. 5). In the third planning area, they were not planned at all.

The city development is presupposed by its territory, and it goes in the southeast direction. Therefore, the concentration of the population and the city center is gradually shifting. In this regard, places of communication, recreation, agitation sites are beginning to appear spontaneously. They are gradually designed and created. For example, the park, children's and sports grounds began to be created around the built Cathedral (point 6). Shopping malls and a network of cafes have also been built here.

If we draw the main axis of the urban space of the city built in the first planning area (Fig. 3, points 1-4), we can see that it has now shifted to the second one (Fig. 3, p. 5-6). Currently, the process of creating public and community places continues. A skate park is being built in the second planning area (Fig. 3, point 7). On the line of axis 2 – point 8, the residents have created the best yard of the city. There is a campus near Volodymyr Dahl East Ukrainian National University. That is, the further development of the city has led to the creation of new public spaces.

If we draw another line of creation of urban spaces of the city through points 4, 5, and 6, and also continue them in the form of a fan on the map of Severodonetsk, it is possible to forecast where the needed public spaces may arise. On the example of the city of Severodonetsk, these are places corresponding to points 9-11 (Fig. 3). It should be noted that the "Sever Lake" project is already being prepared by Severodonetsk Youth Council together with the Center for Modern Change, which provides the development of the shoreline near Lake Chyste for cultural and active recreation of residents and guests (Fig. 3, point 9). It is clear that the next spontaneous point of growth of public or community space will be the place specified by points 10 and 11. Therefore, in urban development, there is a need to plan and organize such spaces, to allot the necessary areas for them.

Currently, the city needs new public spaces in the third planning area to reduce the radius of accessibility and the time to get to them.

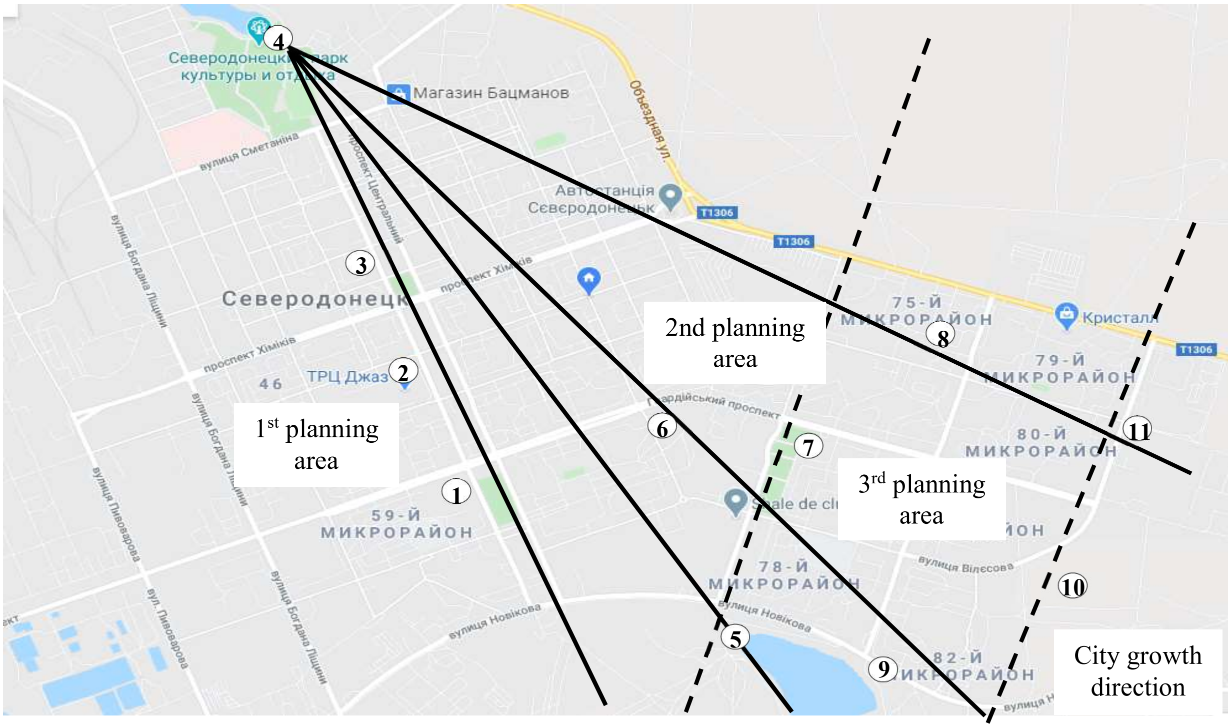

Figure 3 - Public spaces in the city planning pattern of Severodonetsk
1 – Victory Square; 2 – Palace of Culture of Builders, 3 – Peace Square; 4 – City Park;
5 – Lake Chyste, 6 – "Tank" Memorial, 7 – the church and the adjacent park,
8 – the best yard, 9 – the planned design of a recreation area,
10, 11 – promising areas of urban space

Conclusions

According to the analysis of urban spaces based on the example of Severodonetsk, it is established that the main problem in creating public spaces is their uncertainty in the regulations on urban development, namely, their number for the city, functionality, spatial planning pattern.

There have been revealed unstructured individual elements of public spaces; the absence of a clear ranking and functional zoning in some cases; detachment of urban public spaces from the natural landscape.

The forecast of new community and public spaces appearance is made.

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The conditions and factors of negative consequences of reconstruction -thermal modernization of buildings

Sokolenko Valerii[1*], Holodnov Oleksandr[2], Sokolenko Konstyantin[3], Filatiev Myhailo[4]

[1] Volodymyr Dahl East Ukrainian National University https://orcid.org/0000-0002-5073-2694
[2] Volodymyr Dahl East Ukrainian National University https://orcid.org/0000-0002-9722-9164
[3] Volodymyr Dahl East Ukrainian National University https://orcid.org/0000-0003-3334-7855
[4] Volodymyr Dahl East Ukrainian National University https://orcid.org/0000-0001-5608-6737
*Corresponding author E-mail: 13wms13@ukr.net

The negative consequences analysis of public buildings thermal modernization for cases of work stoppage has been carried out. The negative consequences arise for the environment and the facility residents. Organizational and construction mistakes have a long complex character and cannot be corrected. The problem from the technical side turns into an administrative process. The task is to propose organizational and technological measures that would make such cases impossible in the future. It is proposed to carry out work by an integrated method, to complete facade work within a certain section of the wall. Advance financing should be tightly linked to the project sequence of work

Keywords: protection of the environment, reconstruction of buildings and structures, thermal modernization of buildings, external wall insulation

Умови та фактори негативних наслідків реконструкції - теплової модернізації міських будівель

Соколенко В.М.[1*], Голоднов О.І.[2], Соколенко К.В.[3], Філатьєв М.В.[4]

[1, 2, 3, 4] Східноукраїнський національний університет ім. В. Даля
*Адреса для листування E- mail: 13wms13@ukr.net

Теплова модернізація, реконструкція будівель і споруд має на меті продовження терміну експлуатації, поліпшення функціональних та споживчих характеристик об'єкта. Статистика свідчить про непоодинокі випадки, коли значні кошти, закладені в реконструкцію будівлі, суттєво погіршують умови функціонування об'єкта та роботу конструкцій. Негативні наслідки, що викликані зазвичай організаційно-будівельними прорахунками, мають довготривалий комплексний характер, погіршують навколишнє середовище, стан конструкцій, мікроклімат у приміщеннях. У містах Сєвєродонецьк, Рубіжне, Старобільськ є об'єкти, на яких роботи з утеплення фасадів призупинено. Внаслідок цього захисні конструкції фасаду не змонтовано, мінераловатні плити та вітрова мембрана руйнуються, стіни будинків накопичують вологу, що потрапляє у напівзруйнований шар мінераловатних плит. Природно-кліматичні чинники призводять до руйнації теплоізоляції, внаслідок чого у навколишнє середовище потрапляє велика кількість мікроскопічного пилу, шкідливого для людини. Чималі кошти фактично спрямовано на погіршення умов експлуатації замість позитивного ефекту. На нашу думку, ситуація не може бути виправлена простим завершенням невиконаних робіт, оскільки порушено технологію виробництва. Проблема з будівельної переростає в адміністративно-цивільну, оскільки мова йде про тривалий негативний вплив на навколишнє середовище та загрозу здоров'ю мешканців об'єктів реконструкції. Постає завдання щодо розроблення організаційно-технологічних заходів, які унеможливили б подібні випадки у майбутньому. Разом з підсиленням технічного нагляду рекомендується впроваджувати здійснювати роботи комплексним методом, коли протягом звітного періоду повністю виконується комплекс опорядження фасаду в межах виокремленої чарунки – ділянки стіни. Авансове фінансування слід жорстко ув'язувати з проєктною послідовністю виконання робіт, початок робіт не дозволяти без затвердженого проєкту виконання робіт, що пройшов експертизу.

Ключові слова: охорона навколишнього середовища, реконструкція будівель, теплова модернізація будівель, утеплення фасадів

Introduction

Modern houses are designed taking into account the requirements for minimizing heat loss and ensuring effective thermal insulation. A large number of buildings - civil and public, require reconstruction and modernization, the thermal modernization remains a very urgent problem. Neither the state nor city budgets are able to provide funding for full-scale reconstruction programs for existing housing stock, utilities, etc. The solution to the problem is seen through the implementation of various programs, modernization projects, with the involvement of all possible sources of funding, investors, stakeholders. In conditions of a limited resources amount, the task, therefore, consists of the most efficient use of funds, the implementation of modern, relevant projects of reconstruction, modernization of buildings and structures.

Review of research sources and publications

A large number of researchers and scientists were involved in problems of reconstruction of buildings and structures. The issues of energy saving in construction were dealt with by Yu. Tabunshchikov, G. Farenyuk, M. Brodach, M. Timofeev, K. Fokin [1-5], and many others. The problems of thermal modernization of buildings of SRSBC (Kiev) are thoroughly investigated. Ukraine as an industrial, urbanized country has extensive experience in reconstruction, modernization, restoration of industrial facilities, urban development, buildings and structures, accumulated by specialized design, scientific institutions, universities. In general, the experience is systematized by the state building codes [6-8].

Definition of unsolved aspects of the problem

The purpose of the study is to determine the circumstances and factors leading to a negative result in the reconstruction of urban buildings.

According to this goal, the following tasks are solved: the systematization of buildings thermal modernization negative cases; analysis and grouping of negative consequences and threats arising from organizational and construction factors; to develop conditions and constraints to help prevent similar construction miscalculations and consequences in the future.

Problem statement

The purpose is to carry out the analysis of the negative consequences of thermal modernization of public buildings for cases of the work stoppage. The negative consequences arise for the environment and the residents of the facility. Organizational and construction mistakes have a long complex character and cannot be corrected. The task is to propose organizational and technological measures that would make such cases impossible in the future.

Basic material and results

Most of the industrial cities of Ukraine combine similar classifications and objects inherent in the stages of their development. The urban environment unites various types of buildings and structures with various typologies and building classifications. However, there are laws for the development of society and basic principles that establish certain framework conditions for the functioning of cities. There is an inconsistency between the estimated and standard periods of the normal operation of structures, elements of buildings and structures, materials for interior and exterior decoration, engineering systems, and communications of buildings. General requirements for the level of housing comfort and places of people residence are changing, worldview guidelines are changing. Royal residences do not meet the requirements of modernity [9]. This state of affairs is the standard, the problem is partly the fact that real estate has features of inertia and high cost, re-profiling or modernization will require significant material and financial costs [10]. In modern conditions, an increase in energy consumption is an unacceptable trend, the issue of thermal modernization and reconstruction of buildings - residential and public [11] is acute. Numerous projects and programs for saving energy and heat resources have been successfully implemented. The houses of the Soviet era were built according to outdated heat consumption standards and need modernization [12]. For typical projects and homogeneous buildings, almost standardized solutions are used, providing for facade insulation, roof renewal, attic floor insulation, replacement of windows and doors with modern energy-saving ones, and entrance lobby renewal. Insulation of facades mainly involves the device of a ventilated structure using mineral wool boards. Styrofoam plates, etc. liquefied insulation is used less often since it does not meet fire safety requirements. Unfortunately, the statistics of cases when work on facade insulation was suspended, facade structures were not installed have already been accumulated. The obvious and probable reason for the termination of work is insufficient funding. To illustrate the situation, you can give examples in Severodonetsk, Rubezhnoe, Starobelsk Fig. 1 – 3.

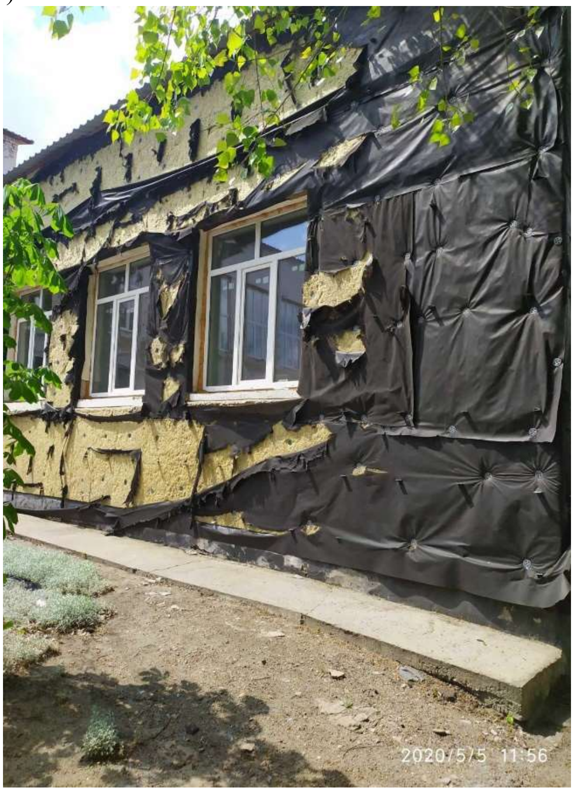
Figure 1 – An example of successful facade thermal modernization

a)

b)

Figure 2 – Building in Starobelsk
a – the state of unfinished construction;
b – a fragment of the destroyed layer
of mineral wool mats

a)



b)

Figure 3 - Building in Rubizhne
a – a fragment of the wall;
b – a fragment of the facade with areas
of complete destruction of mineral wool mats.

According to the totality of building typological features, these are houses with bearing walls made of bricks, floors made of precast concrete elements, medium-rise and multi-storey. The houses have a constructive margin of safety, the modernization possibility without interfering with the constructive system, and, under the conditions of correct design decisions and work, the continuation of normal operation is guaranteed for the design period. Due to the work termination, the temperature and humidity conditions of the outer walls change dramatically and deteriorate. The changes have negative consequences for both the environment and the state of structures and the physical parameters of the interior. There is a multifactorial threat to human health.

The mineral wool mats that are not protected from the side of the facade, are saturated with moisture - atmospheric or capillary, which contributes to the soaking of the wall material. For silicate bricks, humidification is unacceptable, since it impairs the physical and mechanical characteristics. Silicate brick has a water absorption of about 13% and average frost resistance of 35 cycles. In a humid state, the heat-shielding properties of the wall significantly deteriorate. In winter, the dew point moves to the inner edge of the wall. Regular moisture saturation promotes the accumulation of salts, mold and mildew multiply. Degradation of structures occurs. Instead of a positive effect, an artificial structure appears, almost the only function of which is to moisten the wall. The issue of the influence of moisture on the heat-shielding properties of the material of the enclosing structures (external walls) has been studied quite qualitatively. General recommendations are that moisture in external wall structures is harmful, its volume and presence should be minimized, the operating mode of structures should provide for the possibility of drying and removing moisture. A classic example is a horizontal waterproofing of two layers of roll material on the top of the plinth to stop the penetration of capillary moisture.

The presence of moisture contributes to the accumulation of mineral salts, the adhesion of the brick to the mortar of the masonry joints deteriorates, and the masonry joints thaw out. Internal moisture in the indoor air condenses on the walls, flows down, saturates the layers of plaster, wallpaper, lingers in the cracks of the baseboard, floor. The processes become especially intensive in winter. A situational solution is to provide intensive air exchange, drying and air conditioning of premises, surface heating of walls, application of UFO. As a vivid analogy, an example can be taken - a winter fur coat must be wetted before dressing.

The second group of negative influence factors concerns the environment. A layer of mineral wool mats that do not have mechanical protection is destroyed as a result of atmospheric and climatic actions. Water is a unique compound, it has three-phase transformations, it is consistently present in the form of liquid, vapor, ice. The sun's rays and winds increase the effect of moisture. The fiber structure of mineral mats breaks down and a large number of small particles are formed. A large amount of mineral dust gets into the environment, which poses a threat to the health of the residents of the quarter. The process is continuous - that is, the negative impact becomes systemic. The residents of the site are even more vulnerable. Typically, the ventilation system is used for supply and exhaust with the natural draft. Air enters the room through cracks and leaks due to infiltration. Mineral dust penetrates the air.

During the performance of insulation work, builders use personal protective equipment - respirators, overalls, goggles, rubberized gloves. Failure to comply with safety and safety regulations can cause respiratory tract damage, eye irritation and conjunctivitis. With prolonged exposure, the development of oncological diseases of the human lungs is possible. The most dangerous are mineral wool fibers up to 3 microns thick and up to 5 microns long. The impact on the environment from mineral wool mats that passively collapse on the walls has not been fully studied, however, one can only discuss the degree of harmfulness of the impact - low, medium, moderate. The situation is complicated by the fact that it is practically impossible to dismantle mineral wool slabs without damaging the environment. The current legislation contains a number of requirements limiting and prohibiting environmental pollution. According to the provisions of DBN "10.1, the general contractor for the construction organization must obtain permission to perform construction and installation works from the local authorities at the construction site. To do this, she submits a copy of the positive conclusion of the state environmental examination of the documentation on which the facility will be built (if it refers to the List of activities and facilities posing an increased environmental hazard approved by the Cabinet of Ministers of Ukraine), as well as a plan for implementing measures to ensure environmental protection in the process of building the facility and carrying out commissioning works in accordance with the requirements of the environmental legislation of Ukraine and the provisions of the specified conclusion of the state environmental examination. 10.2 Construction and installation works for the construction of any facilities must be carried out in compliance with the requirements of environmental legislation and ensure effective protection of the environment (land, subsoil, water bodies, atmospheric air, flora and fauna) from pollution and damage. Measures to ensure this should be provided for in the design estimates and organizational and technological documentation. 10.7 Construction and installation work in residential areas must be carried out in compliance with the requirements for the prevention of dust formation and air pollution. When collecting waste and garbage, it is not allowed to dump them from buildings and structures without the use of closed trays and storage bins" [13].

For violation of legislation, state building codes, liability is provided.

The Civil Code of Ukraine defines the guarantees of the work quality (Article 859). The Criminal Code provides for liability in case of - Art. 275 Violation of the rules regarding the safe use of industrial products or the safe operation of buildings and structures - Violations during the development, design, manufacture or storage of industrial products of the rules regarding their safe use, as well as violation of the rules regarding the safe operation of buildings and structures during the design or construction, by a person obliged to comply with the following rules, if this has created a threat of death of people or the occurrence of other grave consequences or caused harm to the health of the victim. Liability is also provided for in the event of a violation of the normal functioning of the housing stock - Criminal Code of Art. 270-1. Intentional destruction or damage to objects of housing and communal services. 1. Intentional destruction or damage to housing and communal services facilities, if this has led or could have led to the impossibility of operation, disruption of the normal functioning of such facilities, resulting in danger to life or health of people, or property damage on a large scale.

It is impossible to ignore such, frequent cases. T. Maslow's pyramid defines the basic needs of humankind, and the second is general security. You can observe the nature of the transformation of this concept in human civilization. [9,14]. Primitive man chose a cave for protection, in which he kindled a fire. A house with a fence was supposed to protect from predators and attacks. The history of the development of cities is a fascinating story of the improvement of fortifications – walls, towers, fortresses. The formation of nation-states shifted the defense against external attack to the borderline and contributed to the emergence of police structures - as law enforcement agencies. The competition of political systems has formed block associations that create an international system of protection, deterrence and resistance. Humanity no longer needs protection from predatory animals. The very nature of threats is changing. Natural or man-made disasters cause casualties comparable to those of the wars of the ancient era. Epidemiological problems are not the subject of our article. However, humanity has recognized the threats of the nano and micro levels. The harmful effects of asbestos were not immediately recognized. However, the link between the onset of cancer and the use of asbestos has now been proven. Secondhand smoke is recognized as harmful to the average person. Smoking in public places is limited and administratively punishable.

An unfinished bridge is worse than an unfinished bridge - building wisdom. Some unfinished facilities constitute a suspended threat with a deteriorating effect. For the cases under consideration, it is obvious that the houses have been damaged, the conditions for their functioning and operation have deteriorated, and sources of harmful emissions into the environment have been created. It is noteworthy that the implementation of the reconstruction project has a considerable cost and the influence of time only worsens the situation, that is, the problem will not be solved by itself. For the unsatisfactory ecology of the region, there are no such cases of a systemic threat. However, the question remains as to how long the residents will remain tolerant to the problem.

The negative consequences are due to organizational miscalculations. Relatively speaking, half of a fully completed facade for the same money would not create problems. Construction could be completed upon renewal of funding.

Taking into account the seasonal nature of construction, the specifics of financing and the terms of the contract, it is advisable to introduce the principle according to which a separate wall (or section of a wall) must be completed in one calendar month of the construction cycle.

Using the dependence of the definition of the construction time for rhythmic flows

$$T = k\,(n + m - 1)\,. \tag{1}$$

It is possible to determine the number of divisions.

Typological features of buildings with bearing walls made of bricks determine the approximate width of the building 15-21 m. and a regular structure in height. So the minimum size of the division can be equal to the size (area) of the end wall.

Conclusions

The systematization of unfinished objects, the completed analysis indicates that the negative consequences of the reconstruction occur in the event of design errors, violation of the work execution technology and as a result of organizational miscalculations. In fact, the correct design solution is not implemented, it causes a deterioration in the general condition of the building structures, operating conditions, and internal parameters of the indoor microclimate. The negative impact of unfinished construction only increases over time.

Negative consequences occur for the environment – environmental pollution, and the structures of the building and the residents of the facility. The processes of degradation of external walls are accelerated, the microclimate of the premises is deteriorating.

Organizational and construction mistakes are of a long-term complex nature and cannot be corrected. The technical problem turns into an administrative and legal one. The task is to propose organizational and technological measures that would make such cases impossible in the future. It is proposed to carry out the work by an integrated method, within a certain section of the wall to complete the facade work completely. Advance financing should be closely aligned with the project workflow. In the work contract, prescribe the condition for the complex installation of facade systems within a certain cell (part of the facade).

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Modeling of production process by the method of works maximum approximation

Dyachenko Evgen[1]*, Zyma Oleksandr[2], Pahomov Roman[3]

[1] Poltava National Technical Yuri Kondratyuk University https://orcid.org/0000-0002-8551-0805
[2] Poltava National Technical Yuri Kondratyuk University https://orcid.org/0000-0001-7484-7755
[3] Poltava National Technical Yuri Kondratyuk University https://orcid.org/0000-0001-9169-8296
*Corresponding author E-mail: eugeny1210@gmail.com

The work is devoted to mathematical modeling of production processes. Existing methods of production processes modeling are analyzed. Calculated workflow schemes are proposed when planning the manufacturing process by maximizing work approximation. The dependences of the time parameters calculation and the conditions of the proposed calculation schemes application are given. The method of the time parameters calculation of production processes execution by the method of works maximum approximation is offered. The procedure of calculation and construction of production processes calendar schedule is given. The advantages of the proposed methodology in comparison with existing modeling methods are analyzed

Keywords: mathematical modeling of production, method of works maximum approximation, calculation schemes, calculation of time parameters of production processes execution, calendar schedule

Моделювання виробничого процесу методом максимального зближення робіт

Дяченко Є.В.[1]*, Зима О.Є.[2], Пахомов Р.І.[3]

[1, 2, 3] Полтавський національний технічний університет імені Юрія Кондратюка
*Адреса для листування E-mail: eugeny1210@gmail.com

Метою статті є розроблення розрахункових схем взаємозв'язків між роботами, які дозволять виконувати планування виробничого процесу методом максимального зближення робіт, та отримання залежностей для розрахунку параметрів часу виконання робіт; розроблення методики розрахунку параметрів часу виконання робіт методом їх максимального зближення, а також побудова календарного плану із зазначенням критичного шляху. Виявлено, що різноманіття взаємозв'язків між роботами зводиться до трьох видів: послідовне, паралельне й послідовно-паралельне виконання робіт. Перетворення цих відомих взаємозв'язків на розрахункові схеми дозволяє створити принципово нову математичну модель планування виконання робіт. Запропоновані схеми дають змогу відмовитися від жорстких просторових захваток при організації послідовно-паралельного виконання робіт. Метод максимального зближення робіт являє собою нову аналітичну модель планування виробництва з наочним відображенням організаційних і технологічних взаємозв'язків робіт. Розроблений метод надає можливість простіше переходити від аналітичного моделювання до графічних календарних графіків. Метод максимального зближення робіт легко піддається автоматизації обчислень за допомогою електронно-обчислювальної техніки, а також автоматизації графічної побудови. Це дозволяє більш ефективно виконувати оптимізацію планування виробництва (як за тривалістю, приводячи її до директивної, так і за ресурсами), своєчасно враховувати зміни виробничих обставин. Такий метод дає змогу брати до уваги організаційні й технологічні обмеження, при цьому виключаючи простої бригад, що виконують роботи. Запропоновані розрахункові схеми та залежності для визначення параметрів часу дозволяють охопити все різноманіття взаємозв'язків робіт (послідовне, паралельне, послідовно-паралельне виконання робіт)

Ключові слова: математичне моделювання виробництва, метод максимального зближення робіт, розрахункові схеми, розрахунок параметрів часу виконання виробничих процесів, календарний графік

Introduction

Modern industrial production, which combines a large number of contractors with complex and diverse relationships between them when executing work on a joint project, is impossible without flexible and timely planning. Such planning is possible only on the basis of application of the works organization calculated methods. In addition, the management of modern production is characterized by a multitude of solutions, the choice of the best due to the variety and complexity of technologies is not an easy task. The solution to this problem is possible through the use of economic and mathematical methods for modeling the production process and computer. Thus, developing a management model that would most adequately reflect the main features of the production process and be amenable to automated calculation, would be easy to use and enable the visualization of results, is an urgent task.

Review of the research sources and publications

Today there are many methods of planning the production process, each of which is characterized by its positive and negative qualities, quite widely covered in the literature [1 – 13]. Models that have become widely used in the planning of production processes in construction include Gantt charts, various network models, matrix models. In most cases, these mathematical models are ultimately interpreted into graphical calendar models that are close in structure to linear graphs. Along with the positive qualities that make it possible to use these methods, they have some disadvantages. Thus, the disadvantages of Gantt charts [4, 6] include the lack of clearly illustrated relationships between work, the inflexibility and rigidity of the chart structure, the complexity of its adjusting, the complexity of variant processing, the inability to use computing to automate calculations. The disadvantages of network modeling include the lack of the production processes display clarity, the complexity of the structure, especially in the serial-parallel execution of works, high complexity of calculations. The main disadvantage of matrix modeling is the need to separate objects into rigid spatial captures. Therefore, the existence of serious shortcomings in existing planning methods make research in this area relevant.

Definition of unsolved aspects of the problem

Despite the large amount of work devoted to this problem, a method of modeling the production process, which adequately reproduced the main features of the production process and made it possible to automate the calculations, was easy to use and made it possible to clearly visualize the results of modeling, is not proposed.

Problem statement

The purpose of the article is to develop computational diagrams of relationships between works that will allow planning of the production process by the method of works maximum approximation, and to obtain dependencies for the calculation of the work execution time parameters. Development of a method for calculating the work execution time parameters by the method of their maximum approximation, as well as the construction of a calendar plan indicating the critical path.

Main material

Based on the analysis of the relationships of works that are characterized the construction industry [4, 6], their diversity can be reduced to three ways of linking works in time: this is a serial, parallel and serial-parallel execution of work. Technological and organizational peculiarities of process execution make demands for time lag of one work from another on the minimum acceptable technological or organizational break. Such interconnections of work, taking into account organizational and technological breaks, make it possible to bring their execution as close as possible to serial-parallel execution without splitting the object into captures. As a result, the paper offers computational diagrams that allow you to simulate these three ways of linking work in time and to calculate the work execution time parameters using the maximum approximation method.

The first design scheme 1. Serial execution of works (Fig. 1). The serial execution of works is planned in the case when the full completion of the previous work is required to start the next work, and a time gap is possible between them.

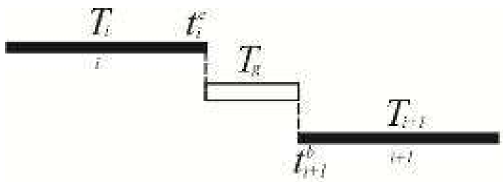

Figure 1 – Design scheme of works serial execution

The strength of concrete is obtained after the work of concreting the foundation for the equipment. Then the equipment installation work is carried out. The completion of a previous job is associated with the beginning of the next job in the serial work execution of. The calculation of time parameters when using such a scheme is performed according to the following dependencies:

$$t_{i+1}^{eb} = t_i^{ee} + T_g \; ; \tag{1}$$

$$t_{i+1}^{ee} = t_{i+1}^{eb} + T_{i+1} \; ; \tag{2}$$

$$t_i^{le} = t_{i+1}^{lb} - T_g \; ; \tag{3}$$

$$t_i^{lb} = t_i^{le} - T_i \, , \tag{4}$$

where t_i^{lb} – late beginning of work i;

t_i^{ee} – early ending of work i;

t_i^{le} – late ending of work i;

t_{i+1}^{eb} – early beginning of work i+1;

t_{i+1}^{lb} – late beginning of work i+1;

t^{ee}_{i+1} – early ending of work i+1;

T_i – duration of work i;

T_{i+1} – duration of work i+1;

T_g – the minimum allowable gap time between works.

The second design scheme 2a, 2b. Parallel works execution (Fig. 2-3). Work is carried out independently of each other in parallel execution. However, it may be necessary to link the beginnings or endings of work.

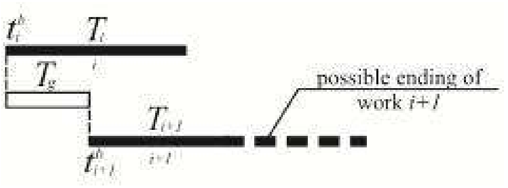

Figure 2 – Design scheme 2a of parallel works execution

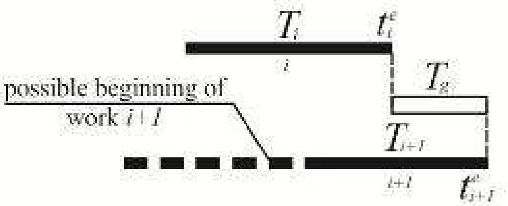

Figure 3 – Design scheme 2b of parallel works execution

For example, electricians should start work on laying electricity before plasterers; the break between works should provide the front of the plastering work throughout the site. In the future, electricians and plasterers do their jobs independently, ending one job affects the ending of another. Painting and sanitary works can be carried out in parallel and independently. In terms of the interrelationship of these works, their beginnings are not interconnected, but the finishing of the painting works should be later than the sanitary works with such a gap in time between their endings, which allows the painters to complete the final painting of the sanitary fixtures and communications. The time parameters when using Scheme 2a, that is, when work is related to beginnings, are defined as follows:

$$t^{eb}_{i+1} = t^{eb}_i + T_g \,; \tag{5}$$

$$t^{lb}_i = t^{lb}_{i+1} - T_g \,, \tag{6}$$

where t^{eb}_i – early beginning of work i.

In the case of combinations of works with finishes, the time parameters are defined as follows:

$$t^{\mathrm{ee}}_{\mathrm{i+1}} = t^{ee}_i + T_{\mathrm{p}} \,; \tag{7}$$

$$t^{\mathrm{le}}_{\mathrm{i}} = t^{le}_{i+1} - T_{\mathrm{g}} \,, \tag{8}$$

where t^{le}_{i+1} – late ending of work i+1.

The third design scheme 3a, 3b. Serial-parallel works execution (Figs. 4, 5). Serial-parallel works execution implies a constant lag of the next job from the previous one for a certain time, which cannot be less than the minimum allowable gap between the works. The minimum acceptable interval is taken depending on the requirements of safety or work technology. For example, between the soil excavator and the completion of a soil manually, such a gap in time is required, which guarantees a safe distance between the excavator and the workers. Between the plastering and painting works take the technologically necessary gap in time, which ensures the plaster will dry up before the painting works begin. In order to ensure a minimum allowable time gap between the works during their serial-parallel execution, it is necessary to observe this gap throughout the work execution (both at the beginning and at the end).

Depending on the ratio of the previous and subsequent works duration, it is possible to establish the need to introduce a gap between the beginnings (scheme 3a) or the endings of works (scheme 3b).

If the duration of the next work is greater than or equal to the previous one, then the delay of the next work beginning from the previous one to the minimum permissible gap guarantees at least a delay during the whole time of their joint execution (scheme 3a, Fig. 4).

If the duration of the next job is less than or equal to the duration of the previous one, then to guarantee the delay of the next job from the previous one during the whole time of their execution by the value of the minimum gap, a sufficient delay of their endings by the minimum acceptable gap (scheme 3b, Fig. 5).

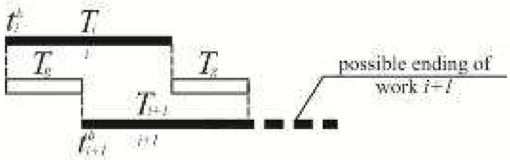

Figure 4 – Design scheme 3a of serial-parallel works execution

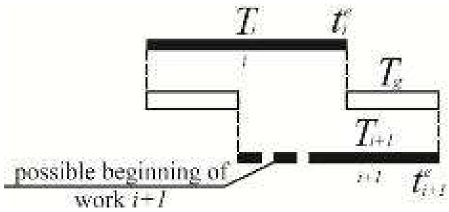

Figure 5 – Design scheme 3b of serial-parallel works execution

When using serial-parallel works execution schemes, the calculation of time parameters is performed by dependencies (5, 6, for scheme 3a) and (7, 8, scheme 3b).

In order to start calculating time parameters, it is necessary to have an organizational and technological scheme of work. It is proposed to draw up such a scheme in the form of a table (Table 1) using the following design schemes. The works are separated by the minimum possible permissible gaps in the development of organizational and technological scheme. The works are entered in the table in the order of their execution. The work code is its serial number. At the same time, the work duration graph is filled. Then each work is considered in relation to previous works, as a result of the analysis the codes of the previous works and the numbers of the design schemes are filled. If the work under consideration is related to several previous works, then the design scheme of relationship and the gap between each of them are indicated. The value of the minimum permissible gap between the works is taken from the conditions of the lag during the technological or organizational break or by the requirements of the backlog in space. The justification for the need to enter the work gap is indicated in the last column of the table.

Calculation of time parameters is proposed to be performed in tabular form (Table 2) according to the given design dependencies. The spreadsheet consists of two parts: the left one, which describes the organizational and technological interrelationships of the works, and the right one, in which the parameters of the works execution time are calculated. First, the left part of the table is filled, the data for which is received in accordance with the organizational and technological scheme (table 1).

The calculation starts with the determination of the early parameters. The calculation is performed for each job, moving from work to work from top to bottom. The calculations use dependencies that correspond to the design diagrams of the relationship between the works. If the work in question is related to several jobs, then the early parameters maximum values calculated for that work are taken into account for the determination of this work execution time parameters.

Table 1 – Organizational and technological scheme of works execution

Code of work	The name of the work	Code of previous works	Accepted link between works: Serial, E-B, Number of the design scheme 1	Parallel, B-B, 2a	Parallel, E-E, 2b	Serial-parallel, B-B, 3a	Serial-parallel, E-E, 3b	Minimum gap in days, T_d	Minimum gap in days, T_i	Justification of the gaps between the works
1	Excavation processes	-	-	-	-	-	-	-	9	-
2	Concreting of foundations	1	-	-	-	+	-	1	21	Safety norms
3	Return backfill	1	+	-	-	-	-	1	12	Mechanism transition
		2	-	-	+	-	-	3		Formwork removal
4	Installation of columns	2	-	-	-	-	+	16	15	Concrete strength set
		3	-	+	-	-	-	2		Safety norms

Table 2 – Calculation of works execution time parameters by the method of works maximum approximation

Organisational relationships: Processes code	Processes name	Previous processes code	Calculation scheme number	Minimal gap, T_g	Duration of the process, T_i	Calculation of time parameters: Early parameters, Start, t_i^{eb}	Early parameters, End, t_i^{ee}	Late parameters, Start, t_i^{lb}	Late parameters, End, t_i^{le}	Time reserves, R
1	Excavation processes	-	-	-	9	0	0+9=9 9	20-9=11 1-1=0 0	21-1=20 0+9=9 9	0
2	Concreting of foundations	1	3a	1	21	0+1=1 1	1+21=22 22	22-21=1 30-21=9 1	38-16=22 33-3=30 22	0
3	Return backfill	1	1	1	12	9+1=10 25-12=13 13	10+12=22 22+3=25 25	23-2=21 21	21+12=33 33	8
		2	2b	3						
4	Installation of columns	2	3b	16	15	38-15=23 13+2=15 23	22+16=38 15+15=30 38	38-15=23 23	38	0
		3	2a	2						

After calculating the early parameters, the late parameters are determined. The calculation is carried out for each work, moving from the bottom up. In the final work, the early and late parameters are the same. The calculation is based on the dependencies that correspond to the calculation schemes. As there are several references to the work in the following works, minimal values are taken from the calculated parameters for each reference of the works execution time.

After calculating the early and late parameters of the works execution time, time reserves are determined. They are calculated as the difference between late and early parameters.

According to the results of the execution time parameters calculating, the mathematical model of production should be represented in graphical form. Graphic representation of the mathematical model allows to present it in a clearer form, to attach the works execution to the calendar basis, which in turn makes it possible to plan the provision of resources production and to control the timely execution of planned works. The most visual and convenient to use are linear calendar graphics. The basis for drawing the calendar schedule is the form1, which is given in [13].

The calculation of the production organization planning by the method of works maximum approximation (table 2) makes it easy to translate it from analytical to graphical form.

Construction of a linear graph is performed according to early time parameters. Scale bar graphs indicate work planed on early parameters, time gaps between works, possible late endings of works, time reserves, links between work, and the critical path of the schedule. An example of constructing a linear graph is shown in Fig. 6.

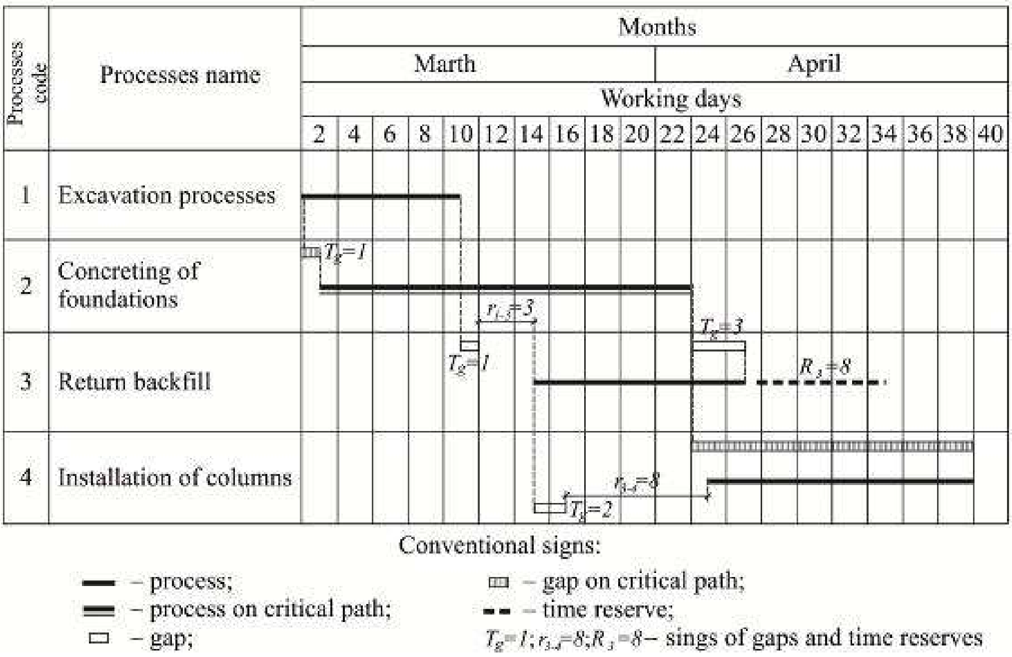

Figure 6 – Linear calendar chart based on the calculation results

Conclusions

It is known that all the diversity of relationships between jobs comes in three types: serial, parallel, and serial-parallel works execution. The transformation of these known relationships into computational schemes allows us to create a fundamentally new mathematical model of work scheduling. The proposed schemes allow to avoid rigid spatial engagements when organizing serial-parallel works execution. Replacing the work lags in space by the lags in time makes it possible to naturally take into account the technological and organizational gaps between works in the time parameters calculations, which in turn allows us to maximize closeness of the works execution between works.

The method of works maximum approximation is a new analytical model of production planning with a visual representation of the organizational and technological interconnections of the work. It allows you to develop the positive qualities of previous models and to some extent eliminate their negative properties. The developed method makes it easy to move from analytical modeling to graphical calendar graphs. The method of works maximum approximation is easily subjected to automation of calculations by means of electronic computing, as well as automation of graphical construction. This makes it possible to optimize production planning more efficiently (both in terms of duration, leading to policy and resources), and to take into account changes in production circumstances in a timely manner. The developed method allows to take into account organizational and technological constraints, while excluding simple brigades performing the works. The proposed calculation schemes and time-dependencies allow us to capture the full range of work relationships (serial, parallel, serial-parallel works execution).

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Challenges in applying expert systems to the investment-construction projects designing

Uvarov Pavlo[1], Tatarchenko Galina[2]*, Shparber Marina[3], Biloshytska Natalia[4]

[1] Volodymyr Dahl East Ukrainian National University https://orcid.org/0000-0002-5660-6859
[2] Volodymyr Dahl East Ukrainian National University https://orcid.org/0000-0003-4685-0337
[3] Volodymyr Dahl East Ukrainian National University https://orcid.org/0000-0003-1296-7510
[4] Volodymyr Dahl East Ukrainian National University https://orcid.org/0000-0002-8840-2885
*Corresponding author E-mail: tatarchenkogalina@gmail.com

He design systematology possibilities in investment and construction activity tasks maintenance with engineering decisions acceptance quality estimation based on organizational and technological designing expert systems are studied. In the research course, the using challenges the expert systems for investment-construction production organizational and technological design were considered. The assessing quality principles and possibilities engineering decisions and intellectualization of the economic analysis functions for investment-construction projects planning and management are formulated. Knowledge scientific and engineering support methods bases and data are offered

Keywords: investment-construction projects, expert systems for organizational and technological design, design systemology, general engineering, database

Проблеми застосування експертних систем при проєктуванні інвестиційно-будівельних проєктів

Уваров П.Є.[1], Татарченко Г.О.[2]*, Білошицька Н.І.[3], Шпарбер М.Є.[4]

[1,2,3,4] Східноукраїнський національний університет імені Володимира Даля
*Адреса для листування E- mail: tatarchenkogalina@gmail.com

Вивчено можливості застосування проєктної системології в задачах планування й управління інвестиційно-будівельною діяльністю з оцінюванням якості прийняття різних видів інженерних рішень на базі експертних систем організаційно-технологічного проєктування. Вивчення цього проблемного напряму дозволить розробляти наукові принципи, методологічні положення і практичні основи створення й використання експертних систем з метою підвищення ефективності та оцінювання якості інжинірингу в ході проєктування і управління інвестиційно-будівельними проектами. У процесі проведених досліджень розглянуто проблеми застосування експертних систем для організаційно-технологічного проєктування інвестиційно-будівельного виробництва; визначено предметні галузі знань комплексного інжинірингу щодо розроблення експертних систем для підготовки інвестиційно-будівельного виробництва та управління проєктами; сформульовано принципи і можливості оцінювання якості прийняття інженерних рішень й інтелектуалізації функцій економічного аналізу, планування та управління інвестиційно-будівельними проєктами; запропоновано методика і принципи побудови експертних систем на основі методів науково-інженерного супроводу формування баз знань і даних. Серед першочергових напрямів подальших досліджень обрано можливості розв’язання тих «стикових» завдань, які є визначальними для реалізації системи «проект – об'єкта будівництва»: розподіл обсягів інвестицій та робіт між учасниками інвестиційно-будівельної діяльності, техніко-економічне обґрунтування ефективності інвестицій й техніко-економічного обґрунтування проєкту, складання технічного завдання на проєктування об'єкта будівництва, проєктування методів і засобів будівництва, експлуатації та реновацій на етапах і стадіях життєвого циклу системи «проект – об'єкта будівництва», прийняття раціональних інтегрованих рішень з монтажу технологічної та будівельної частин проєкту, моніторинг експлуатації й реновації системи «проект – об'єкта будівництва».

Ключові слова: інвестиційно-будівельні проєкти, експертні системи організаційно-технологічного проєктування, проєктна системологія, комплексний інжиніринг баз знань і даних

Introduction

The design systematology in planning and management of the investment-construction activity (ICA) projects together with the different engineering decisions making a quality assessment based on the expert systems for the organizational and technological design (ES-OTD) are considered as one of the challenging directions. They allow developing the scientific principles, methodological provisions, and practical foundations for the expert systems creation and use in order to increase the efficiency and upgrade the quality of engineering in designing and managing the ICA projects [1, 2].

The studies have established that 50% of the research and technical development tasks, as well as over 70% of the organizational and technological preparation tasks and the ICA, projects engineering and legal support, engineering field regardless, require the heuristic procedures use, symbolic coding methods, symbolic logic. More requirements are the professional qualitative evaluation involvement of the experience, as well as the highly qualified experts knowledge in the integrated engineering services related to developing and making different types of engineering and economic decisions (Fig. 1) [2, 5].

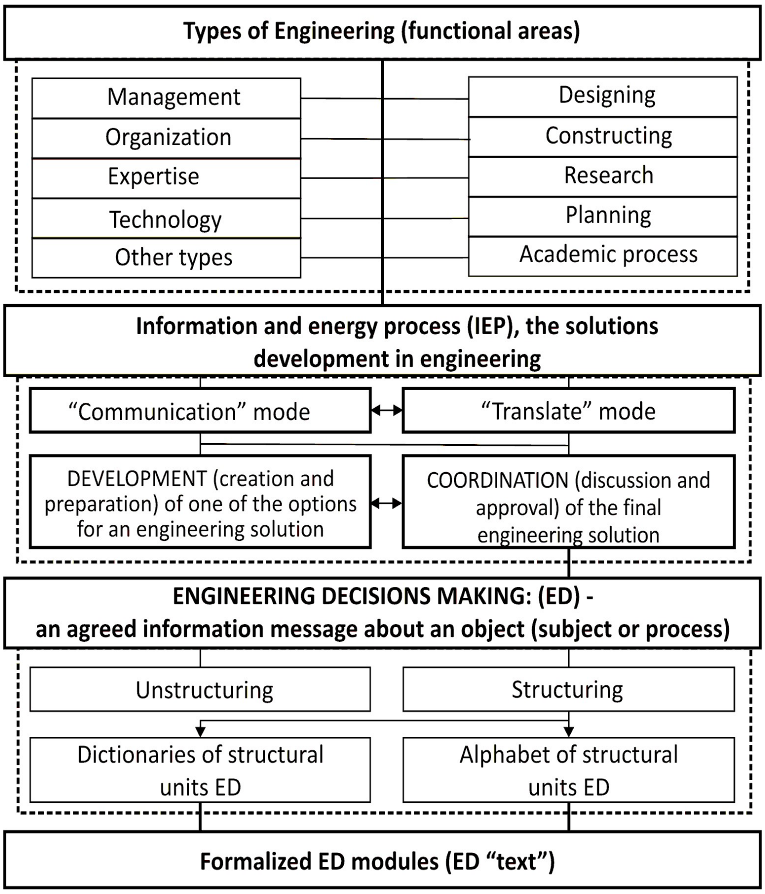

Figure 1 – Engineering decision making in the ICA project systemology

Among the main reasons for this phenomenon, it should be mentioned first of all: it is difficult to formalize the «butt» tasks nature of the investment-constructing projects (ICP) macro- and micro-designing and management; as well as unpredictability and uncertainty of many external environment factors affecting the ICA life cycle dynamics. Secondly, the existing market economy regulatory and legal framework structure is cumbersome and complex (contradictory); its many years of attempts to harmonize with EU standards. Moreover, the inadequacy of the logic mathematical and axiomatic methods and models to the real conditions and principles of the project - construction object (P-CO) general integrated engineering and management organizational forms in their full life cycle development [5, 6]. (Figure 2).

The current situation analysis allows concluding the timeliness and necessity of domestic and foreign experience generalization and systematization in terms of the developing system approaches, updating methods and tools of the expert systems that use information knowledge and procedures for solving poorly formalized general ICP engineering problems. Therefore, the expert systems (ES-OTD) formation funds utilization methodology, which is capable to accumulate, store and purposefully transform information, "derive" new knowledge from the existing, generalize and systematize the experience, self-study, and adapt to engineering changing conditions is an urgent problem, both in the design and its implementation in the ICA projects management.

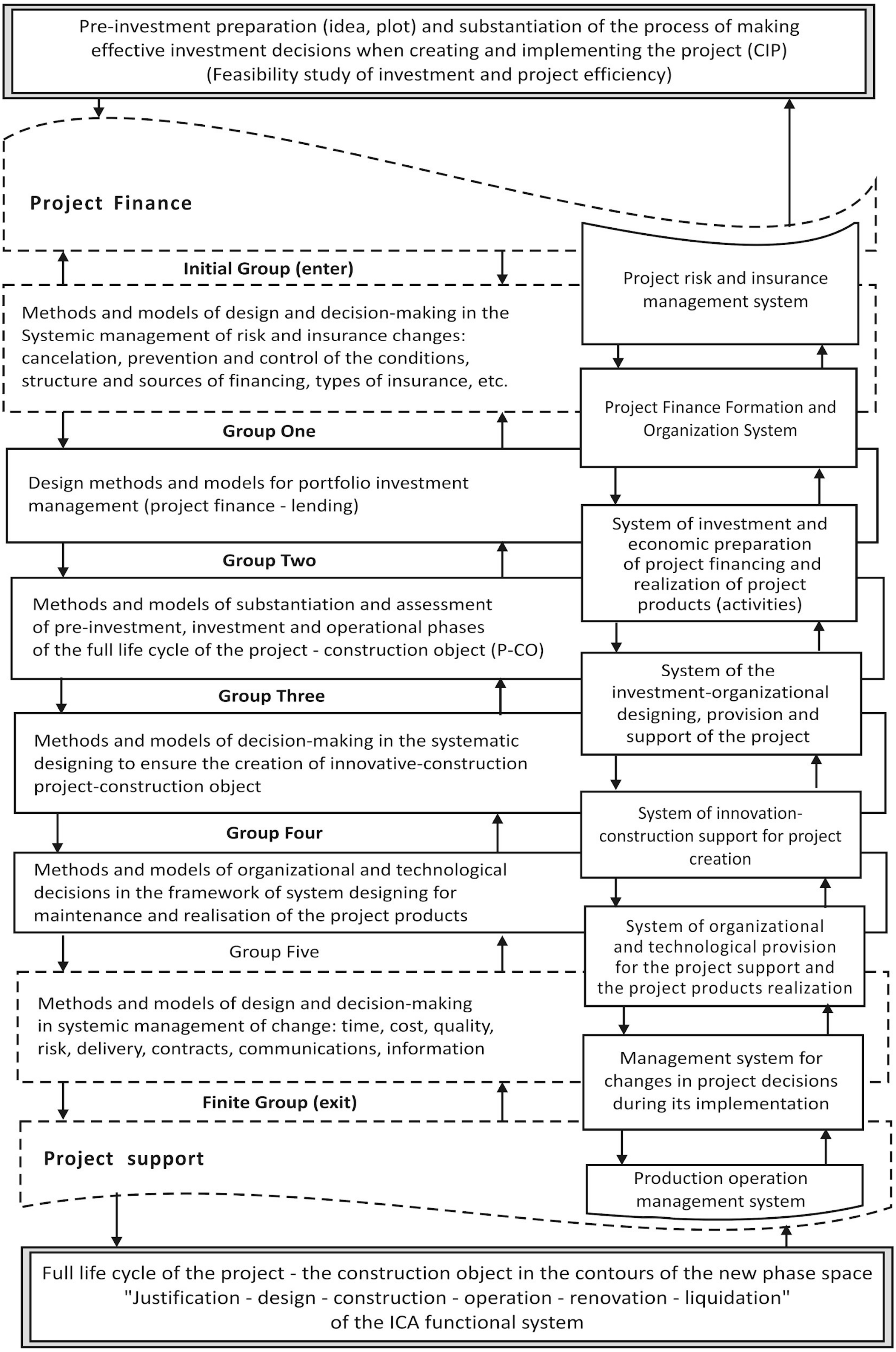

Figure 2 – Functional diagram of the "butt" ICA groups tasks (in the context of the design solidity concept, rationality and constructivism of scientific and engineering decision-making and the project behaviour control)

Review of the research sources and publications

The ICA project systematology field as well as the general engineering P-CO field is under active study on creating and improving methods and expert system-program that operates not with the algorithms, digits, and formulas but uses language logic, semantic structures, and symbols that simulate human behavior, using knowledge and inference procedures to solve poorly formalized problems. Expert system methods typically use a logic-linguistic model (fig. 3) and its interface has two main functions: to provide advice and explanations to the user and to manage the knowledge acquisition. [2, 3, 6, 5, 8].

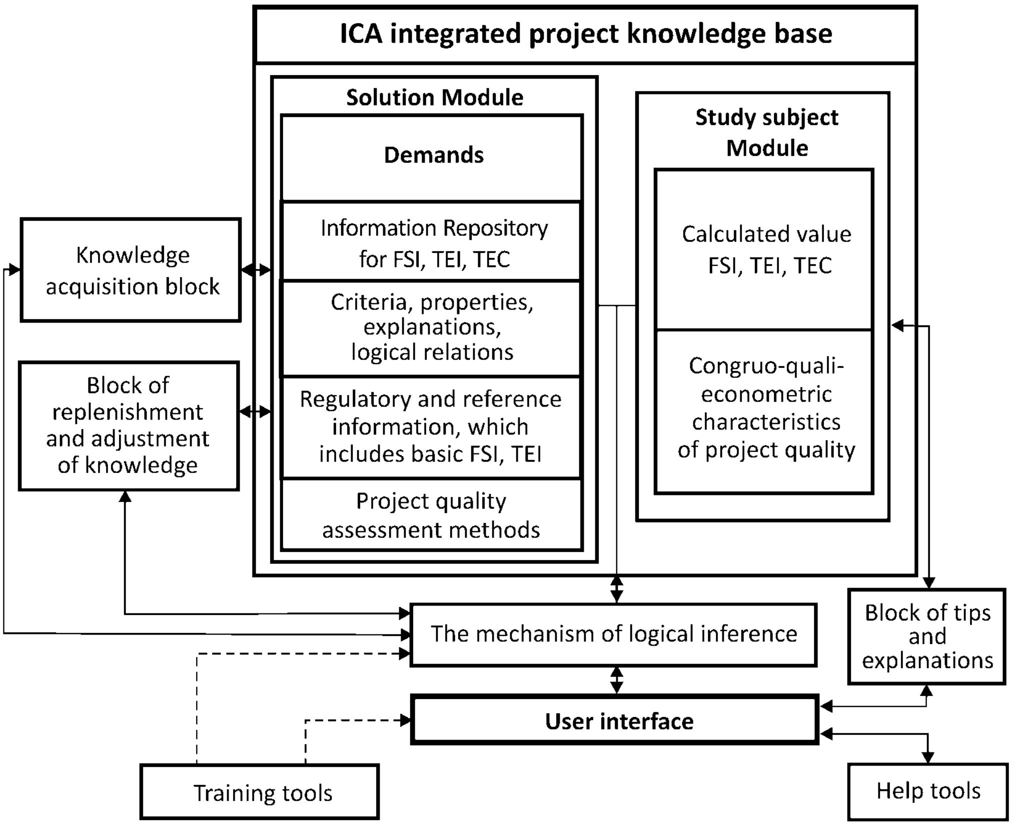

Figure 3 – The structural diagram of the ES-OTP management for quality assessment of innovative ICP integrated engineering models: feasibility study for investment (fsi); technical and economic indicators (tei); technical and economic calculations (TEC)

The paper considered the conclusions of the analytical paper of the scientific school leading specialists V. Glushkov, A. Gusakov, N. Ilyin, Yu. Bogomolov, J. Jones, E. Zavadskas and other scientists and specialists of the domestic and foreign system engineers schools for the expert systems in management and construction. It can be assumed that a unified approach has not yet been developed to a comprehensive quality assessment of the ES-OTP creation and implementation, and engineering and managerial decisions making in the project engineering support. [1, 2, 5, 7].

Definition of unsolved aspects of the problem

The purpose of such an assessment may be one of the project product management (PPM) parameters number: meeting consumer demand for the products that meet the world standards innovative level; the project viability; introduction of the technologies and equipment advanced systems to ensure the production; acceleration of the object commissioning and capacities development; compliance with all resources types the strictest economy; maintaining the ecological balance, etc.

Problem statement – to analyze the challenges using the expert systems for ICA organizational-technical designing and project management. To propose the concept, methodology, and principles to construct ES-OTD (content and structure), based on the methods of research-technical support for the creation of the knowledge database. To justify the approaches regarding the general scientific engineering tasks related to creating and making (selecting) a decision from a variety of alternative options using knowledge and databases, both aiming in increasing the design products competitiveness and quality, and performing other works that determine the composition of engineering support throughout the ICA life cycle.

Basic material and results

The informational research results have shown that scientific- technical and engineering support for P-CO in the ICA system, as a rule, is a “butt” task solution of the project systematology problems. They could occur on different steps and stages of the life cycle (designing – construction – exploitation – renovation - decommissioning and disposal) with the error minimum risk in uncertain terms that are not regulated by existing rules and standards, or due to a lack of sufficient experience or direct analogs in domestic or global practice.

At the same time, complex scientific, technical, and engineering support, as a service set, includes both consulting and technological, as well as construction engineering. That is why designing is considered as the main methodological link in providing engineering support, the results of which ultimately determine, according to the Multi-Criteria Assessment, the investments and innovations effectiveness in general (both at the macro and micro levels). This is because the implementation of technical, organizational, technological, managerial, and economic project conditions - construction object operation goes through the multi-criteria quality assessment of the comprehensive engineering design solution in the full life cycle of ICP.

That is why the methodology development for the expert systems based on the general engineering knowledge database is one of the prioritized research streams in the ICA global practice. The typical ES-OTP (Fig. 3) has the structure that consists, as a rule, of such basic components as a solver (logical-semantic inference mechanism); the database (operational memory); knowledge database; knowledge acquisition tools; explanations, and dialog interface [1, 2, 5, 6]. The expert system core should be the engineering knowledge and database, which should be accumulated in the construction process.

The basic principles and technology of building ES-OTP require the creator interaction – “knowledge engineer” and experts in and experts in the ICA project systemology subject area. The main task of the “knowledge engineer” is to choose the particular type and sort the knowledge presentation form of engineering activity and decision-making strategy (Fig. 4).

There are two main approaches to solving problems related to justification and decision-making using knowledge bases: 1) ready-made solution selection from an alternatives (options) variety embedded in the knowledge base; 2) solutions formation for individual components that are stored in the knowledge base. At the same time, three types of solution strategy options are methodologically distinguished: direct reasoning (direct inference) chain, reverse inference, and mixed-initiative.

At the same time, conceptually, the technology for the ES-OTP development should include the following main stages:

1. Identification – defining the problem and choosing the ES-OTP subject area engineering activities types.
2. Conceptualization - ES-OTP structure definition, goals, hypotheses, solution strategy, components of formation, software, and technology.
3. Formalization - defining the circle of experts, planning expertise, acquiring and presenting knowledge in a formal symbolic form.
4. Testing – the ES prototype development, program verification, logical and semantic consistency, and effectiveness of the conclusions.
5. Trial operation - checking the ES-OTP efficiency in practice.
6. Improvement - ES-OTP adjustment based on the results of trial operation and industrial (software) operation with the knowledge and data banks replenishment, which include rules, data and criterion engineering models, and various organizational and technological solutions aspects.

At the same time, the considered expert systems of the project engineering (ES-OTP) can methodologically perform certain ICP functions:

- data interpretation to determine their meaning;
- technical and organizational-economic systems state determination;
- system monitoring (including radiation and environmental safety) or continuous interpretation of the project data in real-time or in the ICP phase space;
- future development forecast based on modeling the present and the past;
- activities planning and development to achieve the set goals and its scientific and technical support;
- integrated design and construction of buildings and structures in the full life cycle (creation - operation - renovation - decommissioning and disposal);
- forensic construction and technical expertise (FCTE) when investigating the accidents’ causes in buildings and structures, their parts and elements.

The ES-OTP process is the user’s dialogue with the computer system, where, in response to a question posed by the software complex, the user has the opportunity to obtain expert advice or advice using the professional experts' experience, stored in the database. It is important to note that expert systems significantly reduce the complexity of the task by working with a small, subject-limited human knowledge area.

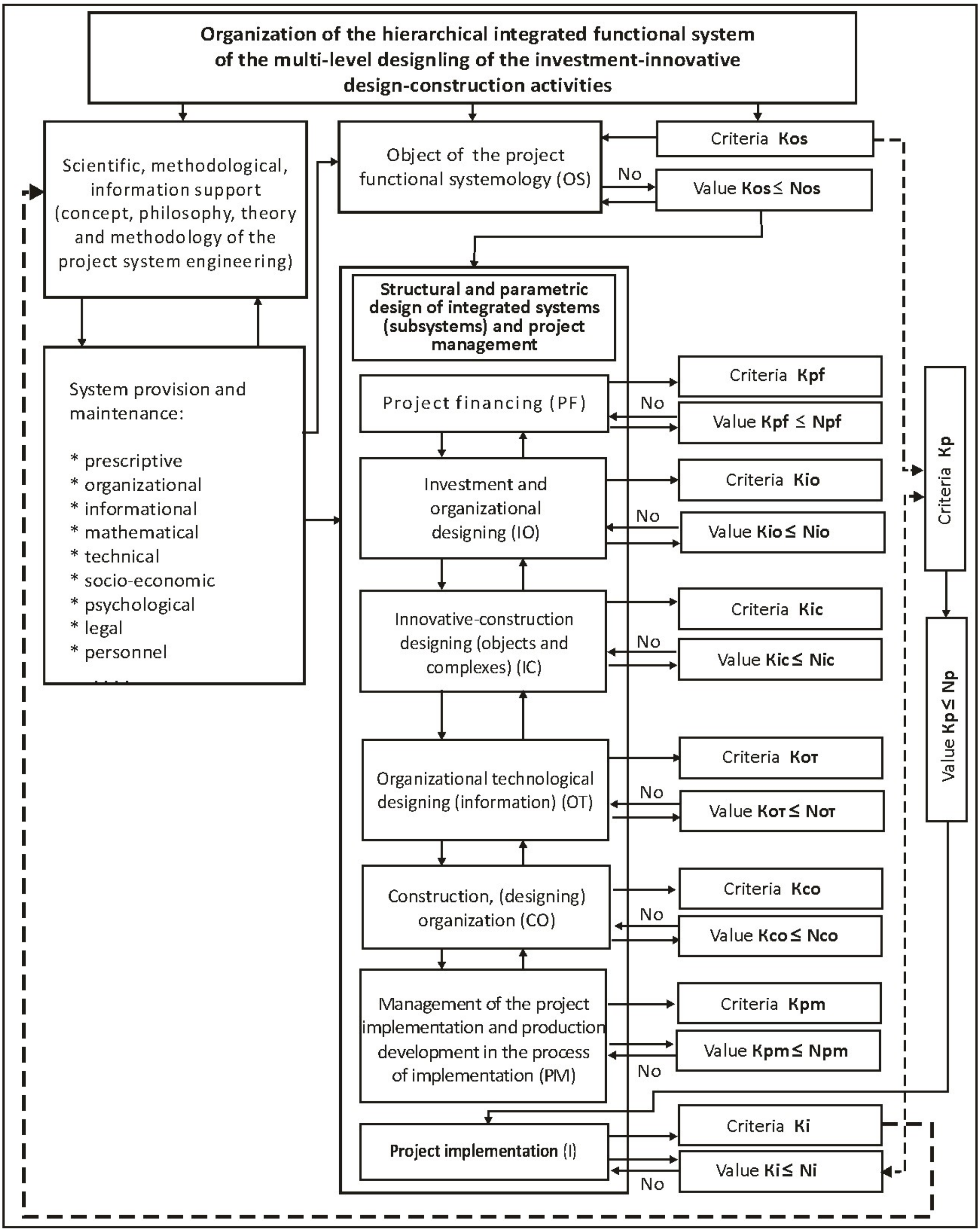

Figure 4 – An enlarged scheme of prescriptive (regulatory) criterion-expert selection of evaluation in ES-OTP:

Kos, Ki, Kp, Kpm - self subordinate criteria in accordance with the generalized assessment of the innovative-designing and investment-construction organization of the project and the design solution implementation;
Nos, Ni, Np, Npm – respectively evaluation criteria normative values;
Kpf, Kio, Kic, Kот, Kco – subsystems local criteria of the organizational and technological design system;
Npf, Nio, Nic, Nот, Nco - local evaluation of local criteria.

Conclusions

1. Due to the large number of self-operating ICA member-organizations, as well as the "butt" tasks considerable volume and complexity in the designing systemology for the formation and the necessity of integrated engineering and management decisions for the preparation and engineering investment-construction projects support are significantly increased by the project decisions making consequences.

2. Certain subject areas of knowledge in general engineering in terms of the expert systems development for the investment-construction production preparation and ICA project management require not so much of calculation procedures and computational operations as logical (meaningful) analysis, synthesis and adaptability, informal methods, qualitative assessments, and the specialists experience.

3. Among the prioritized development areas could be selected and practically implemented solutions of those "butt" tasks that are decisive for the implementation of the projects-construction object – this is the investment and work distribution between the ICA participants, the investment efficiency feasibility study, and project feasibility study in the reference terms development for the construction object design, designing methods and construction means, operation and renovation at the P-CO life cycle steps and stages, making rational integrated decisions for the technological and construction project parts installation, monitoring the P-CO operation and renovation, etc.

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New approaches to strategic program-targeted management of Ukrainian construction sector innovation and economic development

Redkin Oleksandr[1], Pahomov Roman[2]*, Zuma Oleksandr

[1] Poltava National Technical Yuri Kondratyuk University https://orcid.org/0000-0002-6449-050X
[2] Poltava National Technical Yuri Kondratyuk University https://orcid.org/0000-0001-9169-8296
[3] Poltava National Technical Yuri Kondratyuk University https://orcid.org/0000-0001-7484-7755
*Corresponding author Email: pahomov_ri@ukr.net

The situation analysis is carried out and the problems in the innovative and economic sphere of Ukraine development during its independence are revealed. Theoretical researches of modern tendencies, processes and procedures concerning progress organization and management of the leading countries and companies of the world are carried out. The model and algorithm of practical realization of this process are offered. The characteristic of structural and hierarchical construction of the strategic program-target management system of Ukraine progress and its elements is presented. The sequence of effective improvement of the public administration sustainable development system of the national economy and social life is determined. The practical implementation of the proposals outlined in this paper will help accelerate Ukraine's progress towards global economic standards.

Keywords: construction sector, innovation and economic development, program-target management, strategic planning

Нові підходи до стратегічного програмно-цільового управління інноваційно-економічним розвитком будівельного сектора України

Редкін О.В.[1], Пахомов Р.І.[2]*, Зима О.Є.[3]

[1] Полтавський національний технічний університет імені Юрія Кондратюка
[2] Полтавський національний технічний університет імені Юрія Кондратюка
[3] Полтавський національний технічний університет імені Юрія Кондратюка
*Адреса для листування E-mail: pahomov_ri@ukr.net

Установлено, що характерною рисою сучасної глобальної економічної системи є активний вплив держави та її органів влади на стратегічний розвиток передових галузей науки, виробництва й бізнесу, забезпечення їх інтеграції в єдиний механізм науково-технічного та соціально-економічного прогресу постіндустріального суспільства, в якому задіяні не тільки окремі країни та органи їх державного управління, а й провідні компанії світу. Проведено аналіз стану та виявлено проблеми в інноваційно-економічній сфері розвитку України за часів її незалежності. Здійснено теоретичні дослідження сучасних тенденцій, процесів і процедур щодо організації та управління прогресом передових країн і компаній світу. Розроблено нові підходи до формування системи та організації процесів стратегічного планування і програмно-цільового управління інноваційним та економічним розвитком держави, її регіонів, галузей господарювання і підприємств, здатних створювати та реалізовувати на світових ринках наукоємну конкуренто-спроможну продукцію з високим рівнем доданої вартості. Розроблено практичні рекомендації щодо ефективної реалізації цього процесу, зокрема, запропоновано модель і алгоритм стратегічного планування й програмно-цільового управління соціально-економічним та інноваційним розвитком України. Наведено характеристику основних параметрів і елементів структурно-ієрархічної організації системи державного управління стратегічними та програмно-цільовими заходами в процесах розвитку держави та її елементів. На основі узагальнення світового досвіду провідних країн і компаній світу сформовано пропозиції щодо покрокової практичної реалізації ключових заходів, процедур та інструментів стратегічного програмно-цільового управління інноваційно-економічним прогресом України. Практична реалізація розроблених пропозицій буде сприяти прискоренню прогресу України за світовими стандартами господарювання

Ключові слова: будівельний сектор, інноваційно-економічний розвиток, програмно-цільове управління, стратегічне планування

Introduction

A characteristic feature of the modern global economic system is the active influence of the state and its authorities on the strategic development of advanced science branches production and business, ensuring their integration into a single mechanism of scientific, technical and socio-economic progress post-industrial society. According to the practice of the most developed countries of the world, public strategic management of the economy is a system of effective strategic measures of legislative, executive and controlling nature. These measures are implemented by relevant government agencies and public organizations with the aim of stabilization, quality transformation and accelerated transition to sustainable progress of the whole society. This is achieved by the fact that the central authorities determine the most promising directions of the country's strategic development for 5 - 10 years and more and ensure the implementation of the plans with financial and other resources. An innovative model of economic growth is at the heart of these countries' progress.

Ukraine has not created its own mechanism and management system for its development, has not defined a clear strategy for the implementation of this process, the topic of this article is relevant, has scientific novelty and practical value.

Review of the research sources and publications

Analysis of well-known publications [1-18] showed that although today there is some research in the field of theory and practice of innovation and investment management, development of the economy and its production systems, including the use of modern standards of strategic and project management, but they are not enough for the practical implementation in the new conditions of transformation of the economy, public life and, especially, the system of public administration. Today there are still no effective mechanisms, procedures and tools for strategic planning, multi-project and project management of innovative development of the national economy, its high-tech research and production systems and enterprises.

Definition of unsolved aspects of the problem

Despite the experience of successful progress of the leading countries and companies of the world, a considerable number of theories of providing innovative development of different economic systems, today the domestic economy and its enterprises are unsatisfactory. There is a constant manifestation of crisis phenomena in the economy and public life due to the chaotic nature and inefficiency of previous transformations in the state and business. A clear confirmation of this conclusion is the fact that the National Strategy and Development Program of Ukraine has not been developed and legally approved to date, and there are practically no state policies and effective mechanisms for managing this process. That is why it is possible to determine that in our country new systems and tools for strategic and current management of innovation and economic progress of the state, its regions, leading industries and enterprises should be created. At the same time, any development strategies and programs should be aimed at building modern types of high-tech research and production systems and powerful enterprises in Ukraine. At the same time, it is necessary to ensure the formation of a professional and effective structure and processes for managing the functioning and development of socio-economic and production-economic systems, capable of creating and selling the latest high-value products and services with a high level of added value, to increase the competitiveness and well-being of the whole society. The above factors highlight the relevance of the scientific and practical research topic of this article, which reveals the essence, the main purpose and main results.

Problem statement

The purpose of the work is to highlight the results of theoretical research and practical recommendations on the formation in Ukraine of a new system, mechanisms and procedures of strategic, program-targeted and project management of innovation and economic development of the state, national economy, its high-tech scientific-production systems and enterprises capable of creating science-intensive and competitive, high value-added products.

Basic material and results

Today, strategic innovation policy should be an integral part of the overall system of state strategic management of Ukraine's socio-economic and innovation development. It should be based on: a national strategy for innovation and organizational and economic transformation; a set of national measures, through which certain goals of development are realized.

Strategic planning and program-targeting, as the main form of state regulation of innovation and economic development, have been widely used in the countries of the European Union (EU), Japan, South Korea, China. It can be noted that today, in the EU, ten directions of a pan-European wide-ranging innovation strategy are being implemented in the EU on the basis of a program-based method: development of an innovation-oriented education system; formation of the European Institute of Technology; creating a single attractive research-oriented labor market; Strengthening links between research organizations and industry promoting regional innovation and development through cooperation policy programs; reforming the rules of state support for scientific research; strengthening the protection of intellectual property rights; development of digital products and information service; formation of "advanced" markets, favorable for innovation; stimulating innovation through procurement. The key instruments of the EU strategic development policy for recent years have been targeted programs such as: VIII Horizon 2020 Framework Program for Scientific Research 2014 - 2020; EURIKA; EUROSTAR; COST program and more. [5].

Considering Ukraine, it should be noted that with considerable experience of theoretical research, a large number of projects of national strategies and programs in the field of innovative progress of the state and its economy have been developed, but have not yet been adopted due to the lack of a separate legislative and regulatory framework, the organizational and methodological basis for strategic planning and programming of innovative development of the domestic economy, as well as due to the organizational and functional imperfection of the system of state strategic management. Therefore, as noted above, the issue of state regulation and strategic planning organization needs further refinement and is therefore also part of this study.

The innovative progress of the economy today should become an integral part of the overall process of functioning and development of the state. With this in mind, the methodological approach to the strategic planning organization in the field of Ukraine state regulation of socio-economic and innovative development can be schematically reflected in the model of the system formation of national values, interests and national progress goals, as shown in Fig. 1.

Today, the state strategic planning of socio-economic and innovative development of Ukraine is based on the procedures of program-targeted approach with the use of modern tools of scientific prediction of economic progress, including foresight methodology, as one of the leading tools of scientific forecasting. Such planning is part of the policy formulation and implementation process. In the development and implementation of conceptual and practical documents of socio-economic development of the country, it is necessary to solve the following tasks: to scientifically substantiate and determine the long-term goals of social development; identify directions and develop an effective algorithm for achieving goals by coordinating central and local government, corporate systems, institutions, public associations; focus efforts and resources on priority areas of development. It should be remembered that the programmatic approach is a kind of multiproject management, which is widely recognized in leading countries and companies of the world on the basis of American, Japanese, European and other standards of project management (PMBoK, P2M, IPMA, PRINCE) [1–6].

The process of forming the conceptual frameworks and programming documents should begin 2 years before the end of the current Strategy and the National Program. The process of forming and implementing strategic and programmatic documents covers the following three steps.

Stage 1 covers the development of the state Strategy of socio-economic development. At this stage, the situation is analyzed, the dynamics of its changes in the country and the international environment are revealed, negative and positive tendencies and their causes are evaluated, the forecast of the state for the long and medium term, the methodology of foresight is applied to predict the development of the situation and determine the priorities of the country's development.

An obligatory condition for the stage completion is the procedure of promulgation, discussion and mutual agreement of the provisions of the Ukraine Strategy of social and economic development, its consideration and approval by the Verkhovna Rada of Ukraine.

At the second stage, the National Program of Social and Economic Development is being developed. At this stage, the following steps are implemented:

• set tasks for solving the problems identified by the Strategy, directions for solving them and achieving the strategic goals; a competition for the development of projects on the mentioned issues, terms and procedure of holding is announced;

• development, examination and selection of projects as ways of tasks accomplishment are carried out;

• generalize and coordinate projects and regional development programs, existing and new international, targeted state programs, develop an action plan;

• developing a cost estimate of the consolidated plan for all resources;

• an organizational and executive structure is formed for the implementation of the National Program - the main executors, co-executors, control bodies are determined and their functions are determined.

The National Program can be adjusted every year at the same time as the approval of the short-term Program of Socio-Economic Development of Ukraine and the State Budget for the following year (Fig. 1).

Stage 3 concerns the implementation of the Program, the provision of tasks to the contractors and the direct implementation of program activities. At this stage, the implementation of the developed measures is monitored, interim and final results are adopted, their compliance with the goals and the budget of the expenditures, changes to the tasks, regrouping of available resources and means are evaluated.

At the end of this phase, the Verkhovna Rada of Ukraine should review the report on the implementation of the National Program and draw conclusions that are made public through the media.

It is also worth noting that the annual budget plans for the implementation of the National Program of Social and Economic Development are drawn up and balanced during the development of the State Budget.

In parallel with the formation and implementation process of work individual stages and complexes of the Ukraine National Strategy and Program of Social and Economic Development, appropriate measures to ensure innovative development of the national economy should be developed, coordinated, approved and implemented. The process organization conceptual model of formation and implementation of strategic and programmatic documents on state management of innovative economy development can be presented in the form given in Table. 1.

Based on the theoretical and practical experience of organizing state strategic planning and programming of innovative progress in the world and Ukraine (since its independence), the following general conclusions and suggestions can be made regarding the practical improvement of strategic state management of innovative development of the domestic economy.

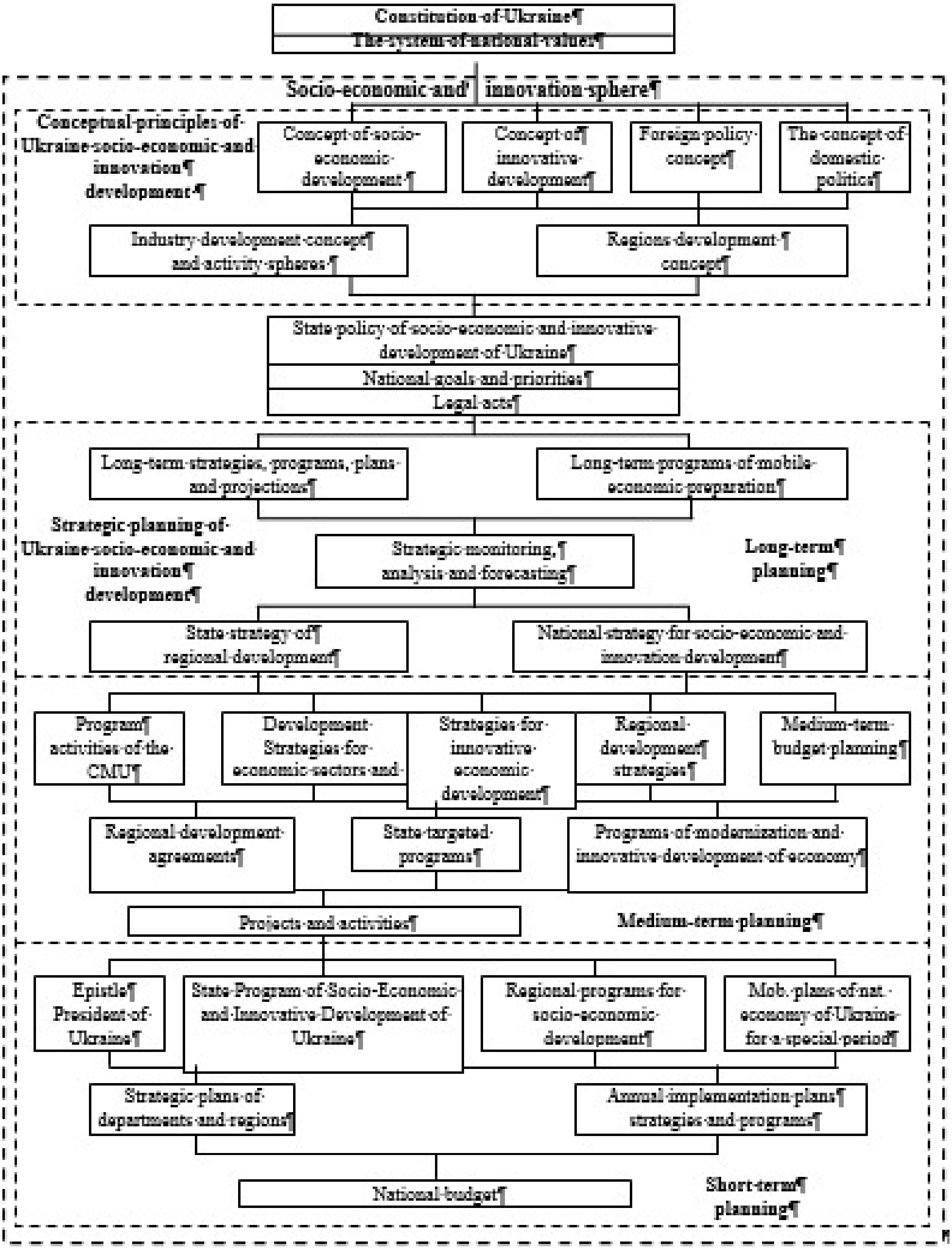

Figure 1 – Scheme of strategic program-targeted planning in the sphere of public administration of socio-economic and innovative development of Ukraine (improved on the basis of [11])

Table 1 – Characteristics of the structural-hierarchical organization of public administration of strategic and program-targeted measures for innovative economic development

Characteristics of organizational and process elements of development management	Characteristics of a strategic state management of economic development hierarchical organization			
	the highest nationwide management level	central government authorities	regional and local authorities	management teams managing the Targeted Development Programs (TDP)
	strategic program-planning and state regulation of economic development			project management
Organizational structures of the entity managing the innovative development of the national economy	1. President of Ukraine, Committee on Economic Reforms and State Development. 2. Cabinet of Ministers of Ukraine, interagency coordination council. 3. The Verkhovna Rada of Ukraine. 4. Ministries and other authorities.	1. Ministry of Finance, Ministry of Economic Development and Trade. 2. Other central executive bodies are their strategic planning departments.	1. Regional State Administration. 2. Regional Council of People's Deputies. 3. Regional Offices for Science, Technology and Innovation.	1. Sectoral, regional and other TDP project management offices. 2. Organizational structures of TDP management teams.
Organizational measures and methodological support for strategic and programmatic management	1. Initiation of actions and coordination management based on the Constitution and Laws of Ukraine. 2. Methodology and standards for strategic management and implementation of strategic national measures.	1. Organization of strategic planning of economic sectors. 2. Regulatory and methodological support for development planning. 3. Organizational and methodological support for improving education.	1. Organization of strategic planning for regional development. 2. Organizational and methodological support for improving education.	1. TDP Planning organization. 2. Development and application of project management standards. 3. Implementation of maturity models for project management skills.
Objective results of managerial influence that characterize the essence of a management object	1. National strategic development priorities (10–30 years). 2. National Concept, Strategy and Development Program. 3. Perspective (for 2–5 years) and annual state plans and budgets of the current implementation of development. 4. Legislation. 5. Regulatory and methodological explanations. 6. Strategic national control.	1. Sectoral development priorities. 2. Industry concepts, strategies and development programs. 3. Sectoral programs and standards of innovation, innovation and technological development. 4. Prospective and annual plans for the implementation of development programs. 5. Regulatory and methodological support, training of staff. 6. Coordination management and controlling of development results.	1. Identifying regional needs and priorities. 2. Regional concepts, strategies and programs of socio-economic development. 3. Regional programs of innovative development. 4. Regulatory and methodological support, training and advanced training. 5. Coordination management and controlling of development results.	1. Implementation of targeted development programs. 2. Organization of TDP professional project management. 3. Ensuring the effectiveness of TDP implementation results.

One of the main shortcomings of the previous transformation period of innovation and economic transformations in the country was the lack to date of the National Concept, Strategy and Program of Socio-Economic and Innovative Development of Ukraine. This shortcoming has largely caused the inconsistent nature of the organization and governance of previous reforms. It enhances the effect of another strategic mistake of the period 1991 - 2020 – the application of the philosophy of the state policy implementation of the domestic economic transformation on the basis of the market self-regulation neoliberal mechanism and the removal of the state from the process of regulating its innovation and technological modernization and socio-economic development. As a result, today Ukrainian society has unfinished economic reforms, increasing the gap between the level of innovation and technological development of the leading countries of the world.

The conducted analysis and generalization of the world and domestic experience of organization and management of economy innovative development allowed to determine ways and improvement procedures of development strategic state program-target regulation and its strategic innovation policy in this sphere, which are given in Table. 2.

Table 2 – Generalization of the experience of innovative development strategic state regulation in Ukraine and other countries of the world

World practical experience of innovative and sustainable economic development	Modern strategic state innovation policy of development of Ukraine	
	disadvantages and remarks	suggestions for improvement
It is established that each country has a very specific national character and diverse approaches to the formation and implementation of national innovation policy. At the same time, common aspects of development are: 1. Each country makes its progress on the basis of the methodology of strategic management of national development programs and projects. 2. The generator and basis of development is the creation and effective activity of an extensive network of the national innovation system and the variety of forms of its participation in innovative projects and programs. 3. In the EU and other socially, economically and technologically developed countries of the world, development is being implemented on the basis of the strategy of national, regional and local development of territories. 4. State regulation of innovation processes is carried out in a wide range of administrative interventions and regulation in development management from the centralized programmatic to the liberal-state mechanism of support and stimulation.	1. Formal definition and weak adherence by the authorities of the legislation and state normative-legal acts on the regulation of the innovation sphere (activity, processes and development, etc.), including scientific and scientific-technical policy. 2. Lack of a clear mechanism for the practical implementation of regulatory functions and instruments in the state innovation policy. 3. Absence of the Concept, Strategy and Development Program of Ukraine developed and legally enshrined. 4. Absence of complex reforms in the main spheres of economy and public life.	1. Formation of a more sophisticated and practical application of the content of strategic and current state innovation development policy, which covers the legislative and procedurally defined Concept, Strategy, Innovation Development Program, procedures and instruments for their implementation. 2. Formation of clear and effective mechanisms for aligning them with strategic processes of managing other spheres of development of the national economy and life of society. 3. Transition to the methodology of strategic program-target and project management of development, the main elements of which should be: 3.1. Identification of contemporary social needs and interests. 3.2. Macroeconomic forecasting and planning. 3.3. Hierarchical definition of strategic, tactical and current development goals. 3.4. Ensuring social orientation of the market economy and development processes. 3.5. Development and approval of a new Concept for Sustainable Socio-Economic and Innovative Development. 3.6. Development and approval of a new National Sustainable Development Strategy and Plan (Program). 3.7. The transition to state programmatic planning and strategic management. 4. Taking into account world and national experience of planning, organizing, coordinating, controlling and regulating the course of implementation and ensuring the effectiveness of development results.

Conclusions

The authors of the article are convinced that the theoretical results and practical recommendations given in this work will allow not only qualitatively and effectively to modernize the system of strategic management of the country's innovation and economic development, but also to ensure its rapid progress on the basis of attracting the best world experience and creating its own mechanism of sustainable economic progress and the whole society by the advanced standards of this process implementation.

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