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The research of the operating mode of the concrete mixture plane depth compactor with a circular vibration exciter

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The paper proposes an essentially new design of a concrete mixture depth compactor made in the form of a vertical plate with a circular vibration exciter mounted on its upper part. A design diagram of the «plane depth compactor – concrete mixture» dynamic system is presented and the vertical plate movement equations describing its linear vibrations in the horizontal plane and torsional vibrations in relation to the center of gravity are provided. It has been determined the regularity of the vertical plate motion during the compaction of concrete mixtures. The provided results of the research enable the substantiation of the rational parameters of the plane depth compactor performing spatial vibrations and the efficient modes of the vibratory action on the concrete mixtures of different consistence.

Keywords: plane depth compactor, concrete mixture, vibration compaction.

Дослідження робочого режиму площинного глибинного ущільнювача бетонних сумішей з вібророзбудувачем кругових коливань

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Запропонована принципово нова конструкція глибинного ущільнювача бетонних сумішей, що виконана у вигляді вертикальної влити, на верхній частині якої змонтовано віброзбуджувач кругових коливань. Описано принцип дії площинного глибинного ущільнювача бетонних сумішей. Складена розрахункова схема динамічної системи «площинний глибинний ущільнювач – бетонне середовище», в якій останнє представлено у вигляді наведеної маси і наведених коефіцієнтів непружного і пружного опорів. Знайдено момент інерції наведеної маси бетонної суміші щодо центру ваги системи, що коливається. Визначено інерційні, пружні і непружні сили опору бетонної суміші, які діють на вертикальну плиту в процесі їх спільних коливань. Складені рівняння руху вертикальної плити, яка контактує з бетонною сумішшю, що описують її прямолінійні коливання в горизонтальній площині і крутильні коливання відносно центру тяжіння. Рівняння руху враховують дію інерційних сил глибинного ущільнювача і ущільнюваної бетонної суміші, сил пружного і непружного опору бетонної суміші і сил тертя нижньої кромки вертикальної плити про бетонну суміш або основу. Встановлена закономірність руху робочої поверхні вертикальної плити, що взаємодіє з бетонною сумішшю в горизонтальному напрямку і викликає в цьому бетонному середовищі нормальні напруги. При цьому реалізується змінний амплітудно-частотний режим руху вертикальної плити, що викликає в ущільнювальному шарі поширення пружно-пластичних хвиль деформацій, які забезпечують створення імпульсного напружено-деформованого стану, що забезпечує ефективне ущільнення бетонних сумішей пластичних з осадкою конуса ОК-3,5 – 4 см сумішей, так і жорстких сумішей жорсткістю Ж=30 – 120 с Наведені результати досліджень дозволяють обґрунтувати раціональні параметри площинного глибинного ущільнювача, що здійснює просторові коливання, і ефективні режими вібраційної дії на бетонні суміші різної консистенції.

Ключові слова: площинний глибинний ущільнювач, бетонна суміш, вібраційне ущільнення.

Introduction

The depth (internal) vibration of concrete mixtures takes less energy than vibration compaction by other methods. Thus, it is topical to create simple, reliable and highly efficient vibration machines for depth compaction.

Review of research sources and publications

For internal vibration depth vibrators [1 - 4], equipped with circular tips of various diameters, submerged into the concrete mixture, is used. They have a small radius of concrete mixture working and are used for the compaction of plastic concrete mixtures. To improve the productivity it is used packaged depth vibrators with individual drives [5]. The design of these devices is rather complex and they are used for very big volumes of concreting. To improve the vibration efficiency the author of paper [6], proposed a plane depth compactor performed in the form of a vertical flat plate equipped with two depth vibrators, each with an individual drive. Big weight prevented this depth compactor from being used in construction industry as a manual mechanism. Moreover, all the depth vibrators are equipped with planetary vibration exciters that quickly break down [7].

Definition of unsolved aspects of the problem

In the process of the research it is necessary to substantiate the rational parameters of the plane depth compactor of a simple design, high reliability and provision of the compaction of concrete mixtures of different consistence.

Problem statement

The purpose of the paper consists in the development of a highly efficient plane depth vibration compactor for concrete mixtures of different consistence.

Basic material and results

The proposed plane depth compactor (Fig. 1) consists of a compacting plate made in the form of vertical plate 1 with stiffening rib 2, bracket 3, rigidly fixed to plate 1, and mounted on this bracket by means of threaded joints 4 of circular vibration exciter 5 with unbalance shaft 6.

The plane depth compactor operates in the following way.

The operator turns on the plane depth compactor and introduces vertical plate 1 into concrete mixture 7, spread as a smooth layer. Under the action of circular vibration exciter 5 the vertical plate performs complex motions, arousing in the compacted medium the resilient viscous plastic deformation waves with rather high frequency. These deformations cause ultimate destruction of the structural connections in the concrete mixture and transform it into thixotropic condition. They result in the intensive reorientation of the mineral particles, the displacement of the air and the formation of a more compact packing.

To determine the law of motion of the vertical plate interacting with the concrete mixture in the operating mode we consider the design model of the «plane depth compactor – concrete mixture» dynamic system (Fig. 1). The movements of the considered dynamic system under the action of the circular vibration exciter whose unbalance generate circular disturbing force Q have been analyzed. It has been decomposed this force into two components: horizontal $Q \sin \omega t$ and vertical $Q \cos \omega t$ forces.

Under the action of horizontal force $Q \sin \omega t$, the vertical plate interacting with the concrete mixture performs complex movements: linear movements in the direction of coordinate axis X, passing via the center of gravity C of the vibrating system and torsion vibrations about the center of gravity C.

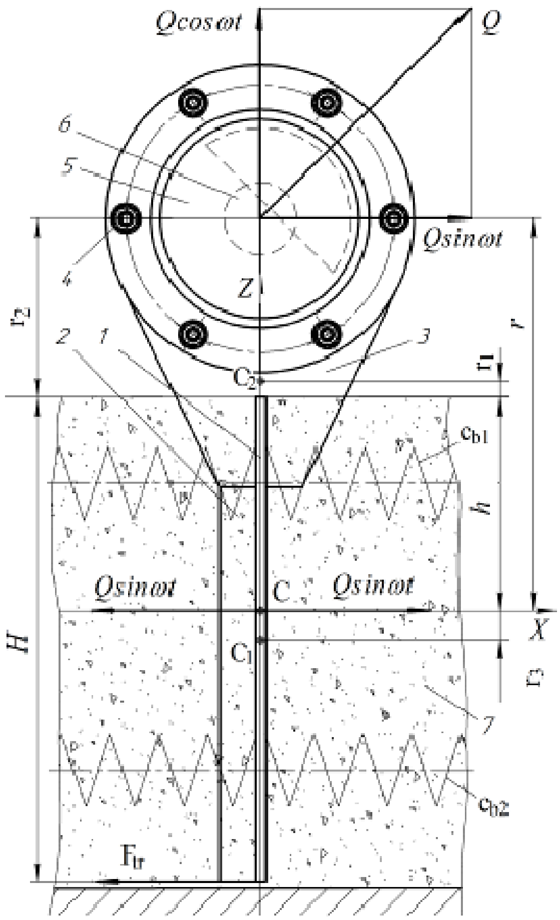

Figure 1 – The design diagram of the «plane depth compactor – concrete mixture» dynamic system

It is determined the vertical shift of the center of gravity C of the vibrating system in relation to the center of gravity of the concrete mixture O_1 from the following dependence

$$r_3 = \frac{m(0{,}5H \pm r_1)}{m + m_b}, \tag{1}$$

where m – the mass of the plane depth compactor;
r_1 – the distance from the center of gravity of the plane depth compactor to the vertical plate upper edge coinciding with the surface of the compacted concrete mixture;

m_b – the modified mass of the concrete mixture determined depending on the method of vibration of the concrete mixture: in half-space [8] or in the form [9].

In expression (1) sign plus is taken in parentheses if the center of gravity of the plane depth compactor is over the upper edge of the vertical plate, and sign minus if it is below.

In this case the movement of the vertical plate contacting the concrete mixture can be described by the following equation system:

– the linear movement in the horizontal plane in the direction of coordinate axis X

$$(m+m_b)\frac{d^2x}{dt^2}+b_b\frac{dx}{dt}+c_bx\pm F_{tr}=Q\sin\omega t\,; \qquad (2)$$

– the angular shift about coordinate axis Y

$$(J+J_b)\frac{d^2\psi}{dt^2}+n_b\frac{d\psi}{dt}+k_b\psi\pm M_{tr}=Qr\sin\omega t\,; \quad (3)$$

where x – the linear movements of the vertical plate over the center of gravity of the vibrating system;

ψ – the angular movements of the vertical plate about coordinate axis Y, passing via the center of gravity of the vibrating system;

m_b, b_b, c_b – the values of the modified mass and modified coefficients of the concrete mixture elastic and non-elastic resistances, determined according to the method of the concrete mixture vibration: in half-space [8] or in a form [9];

F_{tr} – the force of friction of the vertical plate lower edge against the concrete mixture or the base,

$$F_{tr}=mg\cdot f_{tr}\,, \qquad (4)$$

f_{tr} – the coefficient of the friction of the vertical plate lower edge against the concrete mixture or the base;

J – the moment of inertia of the plane depth compactor against the center of gravity C of the vibrating mass,

$$J=J_y+m(0{,}5H-r_3+r_1)^2\,, \qquad (5)$$

J_y – the moment of inertia of the plane depth compactor against its own center of gravity O_2;

r –the distance from the axis of rotation of the unbalance shaft of the vibration exciter to the center of gravity of the vibrating system;

J_b – the moment of inertia of the modified mass of the concrete mixture against the center of gravity of the vibrating system,

$$J_b=m_b\frac{H^2}{12}+m_br_3^2\,; \qquad (6)$$

k_b, n_b – the coefficients of torsional stiffness and non-elastic resistance of the compacted medium at the angular movements of the vibrating system against coordinate axis Y,

$$k_b=k_{b1}+k_{b2}\,;\quad n_b=n_{b1}+n_{b2}\,; \qquad (7)$$

k_{b1}, n_{b1} – the coefficients of torsional stiffness and non-elastic resistance of the compacted medium at the angular movements of the vibrating system against coordinate axis Y at the section of height $h = 0{,}5H–r_3$,

$$k_{b1}=\sum_{i=1}^{n_1}c_{by}F_iz_i^2\,;\quad n_{b1}=\sum_{i=1}^{n_1}b_{by}F_iz_i^2\,; \qquad (8)$$

k_{b2}, n_{b2} – the coefficients of torsional stiffness and non-elastic resistance of the compacted medium at the angular movements of the vibrating system against coordinate axis Y at the section of height $h_1 = 0{,}5H - r_3$,

$$k_{b2}=\sum_{i=1}^{n_2}c_{by}F_iz_i^2\,;\quad n_{b2}=\sum_{i=1}^{n_2}b_{by}F_iz_i^2\,; \qquad (9)$$

F_i – the area of equal elementary sections with the height division of the vertical plate,

$F_i = F / (n_1 + n_2)$;

n_1, n_2 – the number of the divisions of the vertical plate respectively in its upper part at the section of height h and its lower part at the section of height h_1;

Z_i – the vertical distance from the center of gravity of the vibrating system to the i-the marked element;

c_{by}, b_{by} – the values of the specific modified coefficients of the elastic and non-elastic resistances of the concrete mixture determined according to the method of the concrete mixture vibration: in half-space [8] or in a form [9];

M_{tr} – the moment of the forces of friction of the vertical plate lower edge against the concrete mixture or the base,

$$M_{tr}=mg(H-h)f_{tr}\,. \qquad (10)$$

Using the method of linearization of Coulomb friction [10], it is modified equations (2) and (3) to the following form:

$$(m+m_b)\frac{d^2x}{dt^2}+(b_b+b_s)\frac{dx}{dt}+c_bx=Q\sin\omega t\,; \quad (11)$$

$$(J+J_b)\frac{d^2\psi}{dt^2}+(n_b+n_s)\frac{d\psi}{dt}+k_b\psi=Qr\sin\omega t\,, \quad (12)$$

where b_s – the equivalent coefficient of viscous friction in the direction of coordinate axis X,

$$b_s=\frac{4F_{tr}}{\pi A\omega}=\frac{q_r}{A\omega}\,; \qquad (13)$$

n_s – the equivalent coefficient of viscous friction about horizontal axis Y;

$$n_s=\frac{4M_{tr}}{\pi\Phi\omega}=\frac{q_m}{\Phi\omega}\,; \qquad (14)$$

A – the amplitudes of the vertical plate vibrations in the horizontal direction about the center of gravity of the vibrating system;

Φ – the amplitude of the angular (torsion) vibrations of the vertical plate about horizontal axis Y;

$$q_r=\frac{4F_{tr}}{\pi}\,; \qquad (15)$$

$$q_m=\frac{4M_{tr}}{\pi}\,. \qquad (16)$$

Based on the known methods of the classical theory of vibrations [10, 11], it is found the solution to equations (13) and (14) in the following form:

$$x(t) = A\sin(\omega t - \varphi_1)\,; \tag{17}$$

$$\psi(t) = \Phi\sin(\omega t - \varphi_2)\,, \tag{18}$$

where φ_1 – the angle of phases shift between the amplitude of disturbing force Q and the movement along x;

φ_2 – the angle of phases shift between the amplitude of the moment of the disturbing forces and the amplitude of the angle movement;

$$A = \frac{Q}{\sqrt{[c_b - (m + m_b)\omega^2]^2 + (b_b + b_s)^2\omega^2}}\,; \tag{19}$$

$$\Phi = \frac{Qr}{\sqrt{[k_b - (J + J_b)\omega^2]^2 + (n_b + n_s)^2\omega^2}}\,; \tag{20}$$

$$\varphi_1 = arctg\frac{(b_b + b_s)\omega}{c_b - (m + m_b)\omega^2}\,; \tag{21}$$

$$\varphi_2 = arctg\frac{(n_b + n_s)\omega}{k_b - (J + J_b)\omega^2}\,. \tag{22}$$

Substituting dependences (13) and (14) respectively in expressions (19) and (20), we obtain equations for the determination of the amplitudes of the linear and torsion vibrations of the vertical plate in the following form:

$$A^2 + \frac{2Aq_r b_b\omega}{[c_b - (m+m_b)\omega^2]^2 + b_b^2\omega^2} - \frac{Q^2 - q_r^2}{[c_b - (m+m_b)\omega^2]^2 + b_b^2\omega^2} = 0; \tag{23}$$

$$\Phi^2 + \frac{2\Phi q_m n_b\omega}{[k_b - (J+J_b)\omega^2]^2 + n_b^2\omega^2} - \frac{Q^2r^2 - q_m^2}{[k_b - (J+J_b)\omega^2]^2 + n_b^2\omega^2} = 0; \tag{24}$$

Solving equations (23) and (24), we find the final values of the amplitudes of the vertical plate vibrations taking into account the values of the equivalent coefficients of viscous friction b_s, and n_s

$$A = \frac{1}{[c_b - (m+m_b)\omega^2]^2 + b_b^2\omega^2}\left\langle -q_r b_b\omega + \sqrt{q_r^2 b_b^2\omega^2 + M_1\{[c_b - (m+m_b)\omega^2]^2 + b_b^2\omega^2\}}\right\rangle; \tag{25}$$

$$\Phi = \frac{1}{[k_b - (J+J_b)\omega^2]^2 + n_b^2\omega^2}\left\langle -q_m n_b\omega + \sqrt{q_m^2 n_b^2\omega^2 + M_2\{[k_b - (J+J_b)\omega^2]^2 + n_b^2\omega^2\}}\right\rangle, \tag{26}$$

where

$$M_1 = Q^2 - q_r^2\,;\; M_2 = Q^2r^2 - q_m^2\,.$$

We can represent the law of movement of the vertical plate working surface interacting with the concrete mixture in the direction of coordinate axis X, and causing normal stresses in this concrete medium, based on expressions (17) and (18), taking into account expressions (25) and (26), in the form of the following functions

$$X_n(z,t) = x(t) + z\varphi(t) \text{ at } -(H-h) \le z \le h\,. \tag{27}$$

Substituting the values of functions $x(t)$ (17) and $\varphi(t)$ (18) into expression (27) it is obtained the dependence for the description of the law of the movement of the vertical plate working surface contacting with the concrete mixture in the following form

$$X_n(z,t) = A(z)\sin[\omega t + \varphi(z)] \text{ at } -(H-h) \le z \le h\,, \tag{28}$$

where $A(z)$ – the amplitude of the movement of the vertical plate working surface interacting with the concrete mixture, depending on coordinate z,

$$A(z) = \sqrt{A^2 + \Phi^2z^2 + 2A\Phi z\cos(\varphi_1 - \varphi_2)}\,; \tag{29}$$

$\varphi(z)$ – the angle of phases shift between the amplitude of the disturbing load and the amplitude of a certain point movement on the vertical plate with coordinate z,

$$\varphi(z) = arctg\frac{A\sin\varphi_1 + \Phi z\sin\varphi_2}{A\cos\varphi_1 + \Phi z\cos\varphi_2}\,. \tag{30}$$

Considering expressions (13) and (14), it is modified dependences (21) and (22) to the following form:

$$\varphi_1 = arctg\frac{b_b\omega + q_r/A}{c_b - (m + m_b)\omega^2}\,; \tag{31}$$

$$\varphi_2 = arctg\frac{n_b\omega + q_m/\Phi}{k_b - (J + J_b)\omega^2}\,. \tag{32}$$

The analysis of expressions (28 – 30) reveals that the vertical plate of the proposed plane depth vibration compactor performs spatial vibrations during its operation. It provides efficient compaction of the concrete mixture due to alternating amplitude-frequency action. This vibratory action causes normal stresses in the compacted medium in the horizontal plane. They determine the destruction of the structural connections and result in the transformation of the concrete mixture into the thixotropic condition. In this case the forces of internal friction in the mixture sharply decrease due to the release of water, acting as lubrication, into the inter-grain space. The air is displaced out of the compacted mixture, the mineral particles reorient and approach each other forming a more compacted packing.

The obtained dependences used for the functional dependences determination of the concrete mixture compacting process represented in the form of a half-space is analyzed.

Fig. 2 shows the variation of the vertical plate vibrations amplitude of the depth vibration compactor A and stresses σ, occurring in the concrete medium of different consistence at the place of the concrete mixture contact with the vertical plate along its height, i.e. from the upper edge of the vertical plate to its lower edge. In Fig. 2 the origin of coordinates along the height of the vertical plate superposes its upper edge.

The data were obtained at the use of the above given theoretical dependences for the depth plane compactor with the following basic parameters: the depth compactor mass – m = 4.93 kg; the amplitude of the disturbing force of the vibration exciter – Q = 0.981 kN (100 kg); the angular frequency of the forced vibration – ω = 292 rad/s; the distance from the center of gravity of the depth vibration exciter O_2 to the upper edge of the vertical plate – r_1 = 2.85 cm; the moment of inertia of the depth vibration compactor against the axis passing through the center of gravity O_2 – J_y = 0.0365 kg·m^2; the height of the vertical plate – H = 20 cm; the width of the vertical plate – B = 20 cm; the area of the vertical plate surface interacting with the concrete mixture (at the bilateral contact) – F = 800 cm^2.

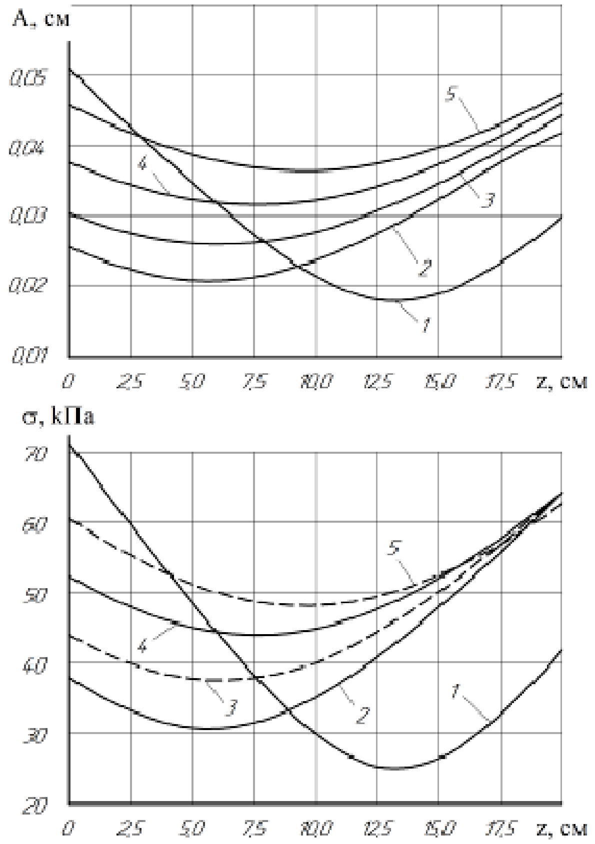

Figure 2 – The variations of the amplitudes of movements A and stresses σ, occurring in the concrete medium of different consistence at the place of its contact with the plane depth vibration compactor depending on the coordinate along the height of the vertical plate:
1 – at the slump of 3.5 – 4 cm;
2 – at mixture hardness H = 30 s;
3 – at H = 60 s; 4 – at H = 90 s; 5 – at H = 120 s

The analysis of the given data (Fig. 2), obtained for the final process of compacting, i.e. at complete compaction ρ_k, reveals that during the process of vibratory compacting by the proposed plane compactor the concrete mixture is subject to the action of the alternating amplitude-frequency vibration along the height.

This vibratory action results from simultaneous linear and torsion vibrations of the vertical plate. The amplitudes change of the vibrations and stresses during the compacting process depending on the mixture relative deformation ε is also continuous. By way of example, Fig. 3 shows the typical change of the amplitudes of vibrations and stresses occurring in the concrete mixture of the hardness of 60 s at the place of its contact with the vertical vibration plate, depending on relative density ε. The farther the vibration source is the lower and smoother the amplitudes of the vibrations in the compacted medium and stresses in it are (Fig. 4).

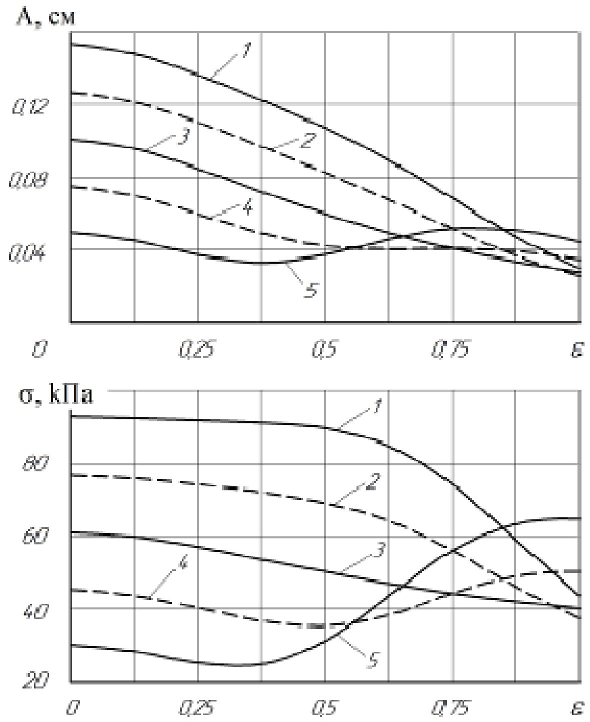

Figure 3 – The variations of the amplitudes of movements A and stresses σ, occurring in the concrete mixture of hardness 60 s at the place of its contact with the plane depth vibration compactor depending on the concrete mixture relative density ε and coordinate z along the height of the vertical plate beginning with its vertical edge:
1 – at z = 0; 2 – at z = 5 cm; 3 – at z = 10 cm;
4 – at z = 15 cm; 5 – at z = 20 cm

It is this ambiguous character of the vibration action on the concrete medium that contributes to the efficient destruction of the internal connections in the concrete mixture, the reorientation of the mineral particles and air displacement with the formation of a more compact packing. The obtained indices determine the amount of the energy put into the concrete medium by the depth plane vibration compactor and enable the determination of the law of the increase of the concrete medium density during the process of its compacting by the vibration action. They also enable the determination of the necessary duration of the vibration action depending on the average values of the amplitudes of vibrations and stresses in the concrete medium, the consistence of the concrete mixture and the distance of the propagation of the resilient-plastic deformation waves providing the required compactness.

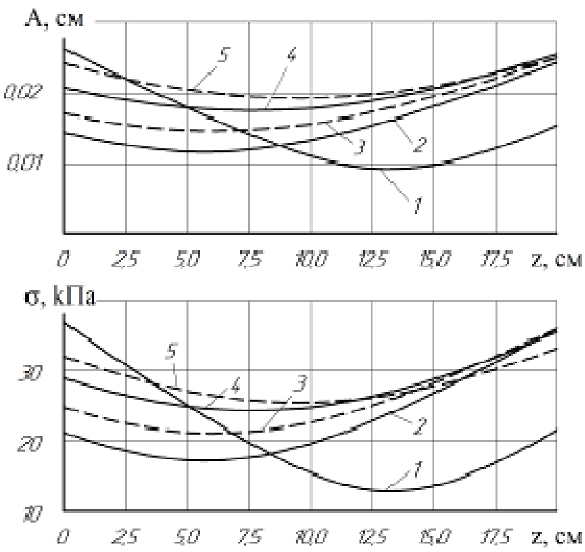

Figure 4 – The amplitudes variations of movements *A* and stresses *σ*, occurring in the concrete medium of different consistence at the distance of *x* = 80 cm from the source of vibration depending on the coordinate along the height of the vertical plate beginning from its vertical edge:
1 – at the slump of 3.5 – 4 cm;
2 – at the mixture hardness H = 30 s;
3 – at H = 60 s; 4 – at H = 90 s; 5 – at H = 120 s

Table 1 contains the values of the required duration of compacting the concrete mixtures of different consistence at the vibration processing length of $x = 80$ cm. The total length of the vibration processing is 160 cm, as vibration processing is performed simultaneously by both the front and the end wall of the plane vibration compactor.

The comparison of the data given in paper [8] and Table 1 reveals that at the equal disturbing force the proposed plane vibration depth compactor, performing simultaneously linear and torsion vibrations, is more efficient than the plane vibration depth compactor performing only linear vibrations. The use of linear and torsion vibrations enables to improve the productivity of the plane depth vibration compactor by 1.62 – 2.32 times at compacting plastic and moderately hard concrete mixtures and by 1.35 – 1.38 times at compacting hard concrete mixtures of the hardness of 60 – 90 s.

Table 2 contains the values of the required duration of compacting the concrete mixtures of different consistence at the length of the vibration processing of $x = 60$ cm and the vibration plate width increased by 1.5 times – $B = 30$ cm, i.e. at somewhat changed parameters of the vibration depth compactor: the mass of the depth compactor – $m = 5.57$ kg; the distance from the center of gravity of the depth vibration compactor to the upper edge of the vertical plate – $r_1 = 0.05$ cm; the moment of inertia of the depth vibration compactor against the axis passing across the center of gravity – J_y = 0.0486 kg·m^2; the area of the vertical plate surface interacting with the concrete mixture (at bilateral contact) – F = 1200 cm^2.

Table 1 – The values of the required duration of the vibration process of compacting t_v concrete mixtures of different consistence at the vibratory processing length x = 80 cm

Concrete mixture consistence	Slump= 3.5–4 cm	H= 30 s	H= 60 s	H= 90 s	H= 120 s
Compacting required time, s	17.0	31.5	41.6	48.7	53.8

Table 2 – The values of the required duration of the vibration process of compacting t_v concrete mixtures of different consistence at the vibration plate width *B* = 30 cm and the vibratory processing length x = 60 cm

Concrete mixture consistence	Slump= 3.5–4 cm	H= 30 s	H= 60 s	H= 90 s	H= 120 s
Compacting required time, s	29.5	60.1	79.2	92.0	100.0

The analysis of the data given in Table 2 reveals that the increase of the area of the interaction of the plane depth vibration compactor by 1.5 times due to the increase of the vibration plate width without the growth of the amplitude of the disturbing force does not result in a significant improvement of the productivity. In this case it is possible to consider admissible (Table 2) plastic concrete mixtures compacting with the slump S = 3.5 – 4 cm and hard concrete mixtures compacting of hardness 30 – 90 s.

Fig. 5 shows the change of the amplitude of the vibrations of the vertical plate of the depth vibration compactor *A* and stresses *σ*, occurring in the concrete medium of different consistence at the place of the concrete mixture contact with the vertical plate along its height at somewhat changed parameters of the vibration depth compactor: the mass of the depth compactor – m = 5.57 kg; the distance from the center of gravity of the depth vibration compactor O_2 to the upper edge of the vertical plate – r_1 = 0; the moment of inertia of the depth vibration compactor against the axis passing across the center of gravity O_2 – J_y = 0.0803 kg·m^2 ; the area of the vertical plate surface interacting with the concrete mixture (at bilateral contact) – F = 1200 cm^2; the height of the vertical plate – H = 30 cm.

The data (Fig. 5) of the change of the amplitude and stresses, occurring at the place of the vertical vibration plate contact with the concrete mixtures of different consistence, are given for the final stage of the compacting vibration process when hardness ρ_k is achieved. Fig. 6 shows the change of the stresses occurring in the concrete mixture at the distance of 60 and 80 cm from the vibration source.

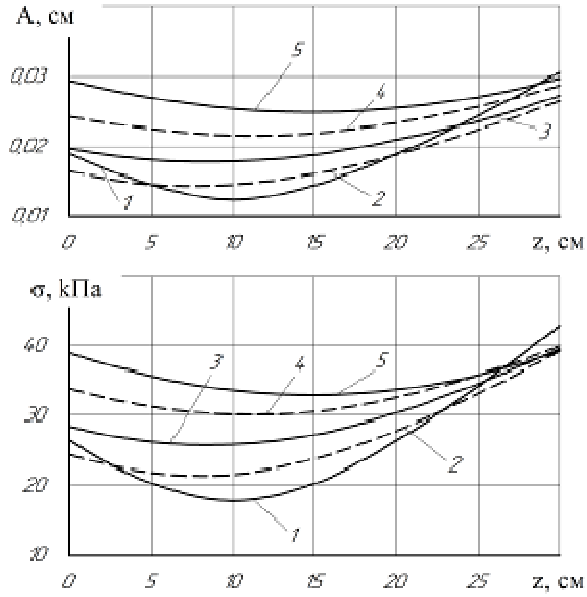

Figure 5 – The variations of the amplitudes of movements *A* and stresses σ, occurring in the concrete medium of different consistence at the place of its contact with the depth vibration compactor vertical plate depending on the coordinate along the height of the vertical plate from 0 to 30 cm:
1 – at the slump 3.5 – 4 cm;
2 – at the mixture hardness H = 30 s;
3 – at H = 60 s; 4 – at H = 90 s; 5 – at H = 120 s

The analysis of the data given in Figs. 5 and 6 reveals that the 1.5-time increase of the area of the vertical plate interaction with the concrete mixture due to the increase of the height changes the character of the variation of the amplitudes and stresses to some degree in comparison with other data stated above. The farther the vibration source is the essentially lower and somewhat smoother the amplitudes of the stresses in the compacted medium are.

Table 3 contains the values of the required durability of compacting the concrete mixtures of different consistence at the vibratory processing length of $x = 60$ cm and $x = 80$ cm, and at the vibration plate height increased by 1.5 times – $H = 30$ cm.

It follows from Table 3 that the growth of the area of the interaction of the plane depth vibration compactor by 1.5 times due to the increase of the vibration plate height without the growth of the amplitude of the disturbing force does not result in significant improvement of the productivity. In this case, it is possible to consider admissible (Table 3) the compaction of plastic concrete mixtures with the slump $S = 3.5 – 4$ cm, and of hard concrete mixtures of hardness 30 – 90 s at the processing length 60 cm and 30 – 60 s at the processing length of 80 cm. To reduce the duration of the vibration process at the increased area of the vertical plate interaction with the concrete mixture it is reasonable to increase the amplitude of the disturbing force of the vibration exciter.

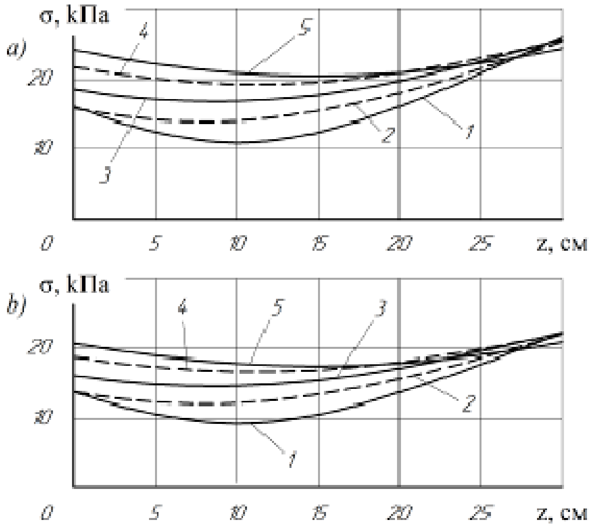

Figure 6 – The variation of the amplitudes of stresses σ, occurring in the concrete mixtures of different consistence at the distance of $x = 60$ cm (a) and $x = 80$ cm (b) from the source of vibration depending on the coordinate along the height of the vertical plate beginning from its vertical edge:
1 – at the slump 3.5 – 4 cm;
2 – at mixture hardness H = 30 s;
3 – at H = 60 s; 4 – at H = 90 s; 5 – at H = 120 s

Table 3 − The values of the required duration of the vibration process of compacting t_v concrete mixtures of different consistence at the vibration plate height $H = 30$ cm and the vibratory processing length $x = 60$ cm and $x = 80$ cm

Concrete mixture consistence	Slump= 3.5–4 cm	H= 30 s	H= 60 s	H= 90 s	H= 120 s
Compacting required time, s	$\frac{33}{39}$	$\frac{65}{76}$	$\frac{83}{96}$	$\frac{96}{111}$	$\frac{104}{122}$
Note 1. The numerator contains the data obtained at the vibratory processing length $x = 60$ cm, and the denominator contains the data obtained at $x = 80$ cm.					

Thus, it has been determined that the use of the plane depth vibration compactor, simultaneously accepting linear and torsion vibrations, provides the efficient compaction of concrete mixtures.

Conclusions

It has been proposed a fundamentally new design of a plane depth concrete mixture compactor made in the form of a vertical plate equipped with a circular vibration exciter in its upper part. It has been presented a design diagram of the «plane depth compactor – concrete mixture» dynamic system and determined the regularity of the movement of the vertical plate contacting with the compacted concrete mixture in the operating mode. The obtained analytical dependences enable to determine the efficient modes of the vibratory action on the compacted medium in the form of an alternating amplitude-frequency deformation of the compacted medium. The presented research results enable the substantiation of the rational parameters of the plane depth compactor, performing spatial vibrations, and the modes of vibratory action on the concrete mixtures of different consistence.

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Mathematical simulation of the motion law of differential mortar pump piston intended for construction mix

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The paper is dedicated to the creation of a differential mortar pump with electromagnetic action for pumping finishing material, which is not sensitive to electric energy gaps, and which is at the same time convenient, easy to use, reliable and economical in operation. The paper presents the mathematical model of the working process dynamics of a differential mortar pump with electromagnetic action, which will allow to study common patterns of pumping processes in the pump in the whole, to solve general problems on their calculation and design, to set and solve problems of reliability control, connected with high-frequency pressure oscillations, the problems of structural optimization and optimal design of all its elements. The control system of a pumping unit with vector controlled asynchronous electric drive is proposed on the basis of the concept of inverse dynamics problems in combination with the minimization of local functionality of instantaneous energy magnitudes, which ensures high-quality pressure regulation under the conditions of parametric perturbations activity and has acceptable energy indices.

Keywords: differential mortar pump with electromagnetic action, mathematical modeling, construction mix.

Математичне моделювання закону руху поршня диференціального насоса електромагнітної дії для будівельної суміші

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Роботу присвячено створенню диференціального насоса електромагнітної дії для перекачування оздоблювального матеріалу, який не чутливий до перепадів електроенергії, зручний і простий у використанні, надійний та економічний в експлуатації. Розглянуто результати математичного моделювання нестаціонарних процесів у насосному агрегаті з однопоршневим диференціальним насосом електромагнітної дії. Проаналізовано модель, що містить рівняння руху елементів системи, котрі враховують несталість зведеного моменту інерції штокового механізму насоса, а також електромагнітні явища в електромагнітній котушці. Запропоновано математичну модель динаміки робочого процесу диференціального насоса електромагнітної дії, яка дозволить досліджувати загальні закономірності перекачувальних процесів у насосі в цілому, розв'язувати загальні задачі з їх розрахунку і проектування, ставити і розв'язувати задачі забезпечення надійності, пов'язані з високочастотними коливаннями тиску, задачі оптимізації її структури й оптимального проектування всіх її елементів. Результати розв'язання диференціальних рівнянь математичної моделі, отримані у цій статті, можуть бути рекомендовано для практичної реалізації у вигляді аналітичних залежностей при розробленні методики розрахунку для створення нових конструкцій диференціальних насосів електромагнітної дії та оцінювання їх ефективності. Запропоновано систему керування насосною установкою з векторно-керованим асинхронним електроприводом на основі концепції зворотних задач динаміки в поєднанні з мінімізацією локальних функціоналів миттєвих значень енергії, яка забезпечує якісне регулювання напору в умовах дії параметричних збурень та має задовільні енергетичні показники.

Ключові слова: диференціальний насос електромагнітної дії, математичне моделювання, будівельна суміш

Introduction

Academic specialists have been researching mortar pumps from 1950-es onward. Notwithstanding that scientific works of that period remain fundamental, they do not contain complete analysis of differential mortar pumps owith electromagnetic action, but only describe their work and design in general.

The basis for the improvement of the effectiveness of a differential mortar pump with electromagnetic action is improvement of the power efficiency required to maintain constrained oscillations and immunity to electric energy gaps.

Review of research sources and publications

Creating efficient pumping equipment is a vexed problem for pumping production sphere, as over the past 10 years mostly obsolete technologies have been used in the field of pumping equipment manufacture. At present, the crucial task is to create a differential mortar pump with electromagnetic action [7 – 14], fit for effective work, as well as to produce a mathematical model [1 – 4], which, in its turn, would describe the overall operation of a differential mortar pump with electromagnetic action. The effective work is the ability of the pump to provide the maximum possible efficiency factor, which, in its turn, depends on the interaction between the coil and the plunger [5, 6], and, as a consequence – to support high performance reliability.

Thus, differential mortar pumps with electromagnetic action are one of the most common pump types, their constructional diversity is extremely high. Obtaining the required quality of a differential pump of electromagnetic action is a current problem, which is of great importance for the development of pumping production sphere.

Problem statement

In accordance with the abovementioned, the purpose of the article is to increase the running efficiency of a differential mortar pump with electromagnetic action. In order to achieve this goal, we have solved the following task: to create a mathematical model of the influence of electromagnetic induction on the uniformity of the construction mix pumping.

Basic material and results

Let’s consider the construction of a differential mortar pump with electromagnetic action for the construction mix, pictured in Fig. 1.

The mortar pump works as follows. Electric current that changes along the sinusoidal wave and induces magnetic induction onto the plunger, drawing it into the middle of the coil, enters into the coil 3.

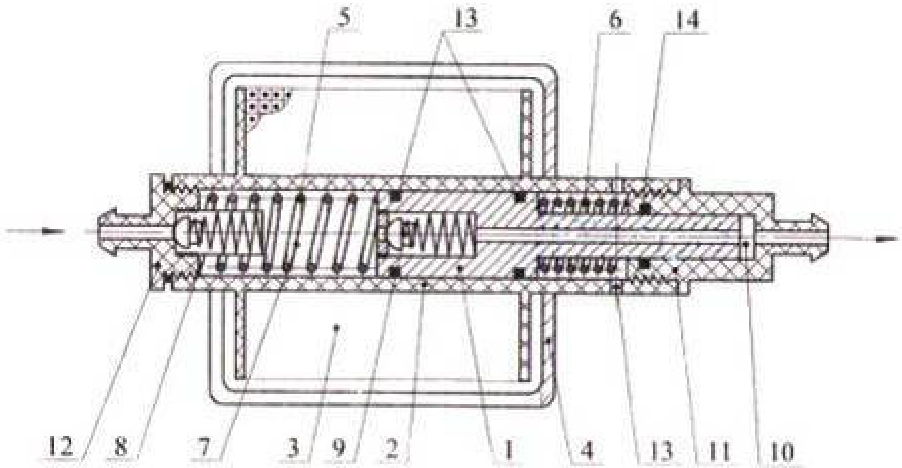

Figure 1 – The structure of the differential mortar pump with electromagnetic action for construction mix pumping:
1 – plunger, 2 – pump body; 3 – coil; 4 – coil flux guide; 5 – suction chamber; 6 – compensating spring; 7 – working spring; 8 – sniffle valve; 9 – discharge valve; 10 – compensating chamber; 11,12 – discharge fitting and suction fitting; 13,14 – lip-type seal

The first cycle of pumping. The plunger starts to move leftward, closing the sniffle valve (t1) and opening the discharge valve (t2). When the pump cavity is filled with mortar mix, the pumping process begins and the pressure in the discharge fitting starts to increase. The higher the motion speed of the plunger is, the more the pressure increases.

At the same time, the working spring 7 begins to shrink and the compensating spring 6 starts to straighten out. When the electric current in the coil falls, magnetic induction decreases and simultaneously the motion speed of the plunger declines until it stops. However, the plunger stops a little earlier before the complete shutdown of magnetic induction, when there occurs the balance moment of magnetic induction and compression force of the working spring 7. When the sinusoidal wave changes its direction, the diode in the power supply scheme cuts off its lower part, and in the second cycle, magnetic induction does not affect the plunger.

The second cycle of pumping. The working spring begins to straighten out, resulting in the opposite motion when the plunger is moving. The motion speed of the plunger begins to increase. When the plunger is moving to the right, the discharge valve 9 is closed and the pumping process is reactivated. Pumping pressure increases in proportion to the increase in path velocity.

At the same time, the sniffle valve 8 is opened and the working fluid is absorbed into the working cavity of the mortar pump. With the displacement of the plunger to the right, the working spring 7 compression is weakened. Straightening of the working spring is prevented by the pumping effort of the working fluid, the effort of absorbing the fluid into the working chamber and by the compensating spring compression. With the slackening of the working spring, the motion speed of the plunger decreases and coincidently the pumping process is reduced. By the moment the plunger stops, voltage is again applied into the coil, and the pumping process is rerun.

Let’s consider mortar pump operation separately for each cycle.

The first cycle of pumping. Plunger movement to the left. Since the plunger 1 moves progressively, then, making a mathematical model of its mechanical motion, we will consider it as a point particle, the mass of which is equal to the mass *m* of the plunger.

Let's consider the motion of a point particle to the left from its full distance right position, which is pictured in Fig. 2 and we will make a computational scheme of this motion, combining the coordinate origin *Oxy* with the initial position of the point particle and pointing the axis *Ox* toward the direction of particle motion. With such a choice of reference system

$x_0 = 0$ and $\dot{x}_0 = 0$,

where x_0 – the coordinate position that determines the position of the point particle at the time $t_0 = 0$,

$\dot{x}_0$ – the projection onto the axis *Ox* of the initial velocity $\vec{v}_0$ (of course, taking into account that v_0 =0, then $\dot{x}_0 = 0$).

a) b)

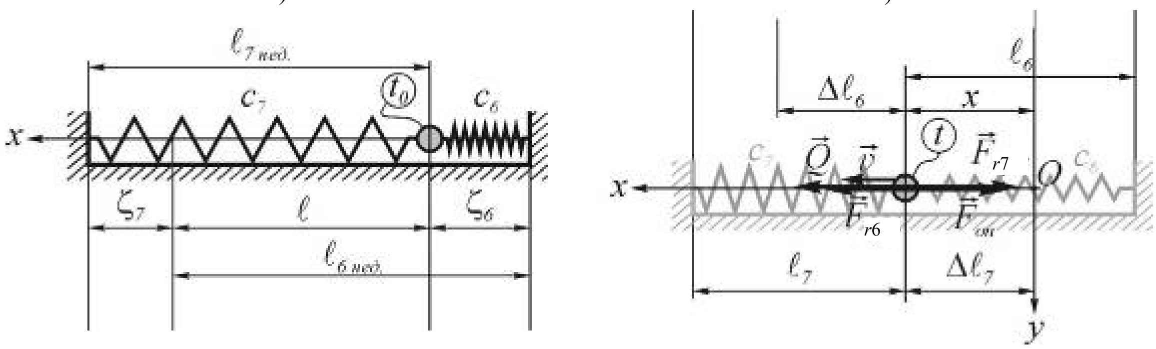

Figure 2 – Computational scheme for leftward plunger movement

Due to the fact that the motion of the point particle along the axis *Oy* is absent, then in accordance with the III-d Newton's law of motion

$\vec{G} = -\vec{N}$,

where $\vec{G} = m\vec{g}$ – gravity force, and $\vec{N}$ – normal response of the walls of the pump body.

Thus, forces $\vec{G}$ and $\vec{N}$ form a balanced system of forces $\{\vec{G}, \vec{N}\}$, which we shove aside on the basis of the corresponding axiom of statics, without breaking down the kinematic state of the point particle <u>under consideration</u>.

As a result, we have forces that have affect on the point particle (see figure 2, b):

– moving force $\vec{Q}$;

– elastic forces $\vec{F}_{r7}$ and $\vec{F}_{r6}$ of the compensating spring 6 and the working spring 7 accordingly;

– motion resistance force $\vec{F}_{on}$.

Let’s find out the meaning of these forces.

The module *Q* of the moving force changes sinusoidally similarly to the change of electric current by virtue of

$Q = Q_0 \cdot \sin pt$,

where Q_0 – the peak value of the moving force (needless to say, the dimension $[Q] = [Q_0] = MLT^{-2}$),

p – the cyclic frequency of the moving force, which is equal to the number of complete cycles of moving force variation per 2π seconds, and, of course, it is equal to the cyclic frequency of the electric current change.

In accordance with Hooke law, modules of elastic forces $\vec{F}_{r7}$ and $\vec{F}_{r6}$ of the compensating spring 6 and the working spring 7 determine the following dependencies

$F_{r7} = c_7 \cdot \Delta\ell_7$ and $F_{r6} = c_6 \cdot \Delta\ell_6$,

where c_7 and c_6 – the spring rate of the compensating spring 6 and the working spring 7

$\Delta\ell_7$, $\Delta\ell_6$ – their deformation in the position of the point particle at any given time t, pictured in Fig. 2.

The physical content of the coefficients c_7 and c_6 – the elastic force value of each spring under deformation, set to unity, and their dimensions are the ratio of the force dimension MLT^{-2} to the length dimension L, that is

$$[c_7]=[c_6]=\frac{MLT^{-2}}{L}=M\cdot T^{-2}.$$

From Fig. 2, b it is obvious that

$$\Delta\ell_7 = x,$$

$$\Delta\ell_6 = \ell_{6нед.} - \ell_6 = \ell_{6нед.} - (x+\varsigma_6) = \ell_{6нед.} - x - \varsigma_6,$$

where x – the coordinate, which determines the position of the point particle at the time t,

$\ell_{6нед.}$ – the length of the undeformed spring 6 ℓ_6 – the length of this spring at the time t

ς_6 – the length of this spring in the initial position of the point particle (at the time $t_0 = 0$).

Taking into account the set values $\Delta\ell_7$ and $\Delta\ell_6$, we will get

$$F_{r7} = c_7\cdot x,$$

$$F_{r6} = c_6\cdot(\ell_{6нед.} - x - \varsigma_6).$$

Since the plunger 1 in the stationary medium – in mortar mix, which fills mortar pump's working cavity, performs progressive motion at low speed, then the motion resistance force will be in opposition to the direction of the plunger velocity vector $\vec{v}$ and in vector form it can be written as:

$$\vec{F}_{on} = -F_{on}\cdot\frac{\vec{v}}{v},$$

where F_{on} – the absolute value (module) of this force;
v – plunger velocity module 1.

With the help of dimensional method [1] we will define the module value F_{on} of mortar mix resistance force. In initial approximation we will assume that this force, which dimension is $[F_{on}] = MLT^{-2}$, is determined by the following parameters, which, of course, are physical values:
v – the plunger motion speed, the dimension of which is $[v] = LT^{-1}$;
S – the area of the plunger pressure on the operating environment (mortar mix), the dimension of which is $[S] = L^2$;
μ – the absolute viscosity coefficient, the dimension of which is $[\mu] = ML^{-1}T^{-1}$.

According to [2], we will seek functional relationship $F_{on} = f(v,S,\mu)$ in the form:

$$F_{on} = k\cdot v^a\cdot S^b\cdot\mu^c, \qquad (1)$$

where k – some nondimensional coefficient (viz $[k]=1$), which cannot be determined by use of dimensional analysis.

According to the theory of dimensional analysis, between dimensions $[F_{on}]$, $[v]$, $[S]$ and $[\mu]$ there must be functional connection similar to the association between physical values F_{on}, v, S and μ, which is determined by formula (1). From this, we have that

$$[F_{on}]=[k]\cdot[v]^a\cdot[S]^b\cdot[\mu]^c,$$

or, taking into account the above-mentioned dimensions,

$$MLT^{-2} = 1\cdot(LT^{-1})^a\cdot(L^2)^b\cdot(ML^{-1}T^{-1})^c.$$

Having completed the obvious transformations of the right side, we will obtain

$$MLT^{-2} = L^{a+2b-c}\cdot T^{-a-c}\cdot M^c.$$

Since the mathematically obtained dependence can be fulfilled only if power coefficients of the corresponding multiplicators are equal, then, making the specified indicators equal, we will obtain the system of algebraic equations:

Having solved this system of algebraic equations, we will find that

$$c=1,\ a=2-c=2-1=1$$

and

$$b=\frac{1-a+c}{2}=\frac{1-1+1}{2}=\frac{1}{2}.$$

Then the desired dependence (1) will have the form

$$F_{on} = k\cdot v^1\cdot S^{\frac{1}{2}}\cdot\mu^1$$

or

$$F_{on} = k\cdot V\cdot\mu\cdot\sqrt{S}.$$

From Fig. 1 it is clear that when the plunger 1 moves to the left, the area S of its pressure on the operating environment (mortar mix) will be determined by the formula

$$S=\frac{\pi\cdot d_1^2}{4},$$

where d_1 – the diameter of the plunger in the working chamber (see Fig. 3).

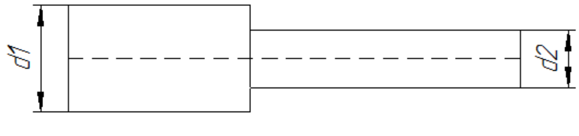

Figure 3 – Diagrammatic representation of the plunger 1

If we take $d_{от.} = \gamma\cdot d_1$, where γ – a certain "diameter reduction factor" (of course, that $\gamma<1$), then

$$S=\frac{\pi}{4}\cdot(d_1^2-\gamma^2\cdot d_1^2)=\frac{\pi\cdot d_1^2}{4}\cdot(1-\gamma^2).$$

Then finally we will get that

$$F_{on} = k\cdot v\cdot \mu \cdot \sqrt{\frac{\pi \cdot d_1^2}{4}\cdot \left(1-\gamma^2\right)} =$$
$$= \frac{k\cdot v\cdot \mu\cdot d_1\cdot \sqrt{\pi\cdot\left(1-\gamma^2\right)}}{2}$$

or

$$F_{on} = k\cdot \frac{\sqrt{\pi\cdot\left(1-\gamma^2\right)}}{2}\cdot \mu\cdot d_1\cdot v .$$

In the general case, the coefficient k in the resistance force formula for the resistance F_{on} of the medium is in the functional relationship to the Reynolds number (Reynolds criterion) *Re* and to the Froude number F_{on} that is

$k = f(Re, Fr)$.

According to [3], «the mathematical relation $k = f(Re, Fr)$ is complex and its extremely difficult to obtain it theoretically. It is common practice to use experimentally obtained values of the coefficient k». But in the case under consideration, when the distance and motion speed of the point particle are insignificant, we will neglect the dependence $k = f(Re, Fr)$, assuming that $k = const$.

Following the algorithm of solving the inverse primal dynamic problem of the point particle [4], we will compose the differential equation of motion of the point particle under the question.

We will record Newton's second law of motion (second principle of dynamics) in the projection on the axis *Ox*.

From the computational scheme (see Fig. 2, b) it is obvious that

$$\sum_{i=1}^{\lambda} F_{ix} = Q + F_{r6} - F_{r7} - F_{on} \tag{2}$$

or, taking into consideration the found force values,

$$\sum_{i=1}^{\lambda} F_{ix} = Q_0\cdot\sin\frac{\pi\cdot t}{\tau} + c_6\cdot\left(\ell_{6\,нед.} - x - \varsigma_6\right) -$$
$$- c_7\cdot x - k\cdot\frac{\sqrt{\pi\cdot\left(1-\gamma^2\right)}}{2}\cdot\mu\cdot d_1\cdot v.$$

Then

$$\sum_{i=1}^{\lambda} F_{ix} = Q_0\cdot\sin\frac{\pi\cdot t}{\tau} + c_6\cdot\left(\ell_{6\,нед.} - x - \varsigma_6\right) -$$
$$- c_7\cdot x - k\cdot\frac{\sqrt{\pi\cdot\left(1-\gamma^2\right)}}{2}\cdot\mu\cdot d_1\cdot v =$$
$$= Q_0\cdot\sin\frac{\pi\cdot t}{\tau} - c_6\cdot x + c_6\cdot\left(\ell_{6\,нед.} - \varsigma_6\right) -$$
$$- c_7\cdot x - k\cdot\frac{\sqrt{\pi\cdot\left(1-\gamma^2\right)}}{2}\cdot\mu\cdot d_1\cdot v =$$
$$= Q_0\cdot\sin\frac{\pi\cdot t}{\tau} - \left(c_6 + c_7\right)\cdot x + c_6\cdot\left(\ell_{6\,нед.} - \varsigma_6\right) -$$
$$- k\cdot\frac{\sqrt{\pi\cdot\left(1-\gamma^2\right)}}{2}\cdot\mu\cdot d_1\cdot v.$$

Substituting the last expression and the value $a_x = \frac{dv}{dt}$ into formula (2), we will obtain the differential equation of point particle motion in the form

$$m\cdot\frac{dv}{dt} = Q_0\cdot\sin\frac{\pi\cdot t}{\tau} - \left(c_6+c_7\right)\cdot x +$$
$$+ c_6\cdot\left(\ell_{6\,нед.} - \varsigma_6\right) - k\cdot\frac{\sqrt{\pi\left(1-\gamma^2\right)}}{2}\cdot\mu\cdot d_1\cdot v ,$$

and, dividing both parts of it into m and carrying out legitimate transformations, we will get

$$\frac{dv}{dt} = \frac{Q_0}{m}\cdot\sin\frac{\pi\cdot t}{\tau} - \frac{\left(c_6+c_7\right)}{m}\cdot x +$$
$$+ \frac{c_6\cdot\left(\ell_{6\,нед.} - \varsigma_6\right)}{m} - k\cdot\frac{\sqrt{\pi\cdot\left(1-\gamma^2\right)}}{2\cdot m}\cdot\mu\cdot d_1\cdot v$$

and

$$\frac{dv}{dt} + k\cdot\frac{\sqrt{\pi\cdot\left(1-\gamma^2\right)}}{2\cdot m}\cdot\mu\cdot d_1\cdot v + \frac{c_6+c_7}{m}\cdot x =$$
$$= \frac{Q_0}{m}\cdot\sin\frac{\pi\cdot t}{\tau} + \frac{c_6\cdot\left(\ell_{6\,нед.} - \varsigma_6\right)}{m}. \tag{3}$$

The deduced equation (3) is the differential equation of plunger movement 1 to the left in the canonical form or the mathematical model of this mechanical motion.

If we introduce the designations, traditional for the theory of oscillations [5]

$$k\cdot\frac{\sqrt{\pi\cdot\left(1-\gamma^2\right)}}{2\cdot m}\cdot\mu\cdot d_1 = 2\cdot n_l ,$$

$$\frac{c_6+c_7}{m} = k^2 , \qquad \frac{Q_0}{m} = h$$

and assume by convention that

$$\frac{c_6\cdot\left(\ell_{6\,нед.} - \varsigma_6\right)}{m} = C_1 = const ,$$

then the differential equation of plunger movement 1 to the left can be given more compact form

$$\frac{dv}{dt} + 2\cdot n_l\cdot v + k^2\cdot x = h\cdot\sin\frac{\pi\cdot t}{\tau} + C_1 .$$

Let's determine dimensions of physical values n_l, k, h and C_1. In accordance with the foregoing formula

$$n_l = k\cdot\frac{\sqrt{\pi\cdot\left(1-\gamma^2\right)}}{4\cdot m}\cdot\mu\cdot d_1 ,$$

$$k = \sqrt{\frac{c_6+c_7}{m}} .$$

Then

$$\left[n_{лів.}\right] = \left[k\right]\cdot\frac{1}{\left[m\right]}\cdot\left[\mu\right]\cdot\left[d_1\right] = 1\cdot\frac{1}{M}\cdot ML^{-1}T^{-1}\cdot L =$$
$$= \frac{M\cdot L}{M\cdot L\cdot T} = \frac{1}{T} = T^{-1} ,$$

$$[k]=\sqrt{\frac{[c_6]+[c_7]}{[m]}}=\sqrt{\frac{M\cdot T^{-2}+M\cdot T^{-2}}{M}}= \\ =\sqrt{\frac{M\cdot T^{-2}}{M}}=\sqrt{T^{-2}}=T^{-1},$$

$$[h]=\frac{[Q_0]}{[m]}=\frac{MLT^{-2}}{M}=L\cdot T^{-2},$$

$$[C_1]=\frac{[c_6]\cdot([\ell_{6\,нед.}]-[\varsigma_6])}{[m]}=\frac{MT^{-2}\cdot(L-L)}{M}= \\ =\frac{MT^{-2}\cdot L}{M}=L\cdot T^{-2}.$$

Note that the values k and n_l have the same dimensions, which allows, if necessary, to compare these values. Both values h and C_1, as it must be, have identical dimensions.

In the theory of oscillations according to [5] physical values n_l, k and h have the following mechanical intensions:

n_l – the attenuation coefficient, which characterizes the resistance of medium at low motion speed of the point particle;

k – cyclic/circular (radian) frequency of eigenvibrations (free oscillations) of the point particle on the spring with stiffness coefficient $c_{ecv}=c_6+c_7$;

h – the largest value of the summand, which determines the maximum acceleration of the motion of the point particle $a_{max}=h+C_1$ under investigation.

Let's solve the equation (4) and construct acceleration profile of the plunger to the left using the free mathematical program "SMath Studio" taking into account the initial conditions:

$$\begin{cases} v_{(0)}=0; \\ P_{(0)}=P_{\min}; \\ x_{(0)}=0. \end{cases}$$

The constructed graph is shown in Fig. 4. The received peak speed of the plunger, taking into account given geometrical dimensions of mortar pump component parts, proceeding from the diameter of the working chamber section (plunger diameter) 25 mm at the productivity of 0,25 m^3/h is 8,63 m/s.

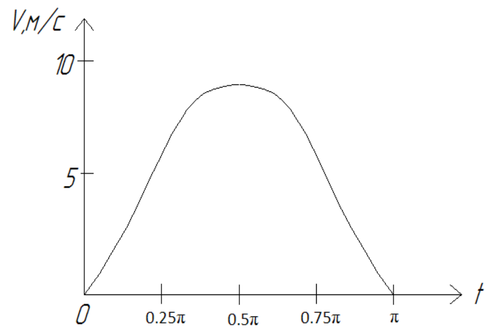

Figure 4 – The graph of plunger speed variation in time when it moves to the left

The second cycle of pumping. Plunger movement to the right. Now let us consider the motion of the point particle to the right from its full distance left position, which is pictured in Figure 2, a, and in Fig. 2, b and we will make a computational scheme of this motion, by choosing the system of coordinates Oxy from the conditions similar to those of the point particle motion on the left. When choosing such a frame of reference, the initial conditions of motion will be:

$x_0=0$ and $\dot{x}_0=0$,

and forces $\vec{F}_{r7}$, $\vec{F}_{r6}$ and $\vec{F}_{on}$ .affect the point particle (see Fig. 4, b)

In accordance with Hooke law, $F_{r7}=c_7\cdot\Delta\ell_7$ and $F_{r6}=c_6\cdot\Delta\ell_6$,

From Figure 2, b it is obvious that

$$\Delta\ell_6=x,$$

$$\Delta\ell_7=\ell_{7\,нед.}-\ell_7=\ell_{7\,нед.}-(\varsigma_7+x)= \\ =\ell_{7\,нед.}-\varsigma_7-x,$$

where x – the coordinate that determines the point particle position at the time t, $\ell_{7\,нед.}$ – the length of the undeformed spring 7, ℓ_7 – the length of this spring at the time t and ς_7 – the length of this spring in the initial position of the point particle (at the time $t_0=0$).

Taking into account the set values $\Delta\ell_7$ and $\Delta\ell_6$, we will get

$$F_{r7}=c_7\cdot(\ell_{7\,нед.}-\varsigma_7-x),$$

$$F_{r6}=c_6\cdot x.$$

The module of the resistance force F_{on} of the mortar mix to the point particle motion is again determined from the formula

$$F_{on}=k\cdot v\cdot\mu\cdot\sqrt{S},$$

and from Fig. 3 it is clear that when the plunger 1 moves to the right, its area of pressure on the working medium (solution) will be as follows

$$S=\frac{\pi\cdot d_2^2}{4}.$$

Then in this case

$$F_{on}=k\cdot v\cdot\mu\cdot\sqrt{\frac{\pi\cdot d_2^2}{4}}=\frac{k\cdot v\cdot\mu\cdot d_2\cdot\sqrt{\pi}}{2}$$

or

$$F_{on}=k\cdot\frac{\sqrt{\pi}}{2}\cdot\mu\cdot d_2\cdot v.$$

From the computational scheme (see Fig. 2, b), we can define, that in this case

$$\sum_{i=1}^{\lambda} F_{ix} = F_{r7} - F_{r6} - F_{on} , \qquad (4)$$

or, taking into account the set force values,

$$\sum_{i=1}^{\lambda} F_{ix} = c_7 \cdot \left(\ell_{7\,нед.} - \varsigma_7 - x\right) - c_6 \cdot x - k \cdot \frac{\sqrt{\pi}}{2} \cdot \mu \cdot d_2 \cdot v =$$

$$= c_7 \cdot \left(\ell_{7\,нед.} - \varsigma_7\right) - c_7 \cdot x - c_6 \cdot x - k \cdot \frac{\sqrt{\pi}}{2} \cdot \mu \cdot d_2 \cdot v =$$

$$= c_7 \cdot \left(\ell_{7\,нед.} - \varsigma_7\right) - \left(c_7 + c_6\right) \cdot x - k \cdot \frac{\sqrt{\pi}}{2} \cdot \mu \cdot d_2 \cdot v .$$

Substituting the last expression and the value $a_x = \frac{dv}{dt}$ into formula (4), we will obtain the differential equation of point particle motion in the form

$$m \cdot \frac{dv}{dt} = c_7 \cdot \left(\ell_{7\,нед.} - \varsigma_7\right) - \left(c_7 + c_6\right) \cdot x - k \cdot \frac{\sqrt{\pi}}{2} \cdot \mu \cdot d_2 \cdot v ,$$

and, dividing both parts of it into m and carrying out legitimate transformations, we will get

$$\frac{dv}{dt} = \frac{c_7 \cdot \left(\ell_{7\,нед.} - \varsigma_7\right)}{m} - \frac{\left(c_7 + c_6\right)}{m} \cdot x - k \cdot \frac{\sqrt{\pi}}{2 \cdot m} \cdot \mu \cdot d_2 \cdot v$$

and

$$\frac{dv}{dt} + k \cdot \frac{\sqrt{\pi}}{2 \cdot m} \cdot \mu \cdot d_2 \cdot v + \frac{c_7 + c_6}{m} \cdot x = \frac{c_7 \cdot \left(\ell_{7\,нед.} - \varsigma_7\right)}{m} . \qquad (5)$$

The deduced equation (5) is the differential equation of plunger movement 1 to the right in the canonical form or the mathematical model of this mechanical motion.

Again, if we introduce the designations, traditional for the theory of oscillations, we will get:

$$k \cdot \frac{\sqrt{\pi}}{2 \cdot m} \cdot \mu \cdot d_2 = 2 \cdot n_r \; ;$$

$$\frac{c_7 + c_6}{m} = k^2$$

and assume by convention that

$$\frac{c_7 \cdot \left(\ell_{7\,нед.} - \varsigma_7\right)}{m} = C_2 = const ,$$

then the above-obtained differential equation of plunger movement 1 to the right can be given the following form

$$\frac{dv}{dt} + 2 \cdot n_{пр.} \cdot v + k^2 \cdot x = C_2 ,$$

in which the dimensions of physical values n_r, k and C_2 as well as their mechanical intensions, are undoubtedly similar to the foregoing.

Let's solve the equation (2) and construct acceleration profile of the plunger to the right at the same axis of reference with acceleration profile of the plunger to the left using the free mathematical program "SMath Studio" taking into account the initial conditions:

$$\begin{cases} v_{(0)} = 0; \\ P_{(0)} = P_{\min};, \\ x_{(0)} = 0. \end{cases}$$

The constructed graph is shown in Fig. 5 and displays the graph of temporal variations for the plunger speed in the complete cycle. The received peak speed of the plunger movement to the right, taking into account given geometrical dimensions of mortar pump component parts, proceeding from the diameter of the working chamber section (plunger diameter) 25 mm at the productivity of 0,25 m^3/h is 9,33 m/s.

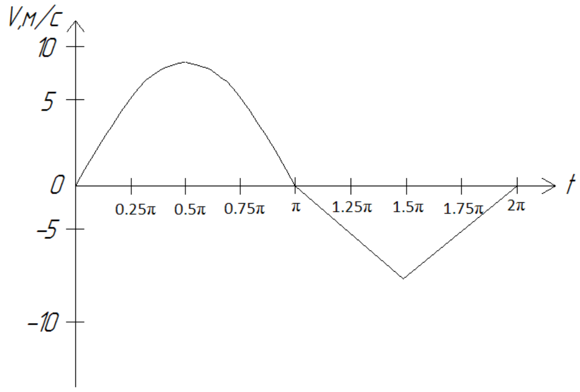

Figure 5 – The graph of temporal variations for the plunger speed in the complete cycle of plunger movement

Conclusions

1. We have obtained mathematical models of differential equations, which reflect velocity history (variations in time of the velocity) of a differential pump piston of a mortar pump used for pumping of construction mixes in the complete cycle of its movement.
2. The analysis of the obtained mathematical models allows to optimize the geometric dimensions of mortar pump component parts, including the geometric dimensions of the springs in order to ensure the mechanical energy conservation during pumping.
3. The graph of temporal variations for the plunger speed in the complete cycle of plunger movement allows us to simulate the required plunger productivity when pumping mortar mixes.

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Mathematical modeling for the technological process of surface soil compaction by the inertial vibratory rammer

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The high efficiency of the technological process of surface soil compaction using vibration and vibro-impact treatment has been determined. High degree intensification of the soil compaction process is achieved by using original inertial vibratory rammers with a hydro-pulse drive. A new mathematical model has been developed for the surface soil compaction processes study by the inertial vibratory rammers. Using numerical modeling, work dependencies are obtained to determine the main operating characteristics for the technological process of surface soil compaction by inertial vibratory rammers based on a hydro-impulse drive.

Keywords: rammer, compaction, shock, vibration, hydro-impulse drive, inertia, soil, valve.

Математичне моделювання технологічного процесу поверхневого ущільнення ґрунтів інерційною вібротрамбівкою

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Визначено високу ефективність технологічного процесу поверхневого ущільнення ґрунтів за допомогою вібраційної і віброударної обробки. Високого ступеню інтенсифікації процесу ущільнення ґрунтів досягнуто застосуванням розроблених оригінальних інерційних вібраційних трамбівок з гідроімпульсним приводом на базі двокаскадного клапана-пульсатора. Розроблено нову математичну модель для дослідження технологічних процесів поверхневого ущільнення ґрунтів інерційною вібротрамбівкою на базі законів гідродинаміки з використанням механореологічної феноменології й узагальнених законів механіки. При розробленні математичної моделі технологічний процес було досліджено на двох фазах: фаза накопичення кінетичної енергії, а також фаза ударної взаємодії інерційної трамбівки з поверхнею ґрунту. На основі розробленої математичної моделі методом кінцевих об’ємів за допомогою чисельного моделювання й методу припасовування отримано розподіл тиску і швидкості робочої рідини в гідроімпульсному приводі інерційної вібротрамбівки. Також було отримано діаграми зміни відносної й абсолютної швидкості рухомих елементів інерційної вібротрамбівки. Розроблена оригінальна модель поверхневого ущільнення ґрунтів дозволила одержати залежності зміни переміщення центрів мас шарів ущільнюючого ґрунту типу суглинків. Одержані робочі залежності основних робочих характеристик інерційної трамбівки на базі гідроімпульсного привода дозволили отримати основні робочі залежності для подальшого підвищення ефективності технологічного процесу поверхневого ущільнення ґрунтів. Отримані результати чисельного моделювання технологічних процесів поверхневого ущільнення ґрунтів інерційною вібротрамбівкою на базі гідроімпульсного привода показали переваги обраного підходу до проектування, а також дозволили довести ефективність розробленої конструкції.

Ключові слова: трамбівка, ущільнення, удар, вібрації, гідроімпульсний привід, інерція, ґрунт, клапан.

Introduction

Technologies characterized by surface treatment processes where pseudo-fluidity of materials is realized [1, 2] with complex rheology and under inertial load conditions, require new developments, study and improvement. Vibrational and vibro-impact technological processes, as well as equipment for their implementation, are widely used [1]. It has been established that if useful vibrations or shock impulses to processing objects are applied, then it can be the flow of a number of technological processes can be significantly intensified, optimal load parameters can be ensured and processing results with high quality parameters can be obtained. In particular, during mechanical surface compaction (ramming), which consists of impacting the load on the soil that falls freely or at a forcibly developed speed, extreme compressive stresses appear in the soil, which cause pseudo-flowability and reorientation of the dispersed soil particles that leads to compaction soil.

Therefore, the study of the changes in the working influence and design parameters of inertial vibratory rammers from a hydro-impulse drive (HID) [1] on the flow of work processes in the surface soil compaction technology increase its effectiveness and outline ways for further development and improvement.

Currently, it is widely used mathematical modeling of workflows in various technological devices. With its help, it is possible to deeply and fully investigate the influence of structural and operational factors on the main characteristics of the device and outline specific ways to improve them, while significantly reducing the amount of experimental research.

Review of research sources and publications

In [2, 3], on the basis of experimental studies, it was found that the best compaction of soils is usually achieved at values of amplitude vibration accelerations close to the acceleration of gravity. Under the action of attractive forces, inertial forces and static load axial forces, particles of a dispersed material tend to reorient and more densely fit each other in a given volume, accompanied by the occupation of more stable equilibrium positions. At the same time, the destruction of the initial structural formations of the «arches» and «bridges» types is observed, followed by a uniform conclusion of the particles forming them, which have an increased mobility in the direction of the static load force by reducing the effective coefficients of internal and lateral friction to zero. In [4, 5] there is researched a fluctuation model particle dispersion medium under the action of vibrations. The phenomena of «vibro-boiling» were analyzed, when a particle loses contact with a vibrating working body, and the bonds between particles decrease and are periodically broken. This state is characterized by loosening of the medium and enhanced circulation of the particles that make it up. The disadvantage of these scientific works is the lack of a mathematically-based behavior of dispersed materials under the action of vibration and vibro-impact load.

In the scientific work [6], a systematic approach to the technological process of forming vibration (VM) and vibro-impact machines (VIM) based on the HID was considered. It enabled to develop a mathematical model for its evaluation. Relationships between the parameters of subsystems on the basis of a vibropress equipment with a HID for forming blanks from powder materials are determined. Based on the mathematical model of fuzzy sets was estimated efficiency of the processing facility. But this technique does not enable to investigate the physicomechanical processes occurring directly in the equipment of the VM and VIM on the basis of the HID.

Definition of unsolved aspects of the problem

Study of physico-mechanical processes that occur in the HID based on the calculations of motion's equations and expenses with a computer [6]. This method does not enable to evaluate the influence of hydrodynamic processes occurring in the executive and regulating units of the HID, despite the complexity of the calculations and the assumptions made in the mathematical description of the working process, which, as experimental data accumulate, can be refined.

Solving these problems is impossible without using the Navier-Stokes equation, which requires the use of the finite volume method to study the complex motion of the working fluid under different flow conditions [5]. Conducting such studies is based on modern methods of mathematical modeling with calculations on a computer using modern, advanced algorithms. It enables to prevent an unjustifiably large number of complex and expensive experimental studies, significantly reduce the time and cost of design work, conduct qualitative and quantitative assessments of physical phenomena with sufficient accuracy for engineering practice [1].

Therefore, the construction of a mathematical model that enables to describe the behavior of a dispersed medium under vibration and vibro-impact load under various operating modes of a HID on the inertial vibratory rammer with the aim of determining the basic performance characteristics of technological processing of building materials is an urgent task.

Problem statement

The aim of the work is to increase the efficiency of theoretical research of the surface soil treatment technology through the development of promising mathematical models for the physical processes of inertial vibration compaction using HID-based devices.

To achieve this goal the following tasks were solved:

– to develop an effective inertial inertial vibratory rammer with HID based on a two-stage pulsator valve. This will allow to realize the most effective modes of vibration impact on the processed media.

– develop the mathematical model for the technology of surface compaction of soils with inertial vibratory rammer based on the HID with a two-stage pulsator valve;

– on the basis of the developed mathematical model, to obtain working dependencies to determine the main characteristics of the studied technological process.

Basic material and results

In Vinnytsia National Technical University, the Department of Industrial Engineering has developed inertial vibratory rammer based on the HID [7]. Figure 1 shows the three-dimensional CAD-model of inertial vibratory rammer based on the HID.

This installation consists of the following main structural elements: to tamping plate 1, through the outer walls 2, attached to the cover 3. The cover 3 is attached to the tamping plate 1 guides 4.

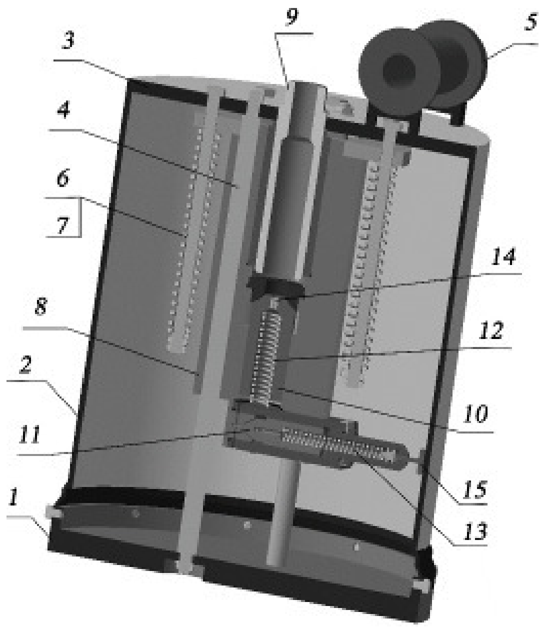


Figure 1 – Three-dimensional model of inertial vibratory rammer based on the HID

In turn, a fastening assembly 5 and a body of a two-stage pulsator valve 8 connected through a power springs 6 and power rails 7 are attached to the cover 3. The housing of the two-stage valve-pulsator 8 additionally performs the functions of a movable hydraulic cylinder in a fixed piston-guide 9. In the case of a two-stage pulsation valve 8 there is a valve of the second cascade 10 and a valve of the first cascade 11. Their mutual movement is regulated by springs 12 and 13, respectively, by the throttle 14 and the adjusting screw 15.

Figure 2 shows the three-component (flat multi-mass) inertial model with irresistible contacts between the masses. It enables to simulate the elastic-viscous properties of soil layers 1, 2, 3 along the x and y axes (the nature of the medium movement along the x and z axes is equivalent, therefore it is advisable to consider only the motion along the x, y axes).

Studies [8] show that oscillations from the working body 4 (see Fig. 2) (ramming plate 1 (see Fig. 1)) inertial vibratory rammer with a mass M are transmitted to the upper layer of soil 1 touching it, with a concentrated mass m_1, and further below are located layers of soil 2, 3, with concentrated masses m_2, m_3. The frequency of oscillations of the entire mass of the soil layer 1 in the zone of vibrations is the same. And the amplitude decreases with distance from the surface of the working body until it goes out altogether.

Thus, a characteristic feature of the operation of inertial vibratory rammers is that the soil is subjected to the vibration effect in a limited area of the working body.

The dimensions of this zone are determined by the oscillation mode and the location of the inertial rammer itself, as well as the properties of the soil itself.

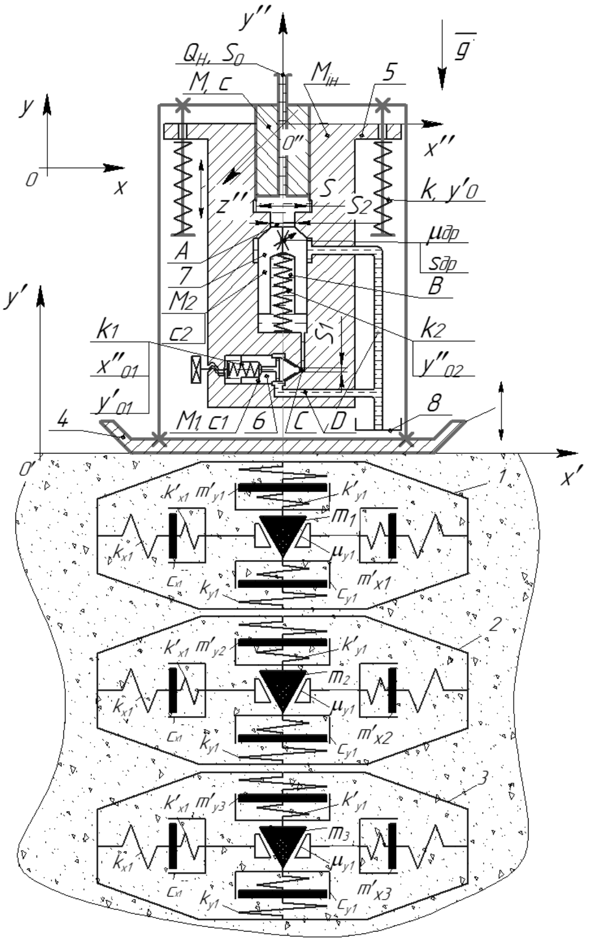


Figure 2 – Dynamic model for the technological process of surface compaction of soil by the inertial vibratory rammer with the HID

There are introduced two moving coordinate systems – $x'O'y'$, rigidly connected to the working body 4, and $x''O''y''$, rigidly connected to the hydraulic cylinder 5 (see Fig. 2) (the body of the two-stage pulsator valve 8 (see Fig. 1)). And also there is introduced the absolute (fixed) coordinate system xOy.

The system of equations (1) for the law of the movement of the soil layers 1, 2, 3 during the contact interaction of the working body 4 with the upper layer of soil 1 is shown in this system of equations:

y_1 – is the absolute coordinate of the working body position4;

y'_1, y'_2, y'_3, x'_1, x'_2, x'_3 – coordinates of the soil masses layers 1, 2, 3;

$k_y = k_{y1}\, k'_{y1} / (k_{y1} + k'_{y1})$, $k_x = k_{x1}\, k'_{x1} / (k_{x1} + k'_{x1})$ – equivalent stiffness of soil layers 1, 2, 3, respectively;

μ_x, μ_y – the projections of the internal friction coefficients on the x and y axes, respectively;

c_{x1}, c_{y1} – component coefficients of viscosity along the x and y axes respectively;

N_{1xy}, N_{2xy}, N_{3xy} – internal reaction forces within the concentrated masses m_1, m_2, m_3 on wedge-shaped platforms at an angle of 45^0;

$m'_{x1} = m'_{y1} = 0$ – additional rheological masses;

$N_{1,4}(t) = k_y y_1' + c_{y1}\dot{y}'_1$ – normal reaction of soil layer 1 to load carrier 4.

$$\begin{cases} -m_1(\ddot{y}_1 + \ddot{y}_1') = -N_{1,4}(t) + k_y(y_1' - y_2')_1 + \\ +c_{y1}(\dot{y}_1' - \dot{y}_2') - m_1 g + 2sign(\dot{y}_1')\mu_y N_{1xy}; \\ -m_2(\ddot{y}_1 + \ddot{y}_2') = k_y(y_2' - y_1') + k_y(y_2' - y_3') + \\ +c_{y1}\left(\dot{y}_2' - \dot{y}_1'\right) + c_{y1}(\dot{y}_2' - \dot{y}_3') - m_2 g + \\ +2sign(\dot{y}_2')\mu_y N_{2xy}; \\ -m_3(\ddot{y}_1 + \ddot{y}_3') = k_y(y_3' - y_2') + k_y y_3' + \\ +c_{y1}\left(\dot{y}_3' - \dot{y}_2'\right) + c_{y1}\dot{y}_3' - m_3 g + 2sign(\dot{y}_3')\mu_y N_{3xy}; \\ m_1\ddot{x}_1' = -k_x x_1' - c_{x1} x_1' + sign(\dot{y}_1')\mu_x N_{1xy}; \\ m_2\ddot{x}_2' = -k_x x_2' - c_{x1} x_2' + sign(\dot{y}_2')\mu_x N_{2xy}; \\ m_3\ddot{x}_3' = -k_x x_3' - c_{x1} x_3' + sign(\dot{y}_3')\mu_x N_{3xy}; \\ y_1' = x_1';\ y_2' = x_2';\ y_3' = x_3'. \end{cases} \quad (1)$$

The law of the movement for the working body 4:

$$-M\ddot{y} = -Mg + N_{1,4}(t) - \delta k\left(\left(L - y'\right) + y_0'\right) + N_{45}, \quad (2)$$

where M – the mass of the working body 4;
δ – the number of return springs with the rigidity k with the initial tension y_0';
L – internal height of the inertial ramp;
N_{45} – is the force of the gan working reaction 4 from the contact interaction with the inertia mass 5.

The law of motion for the inertial mass $M_{ін}$ is:

$$-M_{ін}\left(\ddot{y}' + \ddot{y}\right) = -M_{ін} g + \delta k(\left(L - y'\right) + y_0') - $$
$$-\iint_{\bar{S}} p_{\bar{S}}(t)dS + c\dot{y}' + N_{2y} + N_{1y} + c_2\dot{y}_2'' - \quad (3)$$
$$-k_2(|y''|_2 + y_{02}'') - N_{45},$$

where $p_S(t)$ – is the changing the pressure function of the working fluid in the internal cavity of a two-stage pulsator valve,

$\iint_{\bar{S}} p_{\bar{S}}(t)dS$ – forces acting on the inner surface S' of the inertial mass $M_{ін}$,

c_2 – is the coefficient of viscous friction between the walls of the inertial mass 5 and the piston of the working body 4 (see Fig. 2),

N_{2y} –is the vertical component of the reaction force of the conical support from the conical valve of the second stage 7,

N_{1y} – is the vertical component of the reaction force of the cylindrical and conical surfaces valve of the first cascade 6,

c_2 –is the coefficient of viscous friction between the walls of the inertial mass 5 and the valve of the other cascade (see Fig. 2),

k_2 –is the spring stiffness of the valve of the second cascade,

k_2 – is the preload of the valve spring of the second cascade.

Since the valve of the first cascade 6 along the y axis always moves together with the inertial mass 5, it is therefore advisable to record the law of motion of the valve of the first cascade with mass M_1 only for the x axis:

$$-M_1\ddot{x}_1'' = -N_{1x} + k_1(|\ddot{x}_1| + x_{01}'') - \iint_{S'} p_{S'}(t)dS + c_1\dot{x}'', \quad (4)$$

where $p_{S'}(t)$ – the function of pressure change of the working fluid in the internal cavities C and D of the two-stage pulsator valve;

$\iint_{S'} p_{S'}(t)dS$ – forces acting on the inner surface of the inertial mass S';

c_1 – is the coefficient of viscous friction forces between the walls of the inertial mass 5 and the piston of the working body 4 (see Fig. 2);

N_{1x} – is the reaction force of the conical support from the conical valve 7, moreover $N_{1x} = |N_{1y}|$, since the conical surface is made at an angle of 45^0 to the y axis.

The law of movement for the second cascade valve of mass M_2:

$$-M_2(\ddot{y}_2'' + \ddot{y}'' + \ddot{y}) = -M_2 g - \iint_{S''} p_{S''}(t)dS - N_{2y} - $$
$$-c_2\dot{y}_2'' + k_2(|y''|_2 + y_{02}''), \quad (5)$$

where $p_{S''}(t)$ – is the function of changing the pressure of the working fluid in the internal cavities A and B of the two-stage valve-pulsator;

$\iint_{S''} p_{S''}(t)dS$ – forces acting on the outer surface of the valve of the second cascade S'';

c_2 – is the coefficient of viscous friction forces between the walls of the inertial mass 5 and the piston of the working element 4 (see Fig. 1);

N_{2y} – reaction forces of the conical surface of the valve of the second cascade from the conical seat of the inertia mass 5.

In this case, the inertial forces of the working fluid acting on the operating organs of the inertial vibratory rammer are neglected, as bearing a small contribution to the change in the motion of the inertial vibratory rammer in general.

In order to write down a completely mathematical model of the operation of the inertial vibratory rammer, the operation of the HID at the appropriate working phases of the two-stage valve-pulsator should be considered.

1. Phase of pressure set. In this phase, the conical valves with masses M_1 and M_2 are at rest, and the conical valve M_1 blocks the pressure cavities A, B and C from the drain cavity D (see Fig. 1). It leads to a set of pressure in the pressure cavity A, so the main pressure $p_S(t)$ acts on the area ($S - S_0$). It should be noted that this phase occurs during the period of time t_1, when the mobile body 4 contacts the surface of the upper soil layer 1. At this phase, the inertial mass $M_{ін}$. And at this stage there is a joint movement of the inertial mass $M_{ін}$ and locking elements of mass M_1 and M_2.

For this phase $t_2 \le t \le t_{нт}$ we write the following conditions:

$$\begin{cases} \iint\limits_{S'} p_{S'}(t)dS \le k_1 x''_{01}; \ \iint\limits_{S''} p_{S''}(t)dS < k_2 y''_{01}; \\ y''_2(t) = y''_1(t) = 0; \ \dot{y}''_2(t) = \dot{y}''_1(t) = \dot{y}'(t); \\ x''_1(t) = 0; \ 0 \ge y'_1(t) \ge y'_{1\max}; \ N_{45} = 0, \end{cases} \quad (6)$$

where $y'_{1\max}$ – maximum stroke of inertial mass $M_{ін}$.

2. Phase of operation (opening) of a two-stage pulsator valve. At this phase, the force from the pressure $\iint\limits_{S'} p_{S'}(t)dS$ acting on the area S_1 of the valve of the first cascade 6 is equalized with the tension force of the adjusting spring $k_1 x''_{01}$, that is:

$$\iint\limits_{S'} p_{S'}(t)dS \ge k_1 x''_{01}, \quad (7)$$

leading to open it. When the valve of the first cascade 6 is opened, the pressure cavity C communicates with the drain cavity D. This cavity message results in the relative velocity of the working fluid in the internal cavity of the two-stage pulsator valve. In turn, the fluid velocity in the internal cavity of a two-stage pulsator valve causes a pressure drop in the pressure cavities A and B due to the presence of a throttle hole in the valve of the second stage 7. The pressure different in the pressure cavities A and B results in the appearance of a force cascade move. When it happens, it is in the opening of this valve and, accordingly, the message of the pressure cavity A and drain cavity D.

At this phase, the inertial mass $M_{ін}$ also moves to the maximum displacement $y'_{1\max}$, the valve of the first cascade M_1 to the maximum opening $x''_{1\max}$ and the valve of the second cascade M_2 to the maximum opening. Therefore, for this phase $t_{нт} \le t \le t_{сп}$, there are got the following conditions:

$$\begin{cases} \iint\limits_{S'} p_{S'}(t)dS > k_1 x''_{01}; \ \iint\limits_{S''} p_{S''}(t)dS \ge k_2 y''_{01}; \ y''_1(t) = 0; \\ N_{1x} = 0; \ \dot{y}''_1(t) = \dot{y}'(t); \ 0 \ge x''_1(t) \ge x''_{1\max}; \ N_{45} = 0; \\ 0 \ge y''_2(t) \ge y''_{2\max}; \ N_{2y} = 0; \ \ 0 \ge y'_1(t) \ge y'_{1\max}. \end{cases} \quad (8)$$

3. Phase closing (lowering) of the two-stage valve-pulsator. At this phase, the working fluid is drained through the drain cavity D into the hydraulic tank 8. It causes a pressure drop in the pressure cavities A, B, C. At the same time, the valve of the first cascade with the mass M_1 begins to descend to its original place, (the place of overlap of the pressure cavity C from D). After the pressure cavity C has disconnected from the drain cavity D, the relative velocity of the working fluid in the internal cavity of the two-stage pulsator valve becomes zero. In turn, the relative immobility of the working fluid in the internal cavity of a two-stage pulsator valve causes pressure equalization in pressure cavities A and B. As a result of pressure equalization in pressure cavities A and B, the spring tension of the second stage valve $k_2(|y''|_2 + y''_{02})$ causes the valve of the second stage 7 to return to its original position. When it occurs, there is the closing of the pressure cavity A from the drain cavity D.

At this phase, reverse movements of the inertial mass $M_{ін}$, the valve of the first cascade M_1 and the valve of the second cascade M_2 to the initial position also occur. Therefore, for this phase $t_{сп} \le t \le t_{зк}$, here are the following conditions:

$$\begin{cases} \iint\limits_{S'} p_{S'}(t)dS \ge k_1 x''_{01}; \ \iint\limits_{S''} p_{S''}(t)dS \ge k_2 y''_{01}; \ y''_1(t) = 0; \\ \dot{y}''_1(t) = \dot{y}'(t); \ \ 0 \ge x''_1(t) \ge x''_{1\max}; \ \ 0 \ge y''_2(t) \ge y''_{2\max}; \\ 0 \ge y'_1(t) \ge y'_{1\max}; \ \ N_{45} = 0; N_{2y} = 0; \ \ N_{1x} = 0. \end{cases} \quad (9)$$

After the end of the closing (lowering) phase of the two-stage pulsator valve, the inertial mass 5 returns to its original position with the acquired speed $\upsilon_{ін} = \dot{y}'_1(t_{зк})$. When this occurs, the impact interaction with the working body 4 occurs. With impact interaction, the inertial mass 5 picks up the working body 4 and they together with the initial velocity υ_M, begin to move upwards (a model of an ordinary body thrown vertically upwards with some initial velocity [9]). The total speed of the entire inertial vibratory rammer υ_M is determined from the law of conservation of momentum [10], namely:

$$\upsilon_M = \frac{\left(M_{ш} + M_1 + M_2\right)}{\left(M_{ш} + M_1 + M_2 + M\right)}\upsilon_{ш}. \quad (10)$$

The movement of the working body 4 should be considered for two cases, namely: when the mobile body 4 contacts the surface of the upper soil layer 1 and when the working body 1 does not contact the surface of the upper soil layer 1 (the jump mode after the shock interaction of the inertial mass 5 and the working body 4).

Conditions working body motion 4 for the jump period

$$t_1 \le t \le t_2 \quad (11)$$

after the shock interaction of the inertial mass 5 and the working body 4 (no contact of the working body 4 with the surface of the topsoil 1) for two cases, namely:

$$\begin{cases} \delta k y_0' > \iint\limits_{S} p_S(t)dS;\ y'(t_3) = 0; \\ \dot{y}(t_1) = \upsilon_M;\ \ N_{14}(t) = 0; \end{cases} \quad (12)$$

$$\begin{cases} \delta k y_0' \le \iint\limits_{S} p_S(t)dS; \\ N_{14}(t) = 0; \quad N_{45}(t) = 0. \end{cases} \quad (13)$$

Conditions working body motion 4 for the period $t_0 \le t \le t_1$ (t_0 – is the beginning of the countdown of the inertial vibratory ramming operation) of the working body 4 contacting the surface of the topsoil 1. In this case, two cases must also be considered:

$$\begin{cases} t = t;\quad \delta k y' > \iint p(t)dS; \\ \dot{y}(t) = -\upsilon;\quad y'(t) = 0, \end{cases} \quad (14)$$

where t_2-t_1=$2\upsilon_M/g$ – flight time inertial vibratory rammer.

To determine the pressure change in the HID cavity, it is necessary to supplement the above laws of motion with the Navier-Stokes system of equations and the continuity equation (15, 16) for weakly squeezed fluids.

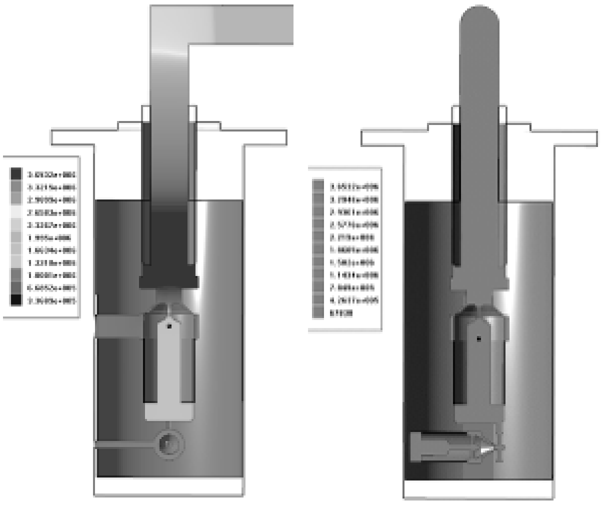
In this system of equations (15, 16) $\Omega \in R^3$ – is the three-dimensional region (internal cavity of the HID) in which the working fluid moves, ρ_0 – is the initial density of the working fluid, p_0 – is the initial pressure of the working fluid, Q_H – is the flow rate of the hydraulic pump when fed into the pressure cavity through inlet pipe, S_0 – cross-sectional area of the inlet pipe of the HID.

$$\begin{cases} \frac{1}{\rho}\frac{d\rho}{dt} + \frac{\partial V}{\partial x''} + \frac{\partial V}{\partial y''} + \frac{\partial V}{\partial z''} = 0; \\ \frac{\partial V}{\partial t} + \left(V\frac{\partial V}{\partial x''} + V\frac{\partial V}{\partial y''} + V\frac{\partial V}{\partial z''}\right) = -\frac{1}{\rho}\frac{\partial p}{\partial x''} + \\ +\frac{\nu}{3}\frac{\partial}{\partial x''}\left(\frac{\partial V}{\partial x''} + \frac{\partial V}{\partial y''} + \frac{\partial V}{\partial z''}\right) + \\ +\nu\left(\frac{\partial V}{\partial x''} + \frac{\partial V}{\partial y''} + \frac{\partial V}{\partial z''}\right); \\ \overline{V}\big|_{d\Omega} = 0;\ \Omega \in R^3; \\ \overline{V}\big|_{t=0,z''=0,y''=0,x''=0} = Q_H / S_0; \end{cases} \quad (15)$$

$$\begin{cases} \frac{\partial V_y}{\partial t} + \left(V_x\frac{\partial V_y}{\partial x''} + V_y\frac{\partial V_y}{\partial y''} + V_z\frac{\partial V_y}{\partial z''}\right) = -\frac{1}{\rho}\frac{\partial p}{\partial y''} + \\ +\frac{\nu}{3}\frac{\partial}{\partial y''}\left(\frac{\partial V_x}{\partial x''} + \frac{\partial V_y}{\partial y''} + \frac{\partial V_z}{\partial z''}\right) + \\ +\nu\left(\frac{\partial^2 V_y}{\partial x''^2} + \frac{\partial^2 V_y}{\partial y''^2} + \frac{\partial^2 V_y}{\partial z''^2}\right); \\ \frac{\partial V_z}{\partial t} + \left(V_x\frac{\partial V_z}{\partial x''} + V_y\frac{\partial V_z}{\partial y''} + V_z\frac{\partial V_z}{\partial z''}\right) = -\frac{1}{\rho}\frac{\partial p}{\partial z''} + \\ +\frac{\nu}{3}\frac{\partial}{\partial z''}\left(\frac{\partial V_x}{\partial x''} + \frac{\partial V_y}{\partial y''} + \frac{\partial V_z}{\partial z''}\right) + \\ +\nu\left(\frac{\partial^2 V_z}{\partial x''^2} + \frac{\partial^2 V_z}{\partial y''^2} + \frac{\partial^2 V_z}{\partial z''^2}\right);\ \rho\big|_{t=0,\Omega} = \rho_0. \\ p\big|_{t=0,\Omega} = p_0 + \left(\frac{(m + m_0 + m_3)g + k y_0'}{\left(S - S_0\right)}\right). \end{cases} \quad (16)$$

A mathematical model of the technological process of surface compaction of inertial vibratory rammer, represented by the systems of equations (1) - (16), was implemented by numerical simulation methods based on the FlowVision [11] and Matlab-Simulink [12] software systems. The simulation results are the pressure distribution in the working cavity of the HID inertial vibratory rammer (Fig. 3).

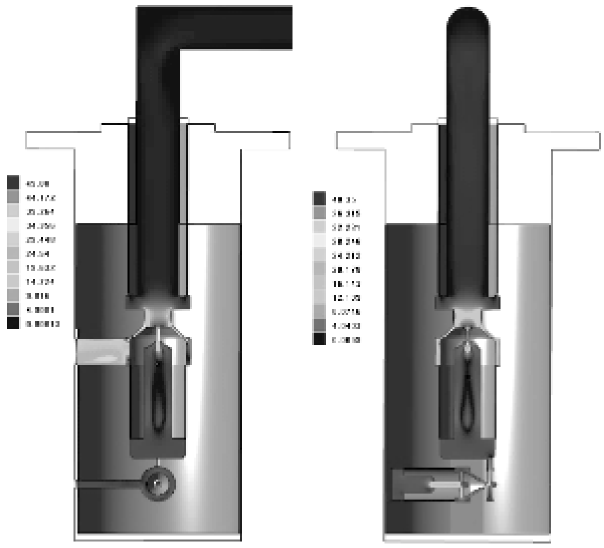
From Figure 3 it can be seen that the throttle 14 mounted in the valve of the second cascade 10 (see Fig. 1) makes a significant character in the pressure change (2 MPa) in the cavity between the valve of the first 11 and the second 10 cascades (see Fig. 1).



a) b)

Figure 3 – Pressure distribution in the working cavity of inertial vibratory rammer based on the HID:
a) in the *yOz* plane; b) in the *yOx* plane

Also, one of the simulation results is the distribution of pressure and velocity in the working cavity of the HID inertial vibratory rammer (Fig. 4).



a) b)

Figure 4 – Velocity distribution of the working fluid in the cavity of inertial vibratory rammer based on the HID:
a) in the *yOz* plane; b) in the *yOx* plane

Analyzing Figure 4, it can be observed extreme values of speeds in the valvesseats of the first 11 and second 10 stages, respectively (see Fig. 1), as well as at the throttle bore 14. These extreme speeds can cause cavitation phenomena that can adversely affect technical state of HID parts.

Numerical modeling of hydrodynamic processes of the HID inertial vibratory rammer enabled to obtain:

1) diagrams of pressure changes (Fig. 5) in working cavities *A* and *B* (see Fig. 2);

2) changes in movement (Fig. 6) of the housing of a two-stage pulsator valve 8, valve of the second stage 10 and valve of the first stage 11 relative to tamping plate 1 (see Fig. 1);

3) changes in the speed (Fig. 7) of the housing of the two-stage pulsator valve 8, the valve of the second stage 10 and the relative valve of the first stage 11 relative to the tamping plate 1 (see Fig. 1).

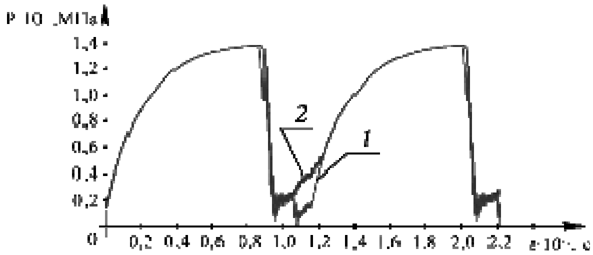


Figure 5 – The pressure distribution of the working fluid in the empty inertial vibrating rammer based on the HID:
1 – in cavity *A*; 2 – in the plane *B*

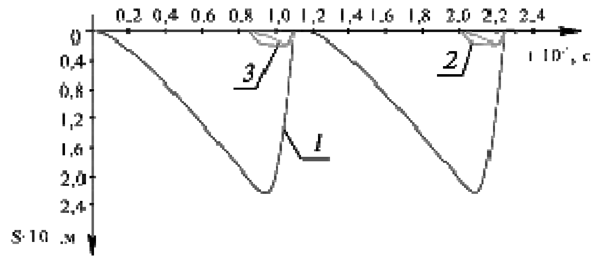


Figure 6 – Changing the movement of moving elements of inertial vibratory rammer based on the HID:
1 – the body of the two-stage valve-pulsator 8;
2 – valve of the second cascade 10;
3 – valve of the first cascade 11

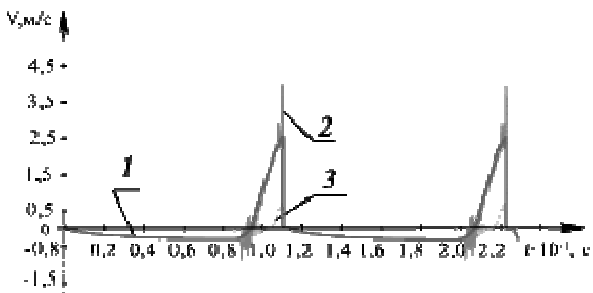


Figure 7 – Changing the speed of moving elements of inertial rammer based on the HID:
1 – the body of the two-stage valve-pulsator 8;
2 – valve of the second cascade 10;
3 – valve of the first cascade 11

Additional absolute displacement diagrams (Fig. 8) of the two-stage pulsator valve 8 body were obtained, taking into account the jump of inertial vibrating rammers over the soil surface 2, as well as the mass' centers of soil layers 1, 2, 3 of the loam type [10] (see Fig. 2).

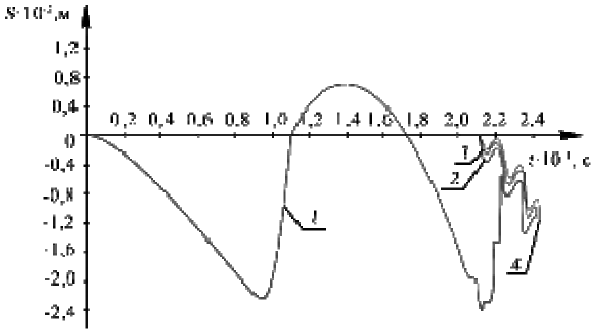


Figure 8 – Change in the absolute movement on technological objects of surface soil compaction:
1 – two-stage valve-pulsator housing;
2 – the mass' center of the soil layer 1;
3 – the mass' center of the soil layer 2;
4 – the mass' center of the soil layer 3

From the absolute displacement diagram for the mass centers of soil layers 1, 2, 3 of the loam type, it can be seen that the maximum deformation of the soil surface (upper layer 1) is 1.2 cm at an oscillation frequency of inertial mass 5 (see Fig. 2) 8.7 Hz.

Conclusions

An effective design of inertial vibrating rammers, based on a two-stage pulsator valve, has been developed. It enables to implement the most effective modes of vibration exposure with surface compaction of the soil.

On the basis of the hydrodynamics laws with the use of mechanical rheological phenomenology and generalized laws of mechanics, a new mathematical model has been developed for the study of technological processes of surface soil compaction with inertial vibratory rammer.

On the basis of the developed mathematical model, using the method of finite volumes using numerical simulation, the working dependences for determining the basic performance characteristics of the technological process of surface soil compaction using inertial vibratory rammer based on the HID are obtained.

The obtained results of numerical simulation for technological processes of surface soil compaction by the inertial vibratory rammer based on the HID. They showed the advantages of the chosen design approach, and also allowed to prove the effectiveness of the developed HID design, based on a two-stage pulsator valve.

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Vertical differential grout pump experimental studies methods validation

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The article is devoted to creation and substantiation of experimental research method of vertical differential grout pump with flowing piston mark PH 2-4, developed by researchers at Poltava National Technical Yuri Kondratyuk University. Experiments type has been substantiated, variation intervals of the studied parameters values have been determined, the preparation of the experimental sample of the grout pump for research has been described, an experimental installation for research has been designed, the measuring means have been selected, the conditions for performing the experiments have been determined. This technique enables to find the rational values for the following parameters of the solution pump when working with plaster solutions of different displacement: working body frequency movement, diameter, mass and height of lifting over the valve, the hole diameter in the valve sockets.

Keywords: plastering works, plaster works complex mechanization, plaster works mechanization equipment, mortar transportation by pipelines, mortar pump, test facility, test procedure, experiment, experimental accuracy.

Обґрунтування методики експериментальних досліджень вертикального диференціального розчинонасоса

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Статтю присвячено створенню та обґрунтуванню методики експериментальних досліджень вертикального диференціального розчинонасоса з проточним поршнем марки РН 2-4, розробленого науковцями Полтавського національного технічного університету імені Юрія Кондратюка. Метою експериментальних досліджень є: перевірка висунутих під час аналітичних досліджень гіпотез, обґрунтування вибору найбільш раціональних значень основних конструктивних параметрів розроблюваного розчинонасоса, обґрунтування сфери застосування методів регулювання подачі шляхом зміни довжини та швидкості руху робочого органа, а також отримання експлуатаційних регулювальних характеристик досліджуваного насоса. Обґрунтовано тип експериментів, визначено проміжки варіювання значень досліджуваних параметрів, описано підготовку експериментального зразка розчинонасоса для досліджень, спроектовано експериментальну установку для досліджень, вибрано засоби вимірювань, визначено умови виконання дослідів. Як критерій ефективності прийнятих конструктивних рішень прийнято об'ємний коефіцієнт корисної дії розчинонасоса. Виготовлена дослідницька установка дозволяє приготувати будівельний розчин необхідної рухомості та складу і визначити час наповнення мірної місткості при різних комбінаціях величин досліджуваних конструктивних параметрів розчинонасоса. Значну увагу приділено визначенню похибок проведення експериментів. Запропонована методика дозволяє гарантувати з імовірністю 90%, що абсолютна похибка у визначенні об'ємного коефіцієнта корисної дії на основі середньоарифметичного значення п'яти замірів при кожній комбінації досліджуваних параметрів, зумовлена систематичною і випадковою складовою, не перевищить 1,9%. Така методика дозволить знайти раціональні значення наступних параметрів розчинонасоса при роботі зі штукатурними розчинами різної рухомості: частота руху робочого органа, діаметр, маса і висота підйому над гніздом кульок клапанів, діаметр отвору в гніздах клапанів.

Ключові слова: штукатурні роботи, комплексна механізація штукатурних робіт, засоби механізації штукатурних робіт, транспортування будівельних розчинів трубопроводами, розчинонасос, випробувальний стенд, методика випробувань, експеримент, похибка експерименту.

Introduction

During the construction of multi-dwelling houses, a significant amount of construction work is made by plastering walls wet processes. They are labor-intensive and difficult to mechanize. Recently, transporting building plaster solutions to the workplace plaster on pipelines technology is widely used for the purpose of their mechanized application on the surface to be treated with the help of a special nozzle. During the transport of plaster mixes by pipelines, there are problems [1, 2, 3], which were tried to solve by a number of scientists.

Review of research sources and publications

The researches of the Department of Building Machines and Equipment [4, 5] Poltava National Technical Yuri Kondratyuk University have made significant progress in solving these problems [6 – 14], proposing the design of new machines and substantiating their effectiveness. Technological process main machine for plastering works is a grout pump. A group of scientists and the author of the article as well, developed a vertical differential grout pump of a new design, which was called PH 2-4 [15 – 21].

Definition of unsolved aspects of the problem

The results of analytical studies and the design of the experimental sample displayed the advantages of the developed grout pump before the existing ones. However, they enabled only the qualitative dependence on influence of the design parameters on the efficiency of its work. Therefore, the purpose of experimental studies is to check the hypotheses proposed in the process of analytical research, to substantiate the choice of the most rational values of the basic design parameters of the grout being developed, to validate the scope of the application supply control methods by changing the length and frequency of the working body stroke, as well as obtaining the operational control characteristics of the pump under test.

To achieve the above objective, the following tasks need to be solved:

– to determine the type of experiments;

– to determine the range of variation of the values of the parameters under study;

– to prepare an experimental sample of a mortar pump for research;

– to create an experimental plant for research;

– to select measurement tools;

– to determine the conditions for the experimentation.

Problem statement

The purpose of this article is to create and substantiate the methodology of experimental research of a vertical differential solenoid pump with flowing piston of the mark PH 2-4.

Basic material and results

Considering the task set for the study of the dissolved pump design parameters influence developed on its volume efficiency, laboratory experiments were selected, since they are less labor-intensive and, due to the high reproducibility of the conditions for the production of experiments, enable to obtain fairly accurate results.

In order to achieve the set goal, it is provided in the process of staging experiments to be varied by the following parameters of the grout pump: height of lift, diameter and ball of the suction valve, as well as the mass of this ball at constant diameter, length and frequency working body movement.

In connection with it, steel balls with a diameter of 40, 50 and 60 mm in weight, respectively, of 0.26, 0.51 and 0.89 kg were purchased for the suction valve (Figure 1, 3). Two Silverstone balls are cast in diameter 60 mm (Figure 1, 3). Due to the presence of internal weighing cores, their masses are 0.28 and 0.55 kg, which is close to the masses of steel beads with diameters, respectively, 40 and 50 mm. Instead of a standard ball valve lifting limiter, adjustable balls (Figure 1, 4 and Figure 2, 3) are made for each ball size, which allow a height change of 0…25 mm to be changed. They are supplied with replaceable valve sockets (Figure 1, 2 and Figure 2, 4). The diameters of these nests are arranged so that for each size of the ball, the value of K is provided in the range of 0.3…0.9 in increments of 0.1 and is 12, 15, 16, 18, 20, 24, 25, 28, 30, 32, 35, 36, 40, 42, 45, 48 and 54 mm.

The stroke length of the operating body of the grout pump is changed by the normal supply regulator within 0…72 mm.

A pair of step pulleys (Fig. 1) is made to change the operating frequency of the working member. The steps of the diameters are calculated in such a way that, in the presence of a standard motor, considering its slipping, the operating frequencies of the operating unit of the grout pump are 126.0, 151.5 and 177.0 rpm (gradation is 25.5 rpm). These frequencies correspond approximately to the theoretical performance of the pump 4, 5 and 6 m^3/h. The type of section and tension of the belt are chosen from those considerations, in order to avoid slipping in the range of loads under investigation (up to 0.6 MPa). The results of search experiments displayed that there is practically no increase in the slippage of an electric motor with increasing load in this range.

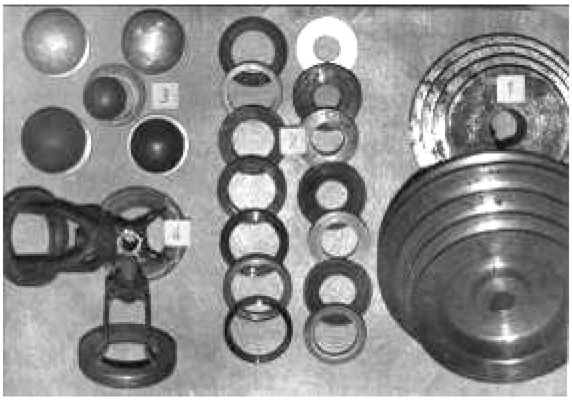

Figure 1 – Grout pump replaceable elements for varying studied design parameters value

An experimental installation (Figure 2) has been designed and manufactured for research of the considered grout pump, which includes the mixer for solution 1, the investigated pump 2, the load device 7 and the measuring capacity 5. The discharge pressure is monitored using a standard pressure gauge 6.

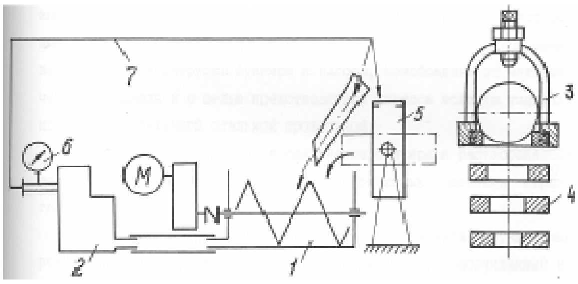

Figure 2 – Experimental installation schematic diagram for the study of grout pumps

The capacity of the mixer for the solution is determined by the required volume of the prepared batch, which is determined from the following considerations: 20 liters of solution are required for filling the loading arm, measuring capacity 60 liters, 10 liters of the suction arm and cavities of the grout pump. In order for the grout pump not to «swallow» the air during the suction cycle, at least 80 liters of solution must remain in the mixer. Since the mixing efficiency is significantly reduced when the mixer's working body is fully buried, we reserve a reserve volume of 110 liters. The mixer capacity in this case is 300 liters.

It is equipped with a reversible belt unit with a rotational speed of 18 rpm, which consists of two equal parts with opposite winding. In this case, when the shaft rotates in one direction, the solution flows to the middle of the bunker, and when it rotates to the other, to its ends. Reversible rotation of the mixer provides faster and better mixing of the solution. In the middle lower part of the mixer there is a pipe for connecting a grout pump.

For ease of maintenance, the studied pump, together with the mixer, is raised above the floor to a height of 0.3 m. The suction inlet of the mortar pump is connected to the hopper by a vacuum sewage rubber hose with a metal frame 75 mm in diameter and 0.7 m long and the pump, freed from the metal frame and in order to prevent air leaks, are carefully crimped with annealed steel wire.

Mutual arrangement and connection of the mixer and mortar pump correspond to the most used in real conditions of the construction site.

A plaster rubber-fabric sleeve with a conditional passage of 50 mm, designed for a pressure of 4.0 MPa, 20 m long, rolled into a coil with a diameter of at least 2 m is used as a load device. It is connected to the pump using a standard quick-release coupling.

The measuring tank with a volume of 60 liters is made in the shape of a cylinder with a diameter of 300 mm with the bottom. The design of the stand enables it to tip over for emptying into the mixer.

The experimental set also includes: a special scraper for cleaning the remaining solution from the measuring container and a device that prevents the solution from passing into the suction hose from the mixer while the suction nozzle is disconnected from the mortar pump for the next setting of variable parameters of the suction valve.

Removal of the spent solution and flushing of the system is carried out using the mortar pump under study. The water supply is provided from the water supply both for flushing and cooling the seal of the rotor pump rod, and for flushing the mixer and measuring container after removing the spent solution from the system.

The appearance of the installation is shown in Figure 3.

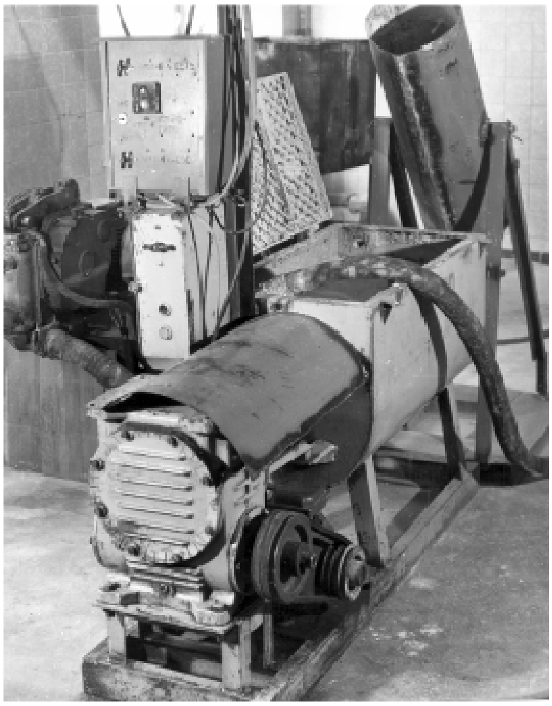

Figure 3 – Experimental installation for grout pumps study

For research, we use the most frequently pumped mortar-pumped lime-sand grout of a 1:3 composition with aggregate fractions up to 5 mm. Its mobility is determined using the standard cone according to the method described in [22].

Exploratory experiments have shown that with stirring in a mixer and when pumping a solution by a pump over a closed ring, its intense heating occurs, which in turn leads to a change in its consistency. A specially conducted series of measurements showed that an increase in the mobility of a solution by 1 cm occurs when its temperature increases by 3…4 ºC.

Following measurement procedure was adopted. Set the required values of the studied design parameters. The solution is prepared somewhat lower mobility than is necessary for the formulation of experience. When the solution in the bunker becomes homogeneous, the mortar pump is turned on and the mobility of the pumped medium is periodically measured. As soon as its value reaches the required value, the pressure is transferred from the mixer to the measuring tank with simultaneous activation of the stopwatch. The time of its filling is noted and the mobility of the solution is measured again. If the mobility during the filling of the tank has not changed, the result of the experiment is considered valid. Otherwise, if the solution is fresh, let it cool and repeat the experiment, and if it is outdated, replace it.

Thus, the experimental setup enables to prepare a grout of the required mobility and composition and determine the time for filling the measuring capacity with various combinations of the values of the studied design parameters of the grout pump.

Measuring instruments and experimental techniques. The chosen volumetric efficiency of a grout pump as a criterion for the effectiveness of design solutions. η_0, %, cannot be measured directly. It is determined by indirect methods – by direct measurement of its functionally determining quantities.

It is advisable to use the following dependency

$$\eta_0 = \frac{Q}{Q_T}100 = \frac{24 \cdot 10^9 \cdot V}{\pi \cdot D^2 \cdot n \cdot L \cdot t}, \quad (1)$$

where Q – experimentally measured pump flow;
Q_T – theoretical supply;
V – measured capacity volume, $V = 60$ L;
D – grout pump piston diameter, $D = 100$ mm;
L – pump body stroke length, mm;
n – working body movement frequency, rpm;
t – time of filling with a solution the measuring tank.

Calibration of the measuring tank was made by weighing method by weighing it in an empty and filled with water condition. Water was poured until the mass of the tank with water exceeded by 60 kg the mass of the empty tank. For this purpose, we used platform scales mobile scale РП-200Ш13 with a scale of 50 g. It is considered the piston diameter to be an exact value equal to 100 mm, since the inaccuracy is enabled by us in this case is negligible compared to the measurement accuracy of other quantities included in the above formula. It is determined the length of the stroke working body using the square with the price of 1 mm division. The time of filling the measuring vessel with a solution and the frequency of working body movement are determined using the stopwatch СДСпр-1-2-800 with a dividing price of 0.1 s. As a result of exploratory experiments, it was established that the inaccuracy of the established number of strokes of the working body within 1 minute is no more than ± 0.5 double strokes. Over this limit, the values did not go out either with an increase in load or with variation of the power network parameters.

The measurement inaccuracy is determined by the recommendations set out in [23, 24]. To do it, in the center of the response surface (in the area of the expected extremum), select the "point" and, using the proposed method, find the volume efficiency here. The following values of the studied design parameters were chosen as such a «point»: $D_{III} = 60$ mm (steel ball), $OK = 10$ cm, $n = 126$ rpm, $H = 15$ mm, $K = 0.7$, $L = 72$ mm. The filling time of the measuring tank was 68.7 s. Then, the volumetric efficiency is

$$\eta_0 = \frac{24 \cdot 10^9 \cdot 60}{\pi \cdot 10^4 \cdot 126 \cdot 72 \cdot 68.7} = 73.5 . \quad (2)$$

The relative inaccuracy δ, %, in the definition of the values included in the formula are:

$$\begin{aligned} \delta_V &= \frac{\sqrt{2} \cdot 0.05}{60}100 = 0.12 ; \\ \delta_n &= \frac{0.05}{126}100 = 0.4 ; \\ \delta_L &= \frac{\sqrt{2} \cdot 1}{72}100 = 1.96 ; \\ \delta_t &= \frac{0.1}{68.7}100 = 0.15 , \end{aligned} \quad (3)$$

where is the multiplier $\sqrt{2}$ means that the inaccuracy is calculated as the value of the lower and upper measurements inaccuracy. All of these inaccuracies are components of the non-excluded residuals of systematic inaccuracy.

Boundaries Θ_j, %, for inaccuracy in determining the volumetric efficiency introduced j onsisting non-excluded residuals of systematic inaccuracy distinguished by formula

$$\Theta_j = \frac{\eta_0}{100} \cdot \delta_j , \quad (4)$$

Then

$$\begin{aligned} \Theta_V &= \frac{\eta_0}{100} \cdot \delta_V = \frac{73.5}{100} \cdot 0.12 = 0.09 ; \\ \Theta_n &= \frac{\eta_0}{100} \cdot \delta_n = \frac{73.5}{100} \cdot 0.4 = 0.29 ; \\ \Theta_L &= \frac{\eta_0}{100} \cdot \delta_L = \frac{73.5}{100} \cdot 1.96 = 1.44 ; \\ \Theta_t &= \frac{\eta_0}{100} \cdot \delta_t = \frac{73.5}{100} \cdot 0.15 = 0.11 . \end{aligned} \quad (5)$$

Non-excluded residuals of systematic inaccuracy boundaries $\Theta(P)$, %, at the figure of compounds $N \geq 4$ is counted [24] by formula

$$\Theta(P) = \pm k \cdot \sqrt{\sum_{j=1}^{j=4} \Theta_j^2} , \quad (6)$$

where k – coefficient determined by the selected confidence level P and figure N compound [23]

$$\begin{aligned} &\Theta(0.9) = \pm 0.95 \times \\ &\times \sqrt{0.09^2 + 0.29^2 + 1.44^2 + 0.11^2} = \pm 1.4 \end{aligned} , \quad (7)$$

An additional inaccuracy introduces inaccuracy of the solution composition dosage and the change in its properties over time. To reduce its influence, all measurements made at each «point» are distributed in time, performing them in different batches and in different sequences with measurements at other «points».

To obtain empirical dependencies, statistical methods were used, such as experiment planning and the least squares method, which enabled not only to reduce the amount of experimental work, but also to obtain a fairly reliable and accurate mathematical interpretation of existing patterns. The most effective, and most importantly, correct use of these methods requires preliminary processing of measurement results, which consists in evaluating the experimental error with the required reliability, determining the required number of experience repetitions, eliminating gross measurement errors, verifying the compliance of the distribution of measurement results with the law of normal distribution and, if necessary, in converting this distribution to normal. Preliminary processing of measurement results is made in accordance with the requirements. [23 – 29].

As a result of measurement at each «point», the arithmetic average of several measurements made here has been used. The number of these measurements depends on the accepted values of the confidence interval and the confidence probability that the true value of the measured value does not go beyond this interval. In other words, the number of experience repetitions is determined by the magnitude of the maximum error, with which the arithmetic mean of the measurements to correspond to the true value of the measured quantity, and the probability that the current error does not exceed the specified maximum value. Increasing the number of measurements at each «point» even with their accuracy unchanged, the reliability of confidence estimates can be increased or the confidence interval for the true value of the measured value can be narrowed. However, it leads to an increase in the complexity of the experiment. Therefore, the number of experience replication in research in the field of technology is not recommended [26] to take $N > 8$, and the use of statistical data processing methods loses meaning [23], if $N < 4$.

The choice of the number of measurements from the specified range is made by calculation [25, 26, 29], for which purpose in the center of the assumed extreme area at the selected point, four measurements of the filling time of the measuring capacity are taken and the corresponding values of the volumetric efficiency are calculated. As a result of the experiment, four values of volumetric efficiency η_{0cp} are got: 73.5, 74.5, 76.0, 74.0 %.

Calculate the arithmetic mean value η_{0cp}, %, received sample

$$\eta_{0cp} = (73,5+74,5+76,0+74,0) / 4 = 74,5 \quad (8)$$

Corrected variance S_{uc}^2 this empirical distribution is determined [26] by formula

$$S_{uc}^2 = \frac{\sum_{i=1}^{N=4}\left(\eta_{0_i} - \eta_{0_{cp}}\right)^2}{N-1} = \frac{1}{4-1}\cdot\begin{bmatrix}\left(73.5-74.5\right)^2+\left(74.5-74.5\right)^2+\\+\left(76-74.5\right)^2+\left(74-74.5\right)^2\end{bmatrix} = 1.17, \quad (9)$$

where η_{0i} – the current value of the volumetric efficiency of the resulting series.

Selective standard deviation S_{uc} is [26]

$$S_{uc} = \sqrt{S_{uc}^2} = \sqrt{1.17} = 1.1 . \quad (10)$$

The value obtained is an unbiased estimate for the standard deviation σ theoretical distribution

Evaluation $S_{uc}\left(\eta_{0_{cp}}\right)$, %, the standard deviation of the random component of the calculated average value error $\eta_{0_{cp}}$ is calculated by formula

$$S_{uc}\left(\eta_{0_{cp}}\right) = \frac{S_{uc}}{\sqrt{N}} = \frac{1.1}{\sqrt{4}} = 0.55 . \quad (11)$$

In normal studies in the technique for finding the dependencies of the influence of various factors, the confidence probability of 0.9 is sufficient [29]. Based on its value, according to [29], the compromise between the value of the permissible absolute error of the experiment is chosen, which is given in the table in fractions of the standard deviation of the theoretical distribution σ, and the number of replications of experience. The permissible error equal to $1.0\cdot\sigma$, achieved when performing 5 measurements ($N=5$) is appropriate.

In this case, the confidence limits $\varepsilon(P)$, random inaccuracy of the measurement result at a confidence level $P=0.9$ и $N=5$ specify [23] according to the formula

$$\varepsilon(P) = t(P,N)\cdot S_{uc}\left(\eta_{0_{cp}}\right), \quad (12)$$

where $t(P,N)$ – Student's coefficient [14].

$$\varepsilon(P=0.9) = 2.132\cdot 0.55 = 1.17 . \quad (13)$$

For distinguishing the sum $\Delta(P)$, %, systematic and random components of the measurement result inaccuracy, it can be estimated the ratio

$$\frac{\Theta(P)}{S_{uc}\left(\eta_{0_{cp}}\right)} = \frac{1.4}{0.55} = 2.54 . \quad (14)$$

As $0.8 < 2.54 < 8$, inaccuracy confidence limit result is determined [14] by formula

$$\Delta(P) = K(\gamma)\cdot\left[\Theta(P)+\varepsilon(P)\right], \quad (15)$$

where

$$\gamma = \frac{\Theta(P)}{\sqrt{3}\cdot k\cdot S_{uc}\left(\eta_{0_{cp}}\right)} = \frac{1.4}{\sqrt{3}\cdot 0.95\cdot 0.55} = 1.5 , \quad (16)$$

$$K(\gamma) = \frac{\sqrt{1+\gamma^2}}{1+\gamma} = \frac{\sqrt{1+1.5^2}}{1+1.5} = 0.72 . \quad (17)$$

Then

$$\Delta(P=0.9) = 0.72\cdot(1.4+1.17) = 1.9 . \quad (18)$$

Thus, performing five measurements of the time of filling the measuring capacity with a solution on each of the «points» under study, guarantees with 90 % probability that the absolute error in determining the volumetric efficiency based on the arithmetic average of these measurements will not exceed 1.9 %.

If, when performing measurements at the «point», a measurement result is obtained that differs sharply from all others, a suspicion arises that a gross error has been made. In this case, it is immediately verified that the basic measurement conditions are not violated. If such a check was not done on time or it was unsuccessful, then the question of one «pop-up» value rejection expediency is solved by comparing it with the rest of the measurement results.

For the number of the experiment repetitions from 3 to 5, the following method for screening gross errors is recommended [26]. The arithmetic average and corrected variance of the obtained data has been calculated and the maximum relative deviation τ suspicious metering has been determined

$$\tau = \frac{\left|\eta_{0_i} - \eta_{0_{cp}}\right|}{S_{uc}} \le \tau_{1-p}\,, \tag{19}$$

where η_{0_i} – «pop-up» value of volumetric efficiency.

Gained value τ compare with the tabular quintile distribution for the maximum relative deviation τ_{1-p}, taken from the table in [26] with a confidence level of 95 % (which corresponds to the level of significance $p = 0.05$). In this case, with five measurements $\tau_{1-p} = 1.87$ if the value obtained does not exceed the table value, then this measurement is not screened out. In case of the measurement exclusion, the characteristics of the empirical distribution should be recalculated according to the data of the reduced sample, after which the screening procedure is repeated for the next maximum absolute deviation of the measurement in absolute value. If, as a result of screening, the sample remains less than four values, it must be supplemented with additional measurements (in our case up to five values) and repeat the procedure.

To test the hypothesis of the normal distribution values the volumetric efficiency obtained at the «point», it is recommended [26] to calculate the mean absolute deviation using the formula

$$CAO = \frac{\sum\left|\eta_{0_i} - \eta_{0_{cp}}\right|}{N}. \tag{20}$$

If the series of values obtained has an approximately normal distribution law, the expression should be true:

$$\left|\frac{CAO}{S_{uc}} - 0.7979\right| < \frac{0.4}{\sqrt{N}}. \tag{21}$$

The issue of eliminating gross inaccuracy and testing the hypothesis of the normal distribution results obtained was solved on a PC using the MathCAD software product. There were no deviations from the normal distribution.

Conclusions

The method of experimental research of a vertical differential solenoid pump with flowing piston of mark PH 2-4, developed at Poltava National Technical Yuri Kondratyuk University, is proposed. The compliance of this method with the purpose of increasing the efficiency research grout pump is substantiated. The equipment necessary for conducting experimental research is selected. It is verified inaccuracy in conducting research. This technique enables to find the rational values of the following parameters of the solution pump when working with plaster solutions of different displacement: working body frequency movement, diameter, mass and height of lifting over the valve, the diameter of the hole in the valve sockets.

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Research of electric car dynamics

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Recently, electric vehicles are becoming widespread throughout the world, so the research is relevant. The results of performed researches of dynamics electric car on the basis of ZAZ-1102 are presented. The calculation method of dynamic loads in electric and mechanical systems of electric cars is developed. The technique considers the electromagnetic processes in the engine, the elasticity of the elastic parts, the oscillatory processes, and damping in the elastic links. On the basis of the developed mathematical model and using the mathematical software apparatus MathCAD, calculations of transients in the electric and mechanical systems have been obtained, based on the obtained results of the research, the constructed graphs. The obtained results of the study of the electric motor drive mechanism can be used for designing, calculating and determining the dynamic loads of electric cars and their hybrids.

Keywords: mathematical model, electric car, drive, mechanical and electrical systems, dynamic loads, flexibility, electromechanical processes, oscillatory phenomena.

Дослідження динаміки електромобіля

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Останнім часом електромобілі набувають поширення в усьому світі, тому проведені дослідження є актуальними. Наведено результати виконаних досліджень динаміки електромобіля на базі ЗАЗ-1102. Запропонована методика розрахунку динамічних навантажень в електричній та механічній системах електромобіля на базі ЗАЗ-1102. У методиці враховано електромагнітні процеси у двигуні, податливість пружних ланок, коливальні процеси, демпфування у пружних ланках. Розроблено нову математичну модель для дослідження динаміки електромобіля. Розрахункову схему електромобіля з приводом наведено у вигляді двомасової пружної системи без урахування податливості нерухомої частини електромобіля, вважаючи, що вона має велику жорсткість. Привід електромобіля забезпечується за допомогою одного електродвигуна. При розгляді динамічних навантажень, які виникають у механізмі привода електромобіля під час його пуску, як основний критерій умов навантаження системи прийнято поворот ведучого колеса з максимальним його навантаженням, тому приведення всіх мас привода, жорсткостей пружних ланок, а також сил здійснено до осі обертання ведучого колеса. Елементи обертових частин привода, ведучі (привідні) колеса які мають значні жорсткості при невеликих розмірах по довжині, прийнято як ланки із зосередженими масами. За допомогою розробленої математичної моделі з використанням математичного програмного забезпечення MathCAD досліджено динаміку пуску привода електромобіля та отримано сили, моменти, прискорення, що виникають у пружних ланках електромеханічної динамічної системи. За допомогою розробленої математичної моделі розраховано перехідні процеси в електричній та механічній системах, побудовано графіки залежностей кутових переміщень, швидкостей і прискорень мас, зміни моменту електродвигуна привода електромобіля від часу, які наведено в цій науковій роботі. Отримані результати досліджень може бути використано при проектуванні, розрахунках та визначенні динамічних навантажень у приводах електромобілів.

Ключові слова: математична модель, електромобіль, привід, механічна й електрична системи, динамічні навантаження, податливість, електромеханічні процеси, коливальні явища.

Introduction

The market for electric cars in the world is gaining momentum. Today, it can be state with confidence, that the future is precisely in the electrified cars. Countries with advanced high-tech industries are interested in environmentally friendly transport and stimulate its development. Large manufacturers of cars and technology companies in China, Sweden, France, the United States, Germany, etc., are planning to expand and improve their products based on state support. Thus, the prospects in the near future of electric cars become wider and more realistic.

If compare an electric car with an ordinary car, where the internal combustion engine is used, it is characterized by a simpler scheme, the minimum number of moving elements. Therefore, the electric car is a more reliable design. The main components of an electric car are as follows: electric motor; rechargeable batteries; simplified transmission; inverter charger on board; electronic control system for structural elements; transformer. Therefore, the electric car components study, including its drive, is relevant.

Review of research sources and publications

Such scientists as Bakhmutov S.V., Karunin A.L., Krutashov A.V., Kapustin A.A., Rakova V.A., Umyashkin V.A., Filkin N.M., Muzafarov R.S., Bazhynov O.V., Smyrnov O.P., Syerikov S.A., Hnatov A.V., Kolyesnikov A.V. and so on devoted their work to the research and analysis of electro and hybrid cars, including their drives to determine the frequency of oscillations. In these works, electric and hybrid cars were described by various calculation schemes and their dynamical systems were considered as one and two mass oscillatory systems that enabled to determine the frequency of oscillations [1 – 4]. The work of foreign scientists Collatc L. [5] and Tondl A.[6], Kluchev V.I., Jagadish H.P., Kodad S.F. [8], Kaplan D. [9] are also devoted to the mechanical systems oscillations frequency determination.

When designing an electric car, the question arises about the evaluation of its run by selecting the design parameters of the traction drive, which includes the parameters of the electric motor, transmission and power supplies [10 – 17].

The developed methods of choosing the design parameters of a traction drive of an electric car to achieve a predetermined mileage are based on well-studied characteristics of batteries (lead-acid, nickel-cadmium and others). Manufacturers of modern traction batteries (nickel-metal hydride, lithium-ion, etc.) at the request of the batteries capacity and other characteristics indicate different conditions of temperature, time and discharge current, which impedes their comparative analysis and leads to ambiguous evaluation of charge-discharge characteristics, which is significantly affected run on electric.

In addition, the previously developed techniques use the simplification of the motion equation of an electric car at constant speed or in cycles that do not correspond to the real conditions of motion.

Therefore, improvement of the calculation methods and choice of structural parameters of an electric car, considering the process of discharging modern traction batteries and a full-scale model of car movement in modern urban conditions, is an actual direction of development of electric car performance characteristics improving during its design.

Definition of unsolved aspects of the problem

An electric car drive can be considered as a oscillational system, in which during the start-up and braking periods there are significant dynamic loads. The requirements for the calculation accuracy of such oscillatory systems need to consider both the individual oscillations of the individual elements and the system as a whole due to the fact that as a result of the own and forced oscillations frequencies coincidence there may be resonance phenomena that cause significant dynamic loads and, as consequence, reduce the durability of structures.

At this time, when calculating electric cars for static and tired strength, their own design oscillations are not considered. However, the carrying capacity of electric car designs can be increased if their design in the calculations considers their oscillations amplitude-frequency characteristics.

Thus, the frequencies and electric car forms designs elements proper oscillations determination enables to compare these frequencies with forced frequencies, as well as to obtain data on the loading of the dynamic system, and thus to ensure the normal operation of car all oscillation parts in modes that are far removed from the resonance.

When designing forming posts and the choice of technological equipment for consolidating the concrete mixtures in the manufacture of reinforced concrete products, the technological capabilities assessment of all varieties of vibration tables with spatial oscillations of the moving frame is required.

Problem statement

The purpose of the article is to highlight the results of the mathematical modeling of oscillatory processes in the electric vehicle drive using mathematical software MathCAD and to determine the dynamic loads on its elements during transient processes in the electric vehicle drive.

Basic material and results

Considering the foregoing, the research of dynamic loads of electric vehicles on the basis of ZAZ-1102 car is conducted at the Department of Building Machines and Equipment Poltava National Technical University Yuri Kondratyuk. The transition processes taking place in the operation of the driving mechanisms of electric cars, largely determine the dynamic load in the elements of the systems are considered. The dynamics of the electric car starting and stopping mechanism processes are significantly influenced by the inertial and rigid parameters of the elements. Start and stop of the electric drive is performed at full load.

When considering the dynamic phenomena that arise during the startup of the electric car drive, in the main case of the system load conditions take the turn of the drive wheel with its maximum load. The calculation scheme of the electric motor drive mechanism loadduring the startup of the car driving mechanism is shown in Figure 1.

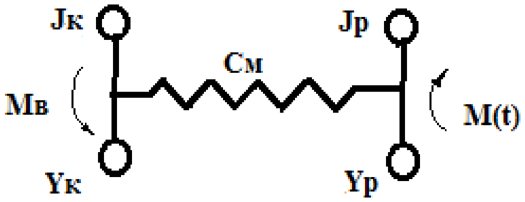

Figure 1 – calculation scheme of the electric motor drive mechanism loading during the startup of the car driving mechanism:

J_P – the electric motor rotor inertia moment, reduced to the axis of drive wheel rotation, considering the masses of rotating mechanisms;
J_K – total moment of inertia of rotating masses relative to the driving wheel rotation axis;
Y_P, Y_K – coordinates of the concentrated masses (angles of reference) motion;
S_M – stiffness of the drive elements are brought

Bringing all the masses of the drive, the stiffness of the elastic parts, and also the forces, the driving wheel rotation axis are made.

The equation of motion is:

$$J_P Y_P'' + C_M(Y_P - Y_K) + J_K(Y_P'' - Y_K'') = vM(t); \quad (1)$$

$$J_K Y_K'' - C_M(Y_P - Y_K) - J_K(Y_P'' - Y_K'') = -vM_{tr}, \quad (2)$$

where $M(t)$ – the moment of an electric motor, which is expressed by differential dependence [7]

$$M(t) = A_0 u + A_1 M'(t) + vA_2 u^2 Y_P(t) \quad (3)$$

A_0, A_1, A_2 – constant electric motor, defined by the expressions:

$$A_0 = 2M_K/S_K, \quad A_1 = 1/\omega_0 S_K, \quad A_2 = 2M_K/\omega_0 S_K \quad (4)$$

M_K – critical moment of the engine;
M_{tr} – moment that creates the total resistance of the electric motor and is brought to the the driving wheel rotation axis
S_K – critical slider of the rotor;
ω_0 – synchronous angular speed of the engine;
v – coefficient characterizing oscillation attenuation;
u – gear ratio of the drive mechanism;
t – time.

For the convenience of solving the equations (1) and (2) system, using the MathCAD software, it is denoted:

$$Y_P = z(t); \quad Y_K = q(t);$$
$$Y_P' = d(t); \quad Y_K' = n(t); \quad (5)$$
$$z'(t) = d(t); \quad q'(t) = n'(t)$$

Then it is got

$$d'(t) = -\frac{C_M}{J_p}z(t) - \frac{v}{J_p}d(t) + \frac{C_M}{J_p}q(t) + \frac{v}{J_p}n(t) + \frac{1}{J_p}M(t);$$
$$n'(t) = \frac{C_M}{J_K}z(t) + \frac{v}{J_K}d(t) - \frac{C_M}{J_K}q(t) - \frac{v}{J_K}n(t) - \frac{1}{J_K}M_{tr}; \quad (6)$$
$$M'(t) = -\frac{A_2 u^2}{A_1}z'(t) + \frac{1}{A_1}M(t) - \frac{A_0 u}{A_1}.$$

Initial conditions are presented in the form

$$t_0 = 0; \quad Y_{K0} = 0; \quad Y_{P0} = 0; \quad M_0 = 0 \quad (7)$$

Generalized technical characteristics of the electric motor and estimated parameters electric driven of the electric vehicles on the basis of ZAZ-1102 car are presented in the table 1.

Table 1 – Generalized technical characteristics of the electric motor and electric driven of the electric vehicles on the basis of ZAZ-1102 car

Parameter		Values of parameters
Type of electric motor		Siemens IP V5135-4WS14
Power of the electric motor, kBt		30
Frequency of the electric motor rotation, r/min		3000
Angle speed rotor of the electric motor, rad / s		314.2
Constant electric motor's	A_0	39470
	A_1	0.3183
	A_2	502.5
Moment of the electric motor rotor inertia J_P, N·m		0.475
Total moment of rotating masses inertia relative to the driving wheel rotation axis J_K, kg·m^2		0.082
Moment that creates the total resistance of the electric motor and is brought to driving wheel rotation axis M_{tr}, N·m		274.8
Gear ratio of the drive mechanism u		3.5
Stiffness of the drive elements are brought S_M, N·m/rad		15150
Coefficient characterizing oscillation attenuation v, м$^{-1}$		823150

Substituting generalized technical characteristics of the electric motor and estimated parameters electric driven of the electric car (Table 1) it is got

$$
\begin{aligned}
&z'(t) = d(t);\\
&d'(t) = 0.098M(t) - 1488z(t) - 80860d(t) +\\
&\qquad + 1488q(t) + 80860n(t)\ ; \qquad\qquad (8)\\
&q'(t) = n(t);\\
&n'(t) = 184800z(t) + 10038000d(t) - 184800q(t) -\\
&\qquad - 10038000n(t) - 3351\ ;\\
&M'(t) = -1670M(t) - 20160d(t) + 453600.
\end{aligned}
$$

As a result of the equations solution (8), the electric motor moment values are obtained, the angular displacement and speed of its rotor and the rotating masses of the drive and their angular accelerations for a given time t (Figures 2 – 7).

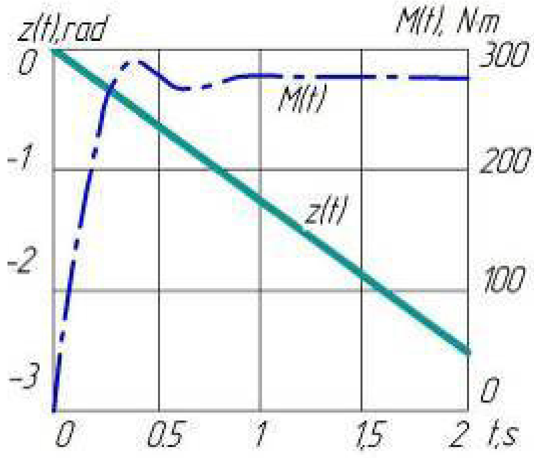

Figure 2 – The change dependences graphs of the angular displacement of the masses brought to the drive wheel rotation axis $Y_P = z(t)$ and the momentary change of the electric car motor drive of the car $M(t)$ as functions of time t

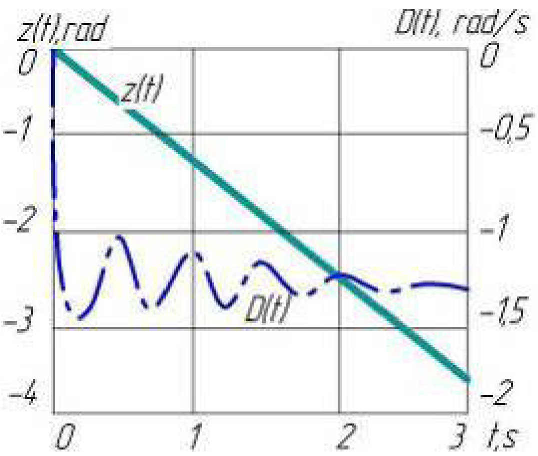

Figure 3 – The dependences graphs of the angular displacement change of the masses brought to the rotation axis of the drive wheel $Y_P = z(t)$ and the angular velocity change of the car electric motor drive $Y_r' = z'(t) = D(t)$ as functions of time t

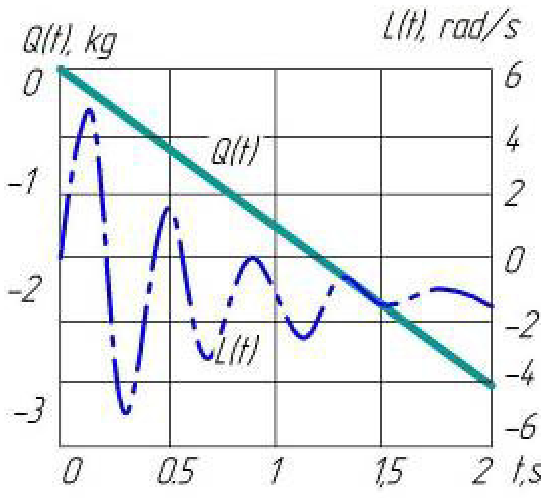

Figure 4 – Changing the angular displacement of the masses brought to the axis of rotation of the driven wheel $Q(t) = Y_k$ and their angular velocity $L(t) = Q'(t) = Y_к'$ as functions of time t

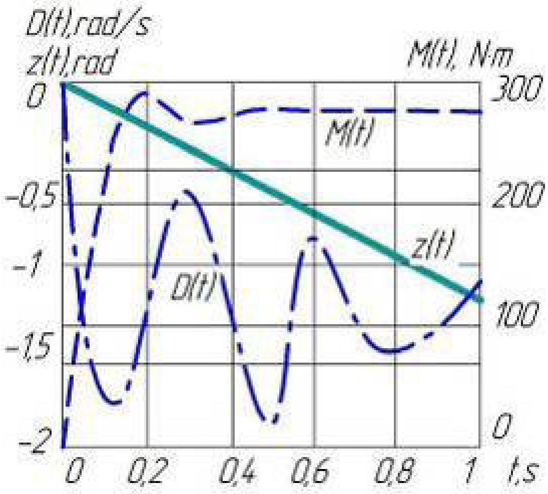

Figure 5 – Graphs of the change dependences of the angular displacement $Z(t) = Y_r$, angular velocity $D(t) = Z'(t) = Y_r'$ and the moment of the car electric motor drive $M(t)$ as functions of time t

To determine the accelerations in the system of differential equations (8) two equations of acceleration $K(t)$ of mass $Z(t) = Y_r$ and $W(t)$ of mass $Q(t) = Y_к$ are added

$$
\begin{aligned}
&K(t) = 2.105M(t) - 31899z(t) + 31899q(t)\ ; \qquad (9)\\
&W(t) = 184780z(t) - 184780q(t) - 3351.
\end{aligned}
$$

By solving the problem with the software program MathCAD, the system of equations (8) with the added equations (9), the values of accelerations change of the electric motor of the car electric motor drive $D(t)$ from time t are obtained.

As a result of the equations solutions(8 – 9), the values of acceleration of the rotor of the electric motor $K(t)$ (Figure 6) and the rotating masses of the drive $W(t)$ (Figure 7) are obtained.

According to the calculations, the built-up graph of the moment of an electric motor in the function of time (Figure 2) shows that the acceleration of the electric motor drive lasts about 0,8 seconds from the beginning of its motion. The maximum value of moment reaches at t = 0,4 s from the beginning of motion.

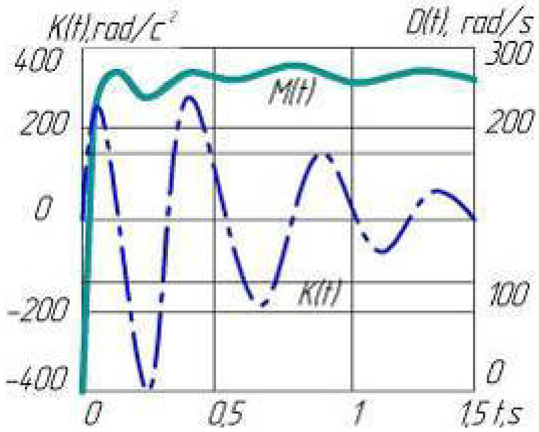

Figure 6 – Graphs of the dependences of the change of the angular acceleration of the electric motor *K(t)* rotor and the electric motor drive mechanism moment *M(t)* as functions of time *t*

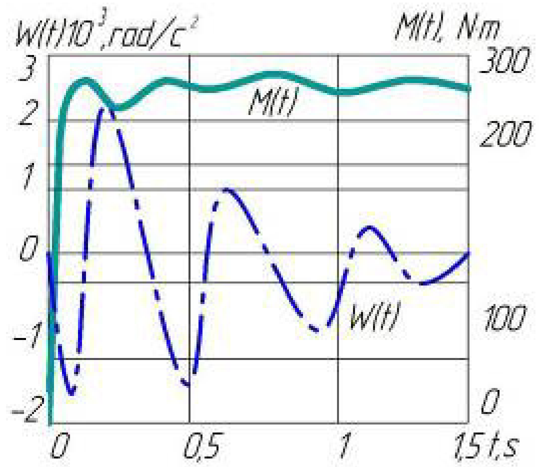

Figure 7 – Graphs of the dependences of the change of the rotating masses *W(t)* and the moment of the electric motor of the electric motor drive mechanism *M(t)* as functions of time *t*

Conclusions

1. The use of numerical methods for the integration of complex differential equations enables to use the proposed method for calculating dynamic loads in electric and mechanical systems of electric vehicles and their hybrids. In this case, the angular oscilational motion of the rotor of the electric motor becomes stable. Numerical calculations for researches of dynamics electric car on the basis of ZAZ-1102 have shown that considering the coefficient of oscillation attenuation ν almost does not affect the results of calculations.

2. The constructed graphs (Figures 2 – 7) of the dependencies of angular displacements, speeds and accelerations, the moment on the shaft of the electric motor rotor as functions of time indicate that since the car starting in its electric drive there are dynamic loads that are oscillatory in nature. By changing the moment of the electric motor and accelerating it is possible to determine the nature of the motion: constant (static) or oscillating (dynamic).

3. Graphs (Figure 2) of the dependences of the change of the masses angular displacement brought to the axis of rotation of the drive wheel Y_P=z(t) and the momentary change of the electric motor of the electric motor drive of the car M(t) as functions of time t in 0,4 s show that after the car start, the moment of the electric motor driving the electric vehicle takes the maximum value, and after 0,8 s the angular vibrational motion of the electric motor rotor becomes steady.

4. Graphs (Figure 5) of the change dependences of the angular displacement Z(t) = Yr, angular velocity D(t) = Z'(t) = Yr' and the moment of the electric motor of the car electric motor drive M(t) as functions of time t show that after 0.8 – 1.0 seconds the moment of the electric motor M(t) of the electric motor drive becomes equal to the static and the motion becomes even in character.

5. The study obtained results of the electric car drive mechanism, using the mathematical software environment MathCAD, can be used for designing, calculating and determining the dynamic loads of electric cars and their hybrids.

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Air velocity modeling velocity of the air around the trunk road train with installed rolling roof fairings

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The new rolling roof fairing installed in the tractor non-bonnet cab main train layout constructing are given. After analyzing existing exhausting elements installed on the tractor highway trailer and a large number of roof rails, it has been substantiated the expediency of installing a movable rope on the tractor roof, which can change the angle of the trailer airflow link with the combination of two movements - vertical and horizontal. The combination of these movements enables to change the parameters of streamline. For this purpose, the basic hydraulic control scheme is designed, which has a number of advantages: the starting units movements smoothness, the ability to continuously adjust the speed in a wide range, low inertia, simplicity of management and automation, high operational reliability and resistance to overload. Due to the modern capabilities and development of sophisticated electronic control systems through the introduction of such a system in the control process of hydraulic cylinders can ensure the system reliability, efficiency, ergonomics and safety equipment.

Keywords: main motorway, streamlined, roof fairings, hydraulic control scheme.

Моделювання швидкості руху повітря навколо магістрального автопоїзда із установленим рухомим даховим обтічником

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Наведено результати конструювання нового рухомого дахового обтічника, який установлюється на кабіні безкапотного компонування тягача магістрального автопоїзда. Після аналізу існуючих обтічних елементів, які встановлюються на тягач магістрального автопоїзда, та великої кількості саме дахових обтічників було обґрунтовано доцільність установлення рухомого обтічника на даху тягача, котрий може змінювати кут обтікання повітрям причіпної ланки за допомогою комбінації двох рухів – вертикального і горизонтального. Поєднання цих рухів надасть можливість найбільш просто змінювати параметри обтічності. Для цього спроектовано принципову гідравлічну схему керування обтічником, яка має ряд переваг: плавність рухів вихідних ланок, можливість безступінчастого регулювання швидкості у широкому діапазоні, мала інерційність, простота керування й автоматизації, висока експлуатаційна надійність та стійкість до перевантажень. Завдяки сучасним можливостям і розвитку складних електронних систем керування шляхом упровадження такої системи у процесі керування гідроциліндрами можна забезпечити надійність роботи системи, економічність, ергономічність та техніку безпеки. Таку гідравлічну схему можливо живити як від двигуна, так і від мережі живлення автомобіля. Керування можна виконувати автоматично, шляхом установлення гідророзподільника та регульованого дроселя до кабіни транспортного засобу і поєднання цих компонентів у єдиний блок управління. Побудована узагальнена тривимірна модель тягача, а також виконано графічне моделювання швидкості руху повітря навколо магістрального автопоїзда різного компонування із встановленим даховим обтічником та без нього за допомогою Microsoft Excel та функції Flow Simulation від SolidWorks.

Ключові слова: магістральний автопоїзд, обтічність, даховий обтічник, гідравлічна схема керування.

Introduction

The main trains are the main goods chain component transportation from the manufacturer to the final recipient because of their mobility and load capacity, as well as the branching of highways. It is not always possible for car dealers to buy a new tractor. The market of second-hand trucks in our country is developing; the old Soviet and post-Soviet trucks are being replaced at European level by large enterprises. For a potential buyer there is a number of estimated indicators – technical, functional, operational, and the optimal option is selected. One of the main performance indicators is fuel efficiency.

Since the main production feature of the main train is the speed of delivery to the destination, then the average technological speed at the same time is quite significant. It is proved that a significant part of the fuel consumed by a vehicle at high speed is needed to overcome the air resistance. Therefore, the decrease of the dynamic resistance of the motor-vehicle movement is the main factor affecting the final reduction of the cost of cost delivery that is the shortest way to obtain additional profits [1 – 3].

Review of research sources and publications

A large number of authors investigated various complex bodies flow processes, which include the main motorway [1 – 5].

The nature and level of traffic main train jams is determined by the shape, structural features and parameters of the air environment. The rectangular shape of the transverse and longitudinal section of the modern highway trains, in combination with flat walls, provides the most useful space for placing the cargo in them, but it is unsatisfactory from the aerodynamics view point. At the same time, in the case of on-board trucks, the main component of their frontal projection is the cab frontal area, then in the main road trains with high bodies, approximately the same size area above the cabin of the body front wall is added [4].

Analyzing the linear series of highway trains, it was determined that for them it is typical the presence of a body significant excess over the cabin, a large gap between them, as well as uncircumcised or rounded cabin and body frontal edge small radius [5]. All of these factors greatly reduce streamlinedness.

The author [4] considered in detail the complex main train flow mechanism with different layout schemes by counter and lateral wind. It has been proven that it has a negative impact on performance, namely aerodynamic stability, exchange rate stability, handling.

Definition of unsolved aspects of the problem

The authors [6 – 9], in theoretical studies and trains aerodynamic properties graphical modeling, simply install separate rails in different places, both the tractor cab and the trailer link. It is permissible at the stage of designing new technology, although there are enough domestic and foreign cars used on roads, for which it doesl not solve the issue. Among the large number of existing inventions, fender, located on the roof is the most commonly used; the side parts of the tractor remain the most effective. Its effectiveness is scientifically proven, but the application of one and the same ramp to different semitrailers, trailers, tanks causes a violation of the tractor aerodynamics. Such use is inappropriate and ineffective. In [9], the expediency of installing a very moving roof ramp, which enables to tune in to any link of the train.

Problem statement

The purpose of the article is to develop a basic scheme and design of the roof ramp, a hydraulic system for changing the roof ramp parameters with the adjusting possibility for any trailer structure, as well as possible options for connecting and adjusting its parameters, based on the previous studies results [8 – 9], as well as different layouts air movement speed graphical modeling around the main train with the installed roof rails.

Basic material and results

The overall height of the sidecar is determined by the trailer height couplings. The main resistance in them is created by air, which flows through the cabin and runs into the front wall of the semitrailer. In addition, in the intervals between the links of the automobile train powerful vortices are formed, which seem to increase the frontal area. Therefore, in various ways to reduce the aerodynamic resistance, various designs of the rails [6] are used which reduce the air resistance.

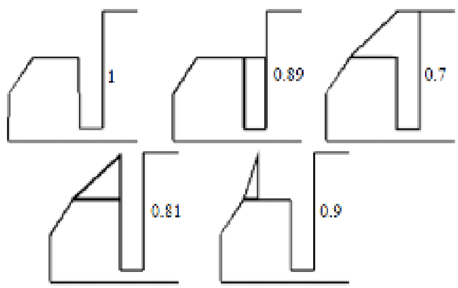


Figure 1 – Ways to improve the vibrancy of the trains by installing the roof rails

The flywheel is a necessary tool that enables to use the power of the truck most efficiently, as well as to save time and money.

Advantages of installing a strap on the roof of a truck for any trailer:

– helps to reduce the aerodynamic resistance of the oncoming airflow, and also increases the safety of traffic due to the road trains greater stability;

– provides high overstream rates due to the appropriate geometric parameters choosing possibility;

– provides ease of use and the ability to adjust the cushion when driving a car;

– enables to mount on any tractor and reinstall on other tractors without losing its technical characteristics and without changing overall dimensions;

– enables to achieve substantial fuel economy;

– enables to move faster without the cost of additional power providing superiority over competitors;

– reduces the load on the engine and reduces noise in the cabin resulting from the flow of air masses;

– improves aerodynamics and increases the car course stability and protects the semi-trailer body from wear, extending its service life;

– improves the truck outward.

The general view of the movable roof rails is shown in Figure 2.

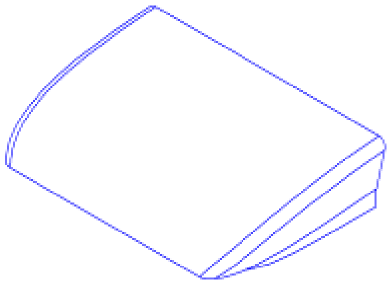


Figure 2 – General view of the proposed roof railing

In [11] for the purpose of determining and constructing a mathematical model, several tractors were processed and their average parameters are presented in Table 1.

Table 1 – The average parameters tractors

Height of cabscab	Length, mm	Width, mm	Height, mm	Engine power, kW	Fuel consumption, l / 100 km
Average	6200	2500	3500	300	19-21
Low	6200	2500	2900	300	20-22

For these tractors, the coefficient change of frontal op pv depending on the form and the site of the roof fairing has been investigated, which is presented in Table 2.

Table 2 – Drag coefficient changes on the parameters of the car and radome

No	Height cab	Fairings	Location	Drag coefficient C_x
1	Average	None	-	1.08
		Straight	Behind	0.77
		Improved		0.52
2	Low	Absent	-	1.09
		Normal	Behind	0.85
		Improved		0.57
		Improved	Ahead	0.87

The research results have been used when calculating the changes in the amount of fuel consumed by the flow of traffic. The relative savings of a tractor with a low cabin and an improved ramp make up 16.3% of the tractor without an outboard and 10.9% of the fitted with a conventional cushion. For an average cab with an improved ramp, the figures are 13.4% and 7.1% respectively [11]. These calculations point to the expediency of its use.

Of all the existing systems, it has been decided to choose a hydraulic control system because it provides:

– the movements smoothness of the output units;

– the possibility of stepless speed control in a wide range;

– small inertia;

– ease of management and automation;

– high operational reliability and resistance to overload.

The hydraulic scheme of the proposed control system is presented in Figure 3.

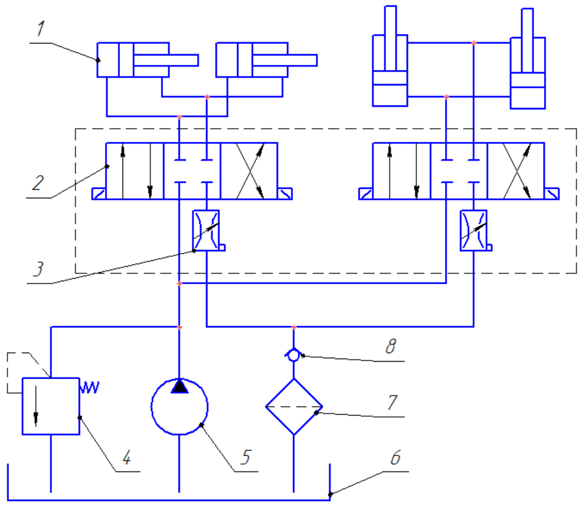


Figure 3 – Hydraulic system for changing the parameters of the roof rails for any trailer structure:
1 – hydraulic cylinder; 2 – hydraulic distributor; 3 – servo-choke; 4 – safety valve; 5 – hydraulic pump; 6 – oil tank; 7 – filter; 8 – check valve

The principle of this control system operation is that the liquid is absorbed from the oil tank by means of an unregulated pump with constant flow direction and fed to a hydraulic distributor with electromagnetic control. In the spool neutral position of the hydraulic distributor, when the pump is running, the pressure on the pipeline between the pump and the distributor begins to increase, with the safety valve triggered and the liquid is merged into the tank. When changing the position of the spool, open passage sections in the distributor and the liquid begin to enter the piston cavity of the hydraulic cylinder. From the rod cavity of the hydraulic cylinders, the fluid through the drainage line passes through a hydraulic distributor, regulated chokes with a servo drive and cleaned by a filter, fall into the drains in the tank.

The hydraulic cylinders rods translational velocity movement is controlled by the chokes. Reversal of the rods is carried out by switching the positions of the hydraulic distributor. In emergency stops (for example, an insurmountable effort), the pressure in the system increases, thereby causing the opening of the safety valve and the discharge of the working fluid into the tank.

Due to modern capabilities and the development of complex electronic control systems, the implementation of such a system in the process of managing the hydraulic cylinders can ensure the reliability of the system (long inter-repair period, control and shutdown during excessive pressure on the system, interconnection with the onboard computer, etc.), profitability (ensuring the operation of the equipment within the limits of rational parameters and operating modes), ergonomics (ease of use) and safety techniques (prevention of the occurrence of many types of injury during system operation).

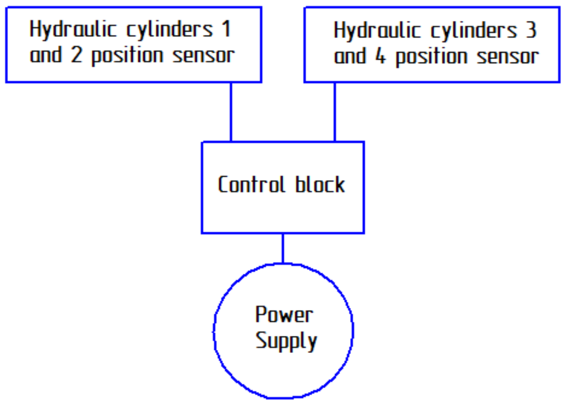


Figure 4 – Structural diagram of the hydraulic system for changing the parameters of the roof rails under any trailing structure

Such a hydraulic circuit can be fed, both from the engine and from the car power supply. Due to modern control electronics, it is possible to change the ramp position without leaving the tractor, by installing a hydraulic distributor and an adjustable throttle to the vehicle cab, and combining these components into a single control unit. Also, when connecting the control unit to the on-board computer, the system itself can set optimal parameters for the conditions, than reduce the human impact on the system and prevent the equipment from failing during strong wind gusts. Hydraulic cylinders sensors show a change in the stem departure and provide precise control of the change in the strapping element overall dimensions.

Main train car air speed graphic modeling around the different layouts with the installed roof rails.

The imaginary autotrain has been divided into the air flow zone with its flow in the projection of the road and the air velocity in these zones has been determined (Fig. 5).

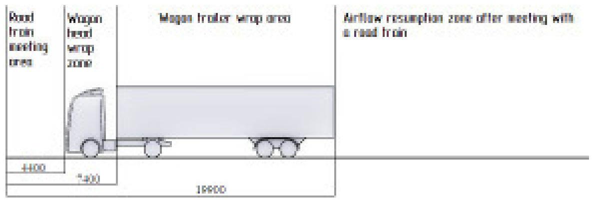


Figure 5 – Schematic representation of air flow zones when traversing the trains

With Microsoft Excel, the air flow around the main trains of different layouts has been simulated at 40, 60 and 90 km/h.

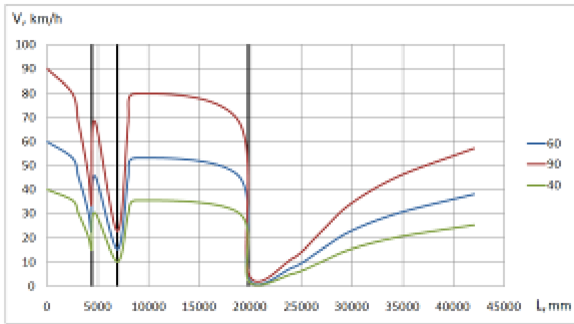


Figure 6 – Figure velocity of the air around the main road train low cab without a radome

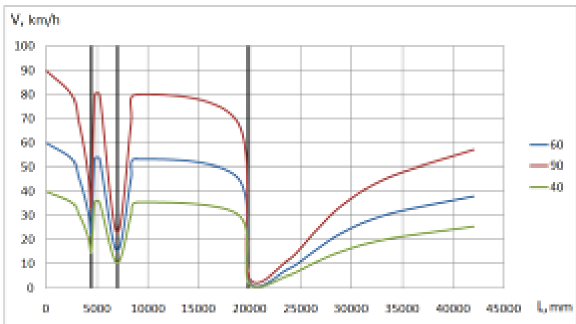


Figure 7 – Figure velocity of the air around the main road train with a low cabin and improved fairing front

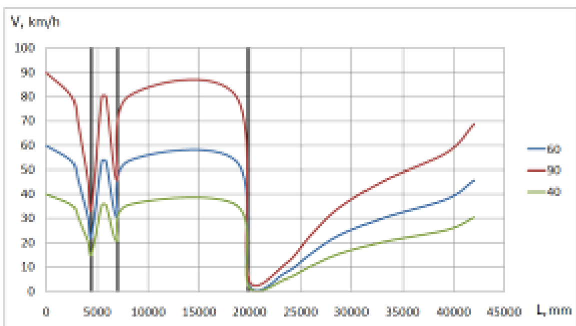


Figure 8 – Figure velocity of the air around the main road trains with low cab with improved fairing behind

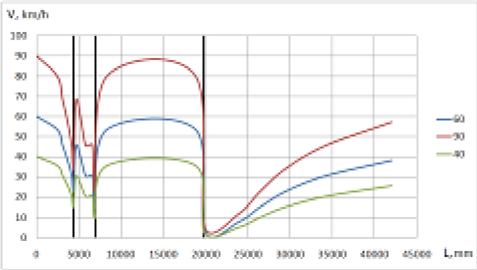


Figure 9 – Chart air velocity around the main road trains with low cabin with conventional fairing

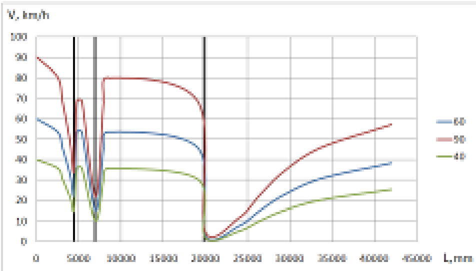


Figure 10 – Chart air velocity in Circle of main road trains with an average cabin without random

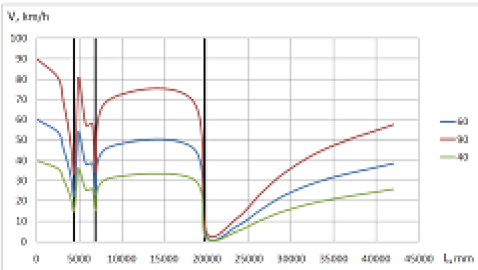


Figure 11 – Chart air velocity around the main road trains with an average cabin with conventional fairing

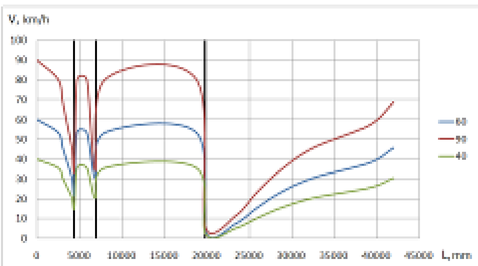


Figure 12 – Chart air velocity around the main road trains with an average cabin with improved fairing

Figure 6 – 12 shows a high-speed flow air different main road trains with and without roof railing. Vertical lines correspond to distances of 4400, 7400, 19900 mm, respectively. In front of any tractor, an excessive pressure is created which counteracts the car and tries to stop it, while in such a zone the speed of air movement decreases until it reaches directly the cabin of the car. After this, a sharp jump of speed corresponds to the self-poison head flow, and the reduction behind it - the gap between the tractor and the impudent warehouse. The distance from 6950 to 19800 corresponds to the trailer flap. After this, the airflow separation from the rear of the trailer occurs, which is evidenced by a sharp decrease in speed. The rest is the restoration of the pest air velocity through the car.

Creation of three-dimensional models is more labor-intensive process than construction of their projections on a plane, but in this case of the three-dimensional modeling there is a number of advantages, among which:

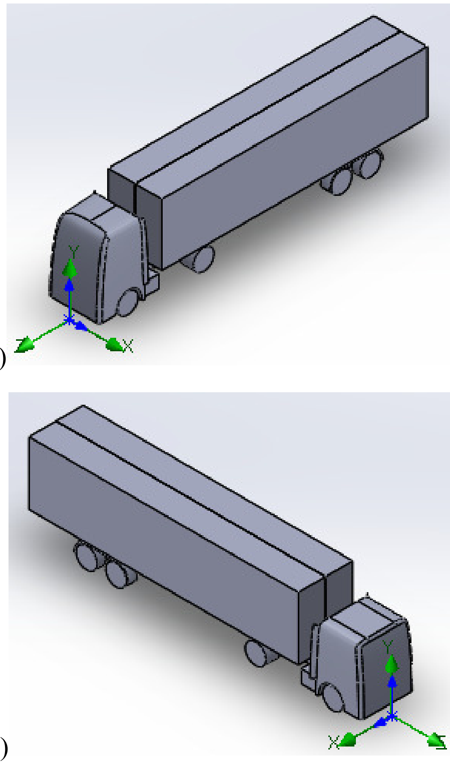
– the possibility of considering the model from any position;

– automatic generation of the main and additional species on the plane;

– verification of interactions;

– engineering analysis;

– extraction of the characteristics required for production.



a)

b)

Figure 13 – The appearance of the tractor:
a – cabin of average height; b – low-cabin

In the future, the Flow Simulation function has been used, which can simulate the flow of liquids and gases, control grid, use of typical physical models of liquids and gases, complex thermal calculation, gas, hydrodynamic and thermal models of technical devices.

With this package, the tractor traversing and flow velocity around it have been calculated.

The results of three-dimensional modeling correspond to the graphical dependences of previously calculated trains areas can be seen at Figures 14 – 20.

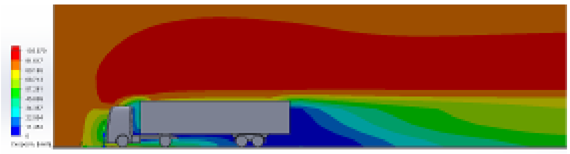


Figure 14 – Flow chart of the main auto train with the average cabin

Figure 15 – Flow chart of the main train with a low cabin

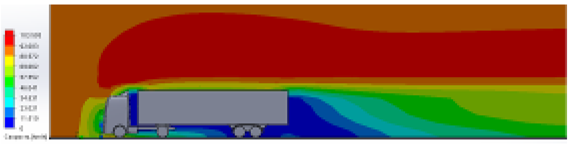


Figure 16 – Flow chart of the main auto train with the average cabin and ordinary racing

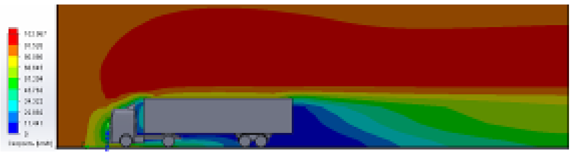


Figure 17 – Flow chart of the main train with a low cabin and ordinary ramp

Figure 18 – Flow chart of the main train with an average cabin and an improved ramp

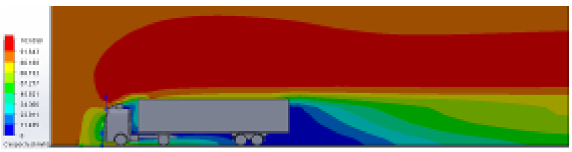


Figure 19 – Flow chart of the main train with a low cabin and a front upgraded ramp

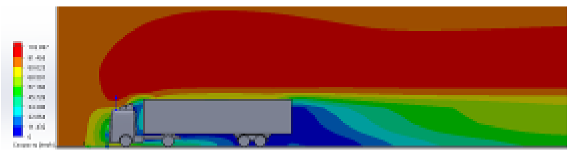


Figure 20 – Flow chart of the main train with a with a low cab and located behind the improved ramp

Conclusions

The study of the main train trail streamline enabled to achieve the following results:

– after an analysis of the existing flow elements installed on the main train trailer and the revised large number of rotors mounted directly on its roof, it was substantiated the expediency of installing a movable rope on the tractor roof that can change the trailer rotation angle link air through a combination of two movements – vertical and horizontal;

– the hydraulic roof railing changing the parameters system with the possibility of adjustment for any trailer structure is developed;

– suggested variants of roof ramp parameters adjustment connection;

– a generalized three-dimensional model of a tractor and an advanced ramp has been created for conducting experiments on it;

– a graphical simulation of the air velocity around the mainline trailer of a different layout with and without the roof railing installed using Microsoft Excel and SolidWorks Flow Simulation functions;

The next step is to create accurate models of the most common tractors and study the passage of such cars through the flow of air, both with and without the outboard, it is possible to create a layout to confirm or refute the data through computer simulation.

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High-strength steel grades application for silos structures

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This paper deals with the work studying of high strength steel in the constructions of silo capacities. The most widespread trademarks of steel have been analyzed. Mechanical and chemical features of examples series of high strength steel of European and American manufacturers have been experimentally tested, which were used for the body's sheet panels making. It has been made checking calculation of storage capacity with the diameter of 11 m from shaped corrugated sheet of different thickness. The material for panels manufacturing is one of the researched examples of steel. It was mentioned reasonable conclusions according to the using of the concerned material for getting economically rational project decisions.

Keywords: high strength steel, vertical silo capacities, thin-walled constructions, mechanical tests

Використання високоміцних сталей для конструкцій вертикальних силосних ємностей

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Дана стаття присвячена дослідженню роботи високоміцних сталей в конструкціях силосних ємностей. Аналізувалися найпоширеніші класи сталей, які застосовуються зарубіжними та вітчизняними виробниками елеваторного обладнання при проектуванні вертикальних циліндричних силосів для зберігання зерна. Надана оцінка впливу параметрів матеріалу на властивості окремих елементів споруди. Проведено комплекс механічних випробувань на розтяг та хімічний аналіз серії зразків високоміцної сталі класів S 550 GD та S 420 GD європейських та американських виробників, які застосовуються для виготовлення гофрованих листових панелей корпусу. Числові результати досліджених характеристик підтвердили повну відповідність матеріалу нормативним значенням відповідних класів. Продовженням даного дослідження було виконання перевірочного розрахунку ємності зберігання діаметром 11 м, яка має циліндричну форму та конічне днище і виконана з профільованих хвилястих листів різної товщини. В якості матеріалу виготовлення застосовувався один з досліджуваних зразків сталі. Були отримані значення критичних факторів для листів корпусу, вертикальних ребер жорсткості і листів конічного днища силосу. Результати розрахунків підтвердили ефективність застосування високоміцних сталей при проектуванні силосних ємностей для зберігання зерна. Несуча здатність елементів була забезпечена на всіх висотних ярусах, проте резерви несучої здатності при цьому виявилися мінімальні. Відповідно проведеного аналізу сформовані аргументовані висновки, щодо використання розглядуваного матеріалу для отримання економічно доцільних проектних рішень. Зазначені конструктивні обмеження, які необхідно враховувати при проектуванні та виготовлені тонкостінних конструкцій з високоміцних сталей. В першу чергу це стосується утворення отворів під болтові з'єднання, влаштування та обробку кромок і врахування граничних розмірів внутрішніх радіусів заокруглень при гнутті деталей.

Ключові слова: високоміцні сталі, вертикальні силосні ємності, тонкостінні конструкції, механічні випробування

Introduction

Modern construction of silo capacities is developing in two directions. On the one hand, there is the development of new conceptions of shaping and economic design [1, 2], on the other, the use of high-strength steels is intensively introduced into the practice of native production. It is known because the quality of the structural material significantly affects the structure reliability, durability, its technical and economic characteristics. It is likely that such innovations have only a positive effect such as reducing of the metalwork weight, improving of performance indicators and increasing freedom in choosing design solutions. However, this step requires the designer careful consideration of the work features of the silo capacity during construction and calculation, since the elements of high-strength steel with a conditional yield curve have an increased sensitivity to strain concentrators and are prone to brittle failure.

Review of research sources and publications

Theoretical and practical research in the field of calculation and design of the thin-walled shell constructions, in particular silo capacities, does not lose its popularity. It is possible to note significant scientific works of both Ukrainian and foreign scientists [1-10]. The estimation of the reducing the metal consumption of a silo construction from high-strength steel possibility was carried out in the previous work [11].

Definition of unsolved aspects of the problem

The main elements of the vertical cylindrical container are the body, consisting of individual sheets of smooth or wavy texture, vertical stiffeners, cone roof constructions and bottom. Silos body sheets perceive axially asymmetric and asymmetric radial loads, while the stiffeners work for compression with bend. It is clear that for the supportive thin-walled shell constructions more expedient to use a material with improved durability. However, there is the question, how argumentative is the use of steel with a conditional yield curve, and whether the high utilization rate of steel is advantageous in terms of ensuring the reliability and the constructions safety.

Problem statement

The main material for making modern constructions of the silo capacities storage is steel. There are a lot of demands of mechanical and technological character to the steel, which is used for making this type of construction. The features of performance of silo capacity elements need to provide not only conditions of durability and plasticity under the influence of corrosive environment, but also the possibility of putting welded joints, bending, cutting or drilling holes. It is likely that these properties depend on the chemical composition of steel. Therefore, it is important to analyze how the type of selected steel affects the constructive and rigid parameters of the construction.

Basic material and results

In accordance with the standards of our country [12], flat products for high strength steels after the heat treatment or thermomechanical rolling is usually limited to a range of values of characteristic resistance R_{yn} of 390...590 N/ mm^2 and by index of the plasticity (the relative elongation), which is not less than 16% and for the heat treated steel R_{yn} > 590 N/mm^2 (the relative elongation is more than 14%).

In the most cases, foreign and native developers of elevator equipment, in the practice of designing containers for grain storage, prefer steel of European and American manufacturers. This step is argued by their better quality and by the general increase of the useful life of the construction. In addition, all constructions of the silos are subject to significant corrosion processes, therefore the galvanization of steel is a necessary condition.

For example, the German company RIELA offers products with corrugated panels of S 550 GD+Z high-alloy steel in accordance with DIN EN 10346 (value of temporary resistance when stretching is 560 MPa), with a thickness of 0,50 mm.

A number of American agro-market leaders, Agri USA (silos CHIEF), use corrugated steel panels with a tensile strength of 483 MPa and a standard galvanized covering G 115 (350 g/m^3); silos of the GSI company have a corrugated steel profile with a minimum resistance of 450 MPa, and a coating of Zincalume (55% Al, 43.5% Zn and 1.5% Si), and the thickness of 4,2...5,2 mm or galvanized steel G 90; the silo capacities of MFS are constructed of steel with a strength of 482,6 MPa, thickness to the 4,166 mm and a zinc coating of standard G 15; SCAFCO products are characterized by the use of steel with a yield strength of 393 MPa; the silos of the American manufacturer VROCK have lateral segments of the galvanized steel G 90 with the strength of up to 65 psi (448 MPa).

The Spanish companies Symaga and Cordoba offer silos made of similar structural steel S 350 GD Z 600, galvanized with the Sendzimir method and with the innovative coating of ProMag, respectively.

The prevalent Canadian silo capacities of the AGI company are made from low-carbon low-alloy steel ASTM 653, Mark 50, Class 1 with a minimum tensile strength of 450 MPa. The technical characteristics of the body sheets of silos of the Westell company indicate the use of steel with a yield strength of 345 MPa (50 ksi).

The Italian manufacturer FRAME for sheets of side walls and stiffeners uses steel with thickness of 0,8...3,5 mm, a minimum zinc coating of 450 g/m^2 and a strength of 420 MPa and the marginal displacement of 350 MPa, and also makes it possible to apply steel of a higher class with zinc coating up to 600 g/m^2.

As for the Ukrainian manufactures of steel containers for grain storage, we have the following tendency. KMZ Industries uses high-quality stainless steel of S 350 GD class with the zinc coating of Zn 275 – Zn 600 from European manufacturers S SAB, Voestalpine, Wuppermann. The silos of the company

"Variant Agro Build" also design all the main elements from Austrian galvanized steel Wuppermann Stahl GmbH of the class S 350 GD and Z 350-M-A (zinc coating of 350 g/m or 450 g/m^2 and more). The “LUBNYMASH” elevator plant, which declares the production of a silo shell from galvanized steel (275 to 450 g/m^2) of the European production of the marks S 350 GD and S 550 GD, is not an exception.

Thus, it can be noted that, for the most part, manufacturers choose the steel for storage containers that have a temporary resistance of 400 – 500 MPa. Among the strengths listed above, the Belgian Steel Arcelor Mittal S 550 GD + Z is of the highest strength. The authors` team has already carried out the calculation of silo constructions with a diameter of 22 m from such steel, followed by a detailed analysis of the bearing capacity of the main elements [11]. The results showed that the use of steel of this trademark really enabled to reduce the thickness of the elements, but at the same time it caused a significant deterioration of the rigid characteristics of the vertical stiffening edges and increased deformability in the constructions of the roof and the bin-top gallery.

Check of the used material correspondence to the requirements of the standard UNSS EN 10346 [13] was this study continuation. A set of mechanical tests for the stretching of 6 standard flat examples of European steel S 550 GD (Fig. 1-a) provided by the "LUBNYMASH" company was made and their chemical analysis was carried out. The experiments were carried out on the basis of the certified laboratory of mechanical tests LLC "Etual-Metal".

a)

b)

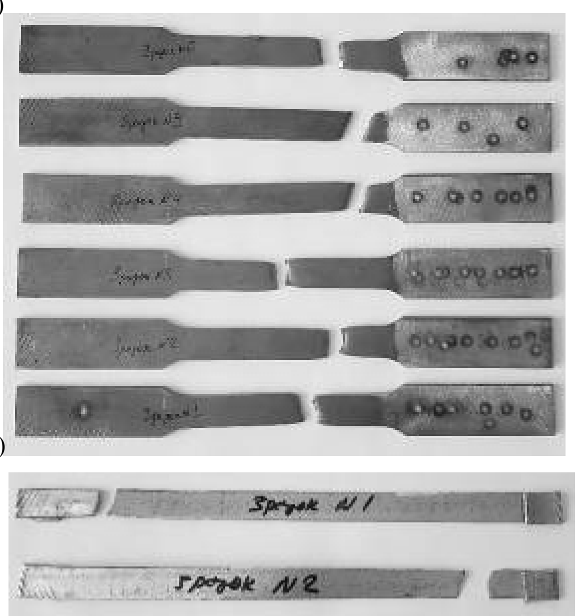


Figure 1 – Examples of the steel after the destruction:
a – European steel;
b – steel of the USA manufacture

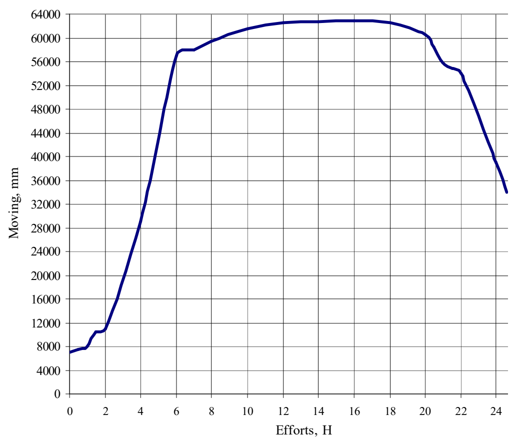
According to [13], the steel S 550 GD (steel number is 1.0531) has the following mechanical properties: the conditional yield curve is $R_{yn,0.2}$ = 550 MPa, the temporary resistance is R_{un} = 560 MPa, and the relative elongation is not given. Accordingly to the melting analysis, the chemical composition of the steel of S 550 GD class should be C = 0,20%, Si = 0,60%, Mn = 1,70%, P = 0,10%, S = 0,045%.

Mechanical tests on the static stretching were carried out at the hydraulic tensile-testing machine P-50 M2 (Armavir, Russia) with a built-in electronic remote control, accordingly to the GOST 1497-84 [14]. The chemical analysis of metal examples was carried out by using an optical emissive spectrometer Q2 ION (Bruker Elemental GmbH, Germany).

The numerical results of the features research (Table 1) showed that the provided steel examples fully correspond to the specified trademark

Table 1 – Check report of mechanical tests on the static stretching

The results f the research and the diagram of stretching (for the example №1)			
№of the examle	Max. efforts, H	Ultimate strength, Н/мм^2	Relative elongation, %
1	63489,299	793,616	8,223
2	**63242,549**	**790,532**	8,057
3	63686,457	796,081	8,687
4	63715,082	796,439	7,637
5	64203,062	802,538	**7,227**
6	63579,683	794,746	8,187

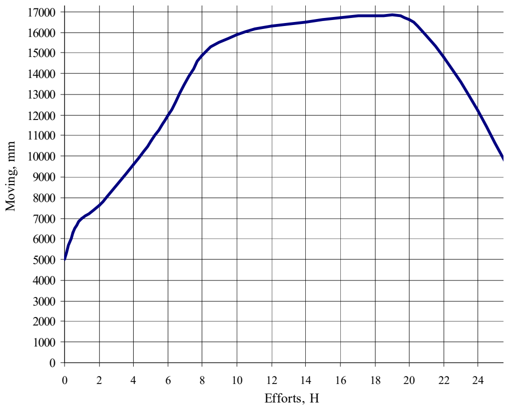


The second stage of the research was the similar tests of increased strength steel of S 420 GD class, accordingly to EN 10346:2015 of the USA manufacture, and used by the GSI company for the construction of vertical steel capacities for grain storage. The examples were made directly from the sheet of sheathing of the silos (Fig. 1-b). The numerical results of mechanical tests on the static stretching are given in the Table. 2.

It is important to note that, in accordance with the guidelines [12], the calculation of the body sheets and the vertical stiffeners of the silo capacities from the steel S 550 GD ($R_{yn,0.2}$ = 550 MPa, R_{un} = 560 MPa) must be made on the strength with additional coefficient of reliability, according to the material γ_u = 1.3 with the using of calculation resistance R_u, which is determined by the temporal resistance under the stretching R_{un}. Besides, when determining the calculation resistance R_u, it is necessary to consider the coefficient of reliability, accordingly to the material γ_m = 1.05, i.e. $R_u = R_{un} / \gamma_m$. Thus, the actual calculated characteristic of the silo element transverse section strength from the steel S550GD is not 560 MPa, but $R_u / \gamma_u \approx 410$ MPa.

Table 2 – Check report of mechanical tests on static stretching

The results f the research and the diagram of stretching (for the example №1)			
№ of the example	Max. efforts, H	Ultimate strength, H/мм^2	Relative elongation, %
1	**16764,448**	**827,874**	**8,481**
2	17103,400	844,612	9,428



Instead, the steel S 420 GD, according to UNSS EN 10346:2014 [13] has the following mechanical features: the conditional yield curve is $R_{yn,0.2}$ = 420 MPa, the temporal resistance is R_{un} = 480 MPa MPa, and the relative elongation is δ = 16%. Accordingly to the instructions [12], for steel with the characteristic resistance of $R_{yn} \leq 440$ MPa, the strength calculation is performed accordingly to the calculation resistance beyond the yield strength R_y and with considering the coefficient of reliability, accordingly to the material γ_m = 1.025. Thus, the calculation resistance is $R_y = R_{yn} / \gamma_m \approx 410$ MPa.

GSI silo testing, which has a cylindrical shape, a conical bottom and made of steel grade S 420 GD, has also been performed. Nominal diameter of the container is 11 m. Constructively the body shell consists of shaped wavy galvanized sheets with the height of 1165 mm (hereinafter, all geometric sizes were set by the measurements). The height of the silo consists of 14 panels. Their thickness has three standard sizes: 0,85 mm are I-VII tiers; 1,2 mm are VIII-XII tiers and 1,36 mm are XIII-XIV tiers. Separate sheets are connected with M10 bolts, which have sealing elements of the strength class 10.9. In the ring direction, the bolts are chequer. Total number of bolts per tier is 48 pcs. The outer stiffeners (Fig. 2) are made of a curved profile of a trapezoidal shape with 210 mm in width and 70 mm in height. The thickness of the transverse section of the rib varies within 1,2...4,2 mm and decreases in height. The conical bottom is formed of three layers of flat sheets with a thickness of 2 mm.

a)

b)

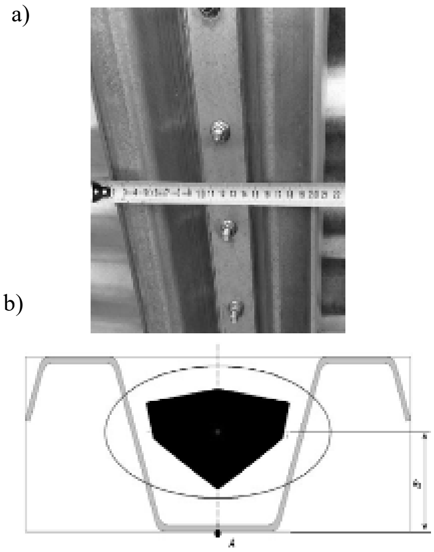
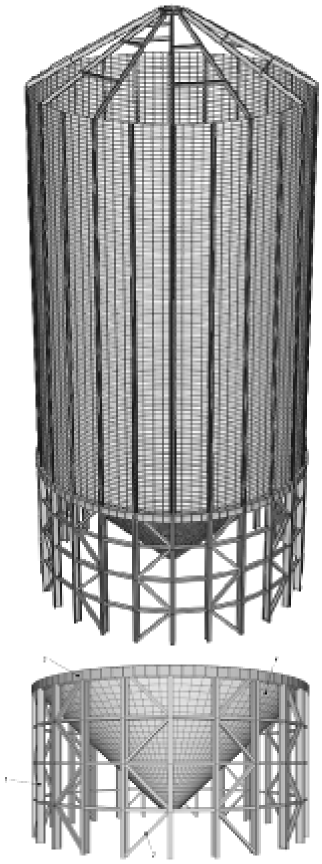


Figure 2 – Vertical stiffeners:
a – general view; b – transverse section with the orientation of the main axes

The mathematical) silo model (Fig. 3) was created in the finite element program complex SCAD 21.1 and included the following groups of constructive elements: 1 – pier column; 2 – braces and spacers of the pier column; 3 – supportive ring of the rigidity of the conical bottom; 4 – bottom sheets; 5 – vertical stiffeners; 6 – body sheets; 7 – elements of the conical roof. Each finite element of the model was characterized by the following main features: a dimension of the space used (one-dimensional, two-dimensional, three-dimensional); a set of nodes, which are located at the boundary between the elements and which are common for the boundary elements; a set of external and internal degrees of freedom in the nodes of the elements; a system of approximation functions.

The compilation correctness of the silos calculation model and the correspondence of calculation results with the actual work of structures at each step was controlled by the following tests: the dimension of the input and output quantities; the nature of the dependence of the result from the change of some input data, including the check of such properties as the expectation of symmetry (asymmetry) or insensitivity to some parameters; the system behavior under the extreme values of parameters; the observation of the conclusions, which are made from the theorems of reciprocity.

The calculation results have shown that the bearing strength of the body sheets (numerical data are given in the Table 3), bolted joints, stiffeners (Table 4) and other elements of the silo capacity is provided on all high-rise tiers, however, at the same time reserves of bearing strength are minimum: for body sheet of the 7th tier the critical factor is of 0.817; for the vertical stiffeners of the 12th tier the critical factor is of 0.973, for 13th is of 0.916, for 14th is of 0.983; for the sheets of the upper tier of the conical bottom the critical factor is of 0.71.



a)

b)

Figure 3 – Finite-element model of silo capacity:
a – general isometric appearance; b – the support part

Table 3 – Critical sheet factors of the body of silos

Tier	t_w, mm	N_{cr}, kN	σ_{cr}, kN/sm^2	K_R
1	0,85	58	6,8	0,208
...				
7	0,85	228	26,8	**0,817**
8	1,2	247	20,6	0,628
9	1,2	265	22,1	0,673
10	1,2	281	23,4	0,714
11	1,2	296	24,7	0,752
12	1,2	310	25,8	0,787

Symbols: t_w is the thickness of the body sheet of the silos at a given level;

N_{cr} is the value of the calculated annular longitudinal effort of force per unit of height from the horizontal pressures of bulk materials in the walls of round steel silos, considering additional efforts from temperature's drops;

$K_R = N_{cr} / (R_y\ t_w) \cdot \gamma_{neto} \le 1$;

$R_y = 41$ kN/sm^2 are a calculation resistance of steel beyond the yield stress;

$\gamma_{neto} = 1.25$ the coefficient of weakening of the transverse section by the holes of bolted joints, accepted on the basis of the analysis of technical documentation.

Table 4 – The bearing capacity calculation of the vertical stiffeners

Tier	t_w, mm	W_n, sm^3	N_m, kN	M_m, kN·sm	K_R
10	2,8	16,373	183	251	0,849
11	3,4	19,638	210	326	0,856
12	3,4	19,638	239	370	**0,973**
13	4,2	23,862	269	436	**0,916**
14	4,2	23,862	300	486	**0,983**

Symbols: W_n is a minimum moment of resistance relative to axis V (see the Drawing 2);

N_m, M_m are respectively, the longitudinal force and bending moment from the external load;

$K_R = \gamma_d\ (N_m / (A_p\ R_y\ \gamma_c) + M_m / (W_n\ R_y\ \gamma_c)) \le 1$ is the value of the critical factor;

γ_d is the coefficient of reliability of the model, which considers the uncertainty of the calculation scheme and other similar circumstances in the form of the constructions sensitivity to local destructions, to initial imperfections, to the joints compliance influence, to the plastic properties of the material, to the presence of dynamic effects or to idealizing of a material work diagram;

A_p is a cross-sectional area of the stiffeners;

$\gamma_c = 1.0$ is a coefficient of working conditions of the constructions.

Table 5 – Calculation of the bearing capacity of the conic bottom sheets

Tier	N_h, kN	N_T, kN	σ_h, MPa	σ_T, MPa	σ_Σ, MPa
upper	624	533	312	267	292
middle	461	370	231	185	212
bottom	270	204	135	102	122

Symbols: N_h, N_T are the boundary calculation values of longitudinal forces (correspondingly horizontal and along the formative) per unit length of the conic bottom section;

σ_h, σ_T are a strain in the sheets of the conical bottom in the ring and axial directions, respectively;

$K_R = \sigma_\Sigma / R_y$ is a value of the critical factor.

Another important factor is considering when designing thin-walled constructions of high strength steels is a number of constructive constraints, which are connected with the installation of holes under the bolted joints. According to the requirements [15], the holes for bolted joints must be formed by drilling, punching, gas-thermal or plasma cutting; thus the formation of holes using the method of punching is prohibited for steels with a yield strength of more than 350 MPa. Also, the steel part edges of class C 440 and above are subjected to planning or milling. According to the clause 9.14 [15], when bending parts of carbon steel on bending presses, internal radius of rounding are assumed to be not less than $1,2t$ for constructions that perceive static load. For parts made of low-alloy steel (constructive steel S 550 GD should be considered in this case as a low-alloyed), the boundary size of internal radius of rounding should be 50% higher than for carbon steel.

The last argument considered when choosing a steel trademark for the manufacture of a silos construction is the coefficient of material use $k_{yu} = R_y / R_u$, which characterized by the ratio of yield strength R_y to temporary resistance R_u (strength limit). For the steel trademarks considered, this value is almost equal to one $k_{yu}^{S550GD} \approx 0.98$ [11], $k_{yu}^{S420GD} \approx 0.88$. The reserve of bearing strength of the constructions made of high-strength steels with such a coefficient value k_{yu} is rather little. It should be considered that, first of all, the destruction of constructions is fragile (quick, we can also say, instant); secondly, the deflected mode of constructions is extremely dependent on the presence of strain concentrators in the form of holes, distortions of the elements shape (including bending), damages of the manufacture, i.e. any factors, which form the construction, that are unfavorable for flowing of the power flows.

Conclusion

1. Calculations of silo constructions made of high-strength steels confirm the high efficiency of this material when designing silo capacities for grain storage.

2. Checking calculation of the silo capacity elements, made of the S 420 GD steel, even when the thickness of flat products is approximate to 1 mm (the minimum thickness of the body sheets is 0,85 mm, of the stiffeners is 1,2 mm and of the bottom is 2 mm), is provided at every high-rise level, but the reserves of the bearing strength is minimal.

3. There are a number of constructive constraints that must be considered when designing thin-walled constructions of the high-strength steel. First of all, this applies to the requirements for holes, edges and bending of parts.

4. Experimental tests of the steel chemical composition and properties of European and American manufacturers showed full correspondence to the normative values of the respective trademarks. However, in spite of the improved strength characteristics of the S 550 GD steel in comparison with the S 420 GD, in accordance with the guidelines [12], the numerical values of the actual design characteristics of the transverse section elements of the silo capacity of both steel types must be chosen identically.

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Research of the specific steel shells progressive collapse prevention

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The article deals with coatings in the form of the specific steel shells. After a detailed analysis certain number of accidents and collapses, these collapses can be classified as “progressive” collapse. The main purpose of the article is the development of design algorithms for evaluation of the stress-strain state and preventing the progressive collapse of the specific steel shells. The method of prevention progressive collapse has been developed in the form of a constructive modernization. The comparative finite-element analysis of the strained-strain state of the specific shells original models, models of discrete-continual ribbed shells (with constructive upgrading) and models of solid ribbed shells has been carried out. From the analysis results it can be concluded that the proposed modernization method can be considered as one of the possible options for preventing progressive collapses and increasing the bearing capacity of specific steel shells.

Keywords: steel shell, arched thin-walled profiles, progressive collapse, buckling, open-type cylindrical compound shell.

Дослідження запобіганню прогресуючого обвалення металевих оболонок спеціального виду

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В статті розглянуті покриття та безкаркасні споруди у вигляді металевих оболонок спеціального виду. Основним конструктивним елементом цих спеціальних оболонок є аркові тонкостінні профілі холодного деформування, які виготовляються безпосередньо на будівельному майданчику. Розглянуті основні аномалії експлуатації зазначеного конструкцій зазначеного виду. Основна з означених аномалій експлуатації полягає у певній кількості аварій та обвалень, які після детального аналізу можна класифікувати як прогресуюче, або «лавиноподібне» обвалення. До основної мети статті відноситься розробка розрахункових алгоритмів оцінки напруженно-деформованного стану та заходів, що попереджують настання прогресуючого обвалення металевих оболонок спеціального виду. Розроблено спосіб запобігання прогресуючого обвалення, у вигляді конструктивної модернізації оригінального вузла з’єднання аркових конструктивних елементів. Проведено порівняльний скінченно-елементний аналіз напружено-деформованого стану, амплітудно-частотних характеристик та стійкості оригінальних моделей металевих оболонок спеціального виду, моделей дискретно-континуальних ребристих оболонок (з імплементацією конструктивної модернізації вузла з’єднання) та моделей континуальних ребристих оболонок. Результати аналізу подано у вигляді мозаїк вертикальних переміщень, головних стискаючих та розтягуючих напружень, порівняльної діаграми коефіцієнтів запасу стійкості та порівняльної діаграми значень частот власних коливань в залежності від номера форми власних коливань. Отримані результати чисельних досліджень, свідчать про ефективність запропонованої конструктивної модернізації металевих оболонок спеціального типу, в якості заходів, що попереджують настання прогресуючого обвалення зазначених оболонкових систем.

Ключові слова: металева оболонка спеціального виду, аркові профілі, прогресуюче обвалення, стійкість

Introduction

The considered coatings are based on arched structural elements that are combined with each other by a folded joint into a folded open-type ribbed cylindrical shell (Figure 1). The construction of structural elements takes place directly at the construction site, in addition, the mobility of the equipment (profile bending equipment) and transport compactness of the original construction material (coiled steel) enables to build objects for various purposes in the shortest possible time.

Figure 1 – The special type steel shell

Despite the frequency of use, the stress-strain state and the stability of the designated special coatings remain poorly considered. Anomalies and collapses that occur during this type of metal special shells operation are of considerable scientific interest.

Review of research sources and publications

These structures are considered in the works of Zverev V. [1] and Zhidkova K. [2] where arched structures are considered on the basis of volume-formed rolled metal. In the calculation of structures where the profiles have corrugated limits, the method based on the replacement of such faces is anisotropic with plates of similar thickness, which is widely used. Characteristics of rigidity which are found from the condition of equality of linear displacements of corrugated and flat anisotropic plates, along the respective coordinate axes, this simple but effective engineering method was proposed in the works of Andreeva L.Ye. [3, 4], where three types of corrugations are considered: trapezoidal, serrated and sinusoidal. In [5], the effect of virtual imperfections on the stress-strain state of the considered arch coverings was evaluated. The article [6] is devoted to defining the features of cold-formed trapezoidal arched profiles cross-sections work of the “IIA” type system as part of the coating shell. In [7], algorithms for calculating special type of arch covers have been added.

Articles [8], [9] provide information on the complexity of operation, a certain number of accidents and their probable causes for shell systems that are considered.

Definition of unsolved aspects of the problem

The most common causes of collapses appearing in expert opinions regarding an accident include:

– project errors;
– technological defects;
– violation of the exploitation rules.

Analyzing the collapse nature and rate, the so-called “progressive” or “avalanche” collapse occurs, in other words, a buckling of the described cylindrical ribbed shells occurred. In Fig. 3 shows a frame-by-frame fixation from a CCTV camera, the process of the arched structure collapse.

Problem statement

The main goal of the article is the development of computational algorithms for assessing the stress-strain state and measures that prevent the onset of the specific steel shells progressive collapse.

2017, The collapse of the granary (Ukraine)

2018, The collapse of the coating (Ukraine)

2014, The collapse of a warehouse (Russia)

2017, The collapse of parking structures (Belarus)

2018, The collapse of the granary (Ukraine)

2016, The collapse of the IceArena (Russia)

Figure 2 – Instances of the anomalies of exploitation of special type shells

Figure 3 – A 60 meter long structure collapsed in less than 10 seconds

Basic material and results

Establishing and ensuring the protection of buildings and structures from possible progressive collapse covers a wide range of approaches considering the nature and effects of various loads. At the same time, general recommendations are to use the simplest calculation methods that meet the requirements of construction, but the non-triviality of metal shell structures are considered, encourages their transformation and the used numerical models improvement.

The improved algorithm (and its verification) of these structures stress-strain state analysis reflect the possibility of progressive collapse, it is considered in the article that is now in-print entitled «Progressive collapse of the special-type arch systems: modeling algorithm» that has been written by the authors P. Reznik, L. Gaponova, S. Grebenchuk, R. Koreniev. A feature of this approach is to display the design nonlinearity of the specific metal shells and to use the potential deformation energy as a criterion for the carrying capacity exhauster 10] with the implementation of information technologies and approaches mentioned in [11]. The algorithm is generally illustrated in Fig. 4.

An example of the short shell model calculation results, with the implementation of the above principles, is shown in Fig. 5.

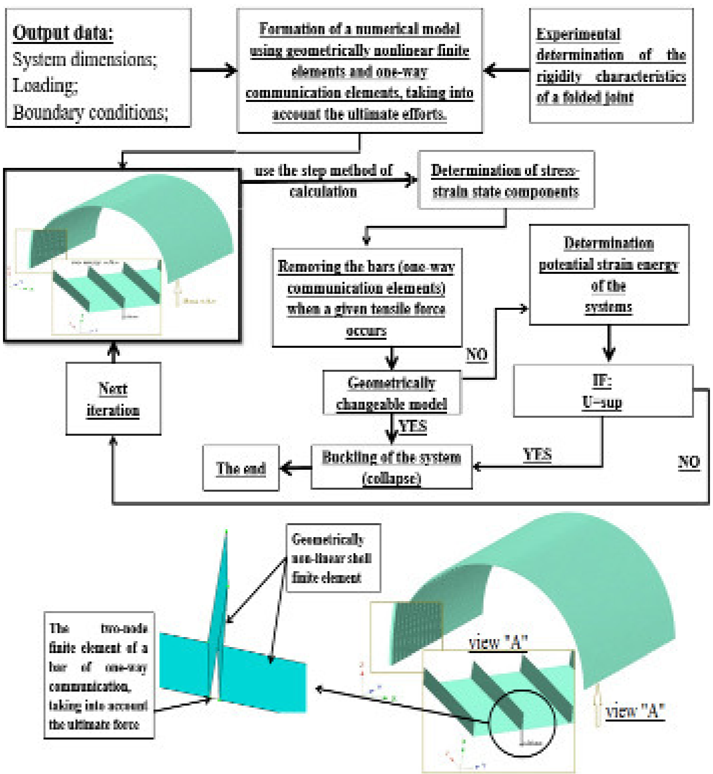

Figure 4 – The flowchart of the research stress-strain state algorithm of the arch-type system, considering the deformed scheme and probability of progressive collapse

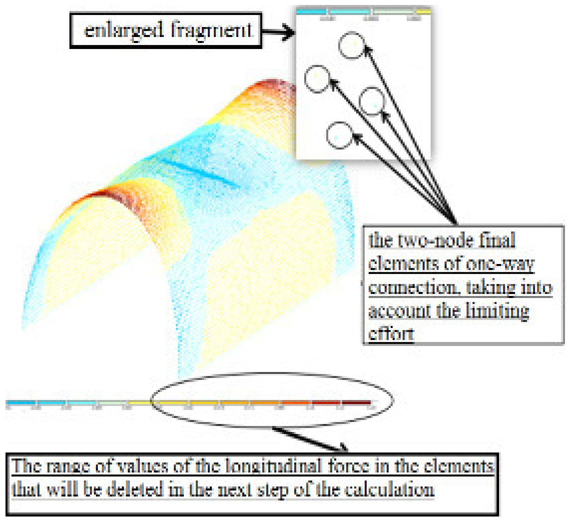

Figure 5 – An example of visualizing the values of the longitudinal force in the elements at the i- step of the calculation (the deformed scheme)

However, along with the need to consider the possibility of progressive collapse, methods of its prevention also remain the main open question. This is especially relevant for the specific systems which are, in fact, compound shells, as it has been noted earlier. However, it is necessary to ensure adequate commonality of the arched structural elements work. The proposed constructive implementation provides for the installation of bolted joints, as shown in fig. 6.

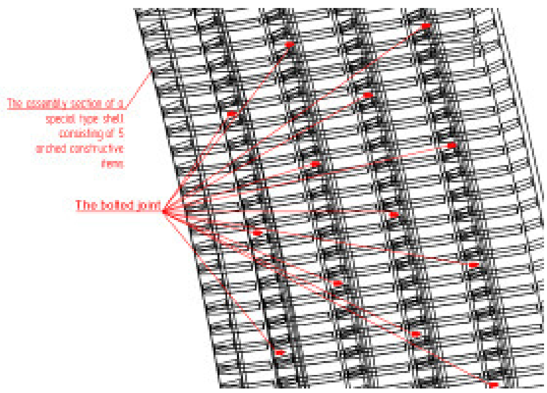

Figure 6 – The proposed constructive implementation of the arrangement of the bolted connection

This modernization enables to ensure the arched structural elements deformations compatibility, and to transform the stress-strain state of a specific composite shell into a stress-strain state close to a continuous ribbed shell, thereby obtaining a discrete-continuous ribbed shell. It is confirmed by comparing and analyzing the stress-strain state and stability of the computational models: a continuous ribbed shell, as well as the above-described discrete-continuous shell (using a bolted joint).

Consideration of qualitative and quantitative results has been carried out in the environment of PC "Lira" (license No. 1/6258), funded by the finite element method [12].

Researches have been conducted on the basis of accepted hypotheses and assumptions:

1. The principles of the shells classical theory are adopted, based on the Kirchhoff-Love hypotheses [13].
2. The torsion and displacements difference of the shell sides elements are not permissible and are not considered. According to the manufacturing technology, the supports are assumed to be articulated and fixed, that is, in the researched FE-models, linear displacements in the nodes of finite elements adjacent to the onboard element are prohibited.
3. Researches have shown that the end diaphragms of cylindrical shells are so rigid that in the majority of cases they can be considered to be non-deformable in their own plane, and from their own plane they are assumed to be absolutely flexible, that is, those ones that do not perceive perpendicular efforts [14]. In order to reduce the dimension of the finite element models under study, the simulation of diaphragm designs has not been performed, and according to the above, at the nodes of finite elements bordering the diaphragm, linear displacements in the plane of the diaphragm, that is, along the global X and Z axes, of the coordinate system of the used software complex have been prohibited.

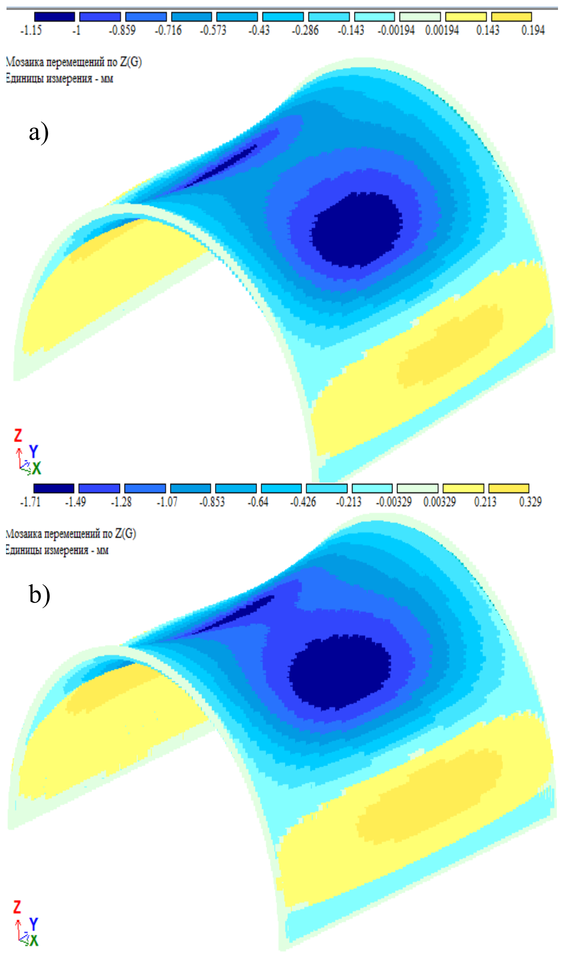

Figure 7 – Displacement along the vertical axis "Z":
a) FE model of a ribbed continual shell;
b) FE model of a modernized special type shell (bolted)

The models of the short shells [15], which have the following geometric ratios, boundary conditions, and features have been researched:

• L = D = 2R, with R = 11.63 meters, H = R, “R” is the radius; “H” is the camber of arch; “D” is the length of the shell;

• Shell thickness t = 1.2 mm;

• In the ribbed shell model, the thickness of the reinforcing rib is equivalent to the double thickness of the shell itself, that is, $t_r = 2t = 2.4$ mm.

• To simulate a bolted joint, the principle is shown in fig. 4, however, instead of the FE 255 (two-nodes finite element of one-sided bonds), the FE10 has been used (universal spatial rod ∅4 mm Stell 235).

• In places where the shell mates with the onboard element - a fixed hinge, that is, linear displacements along the X, Y, Z axes are prohibited in the used global coordinate system of PC "Lira";

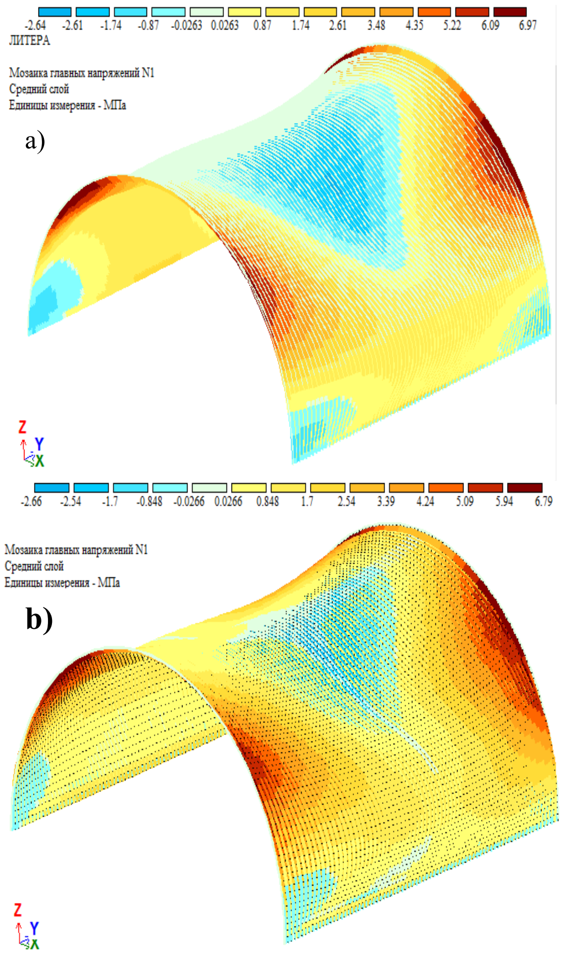

Figure 8 -- Mosaic of distribution the main stresses "σ_1" MPa:
a) FE model of a ribbed continual shell;
b) FE model of a modernized specific shell (bolted)

A uniformly distributed load has been applied, which is equivalent to 0.0015 MPa.

As a result of analyzing the components of the stress-strain state, at the Figures 7–9 it is shown the mosaics of vertical displacements distribution along the vertical (along “Z”), and also the mosaics of the main ones stretch and compress stresses σ1 and σ3.

For a more detailed analysis, for the researched shell models, as well as for the shell model, with similar dimensions and boundary conditions, but the one that reflects the possibility of progressive collapse, and does not contain constructive modernization (that is, using FE 252 - one-way connection) modal analysis and comparison of amplitude-frequency characteristics (AFC) have been done. The comparison of amplitude-frequency characteristics (AFC) of systems is illustrated in the graph of Fig. 10.

Since, as it is known, in most cases, the main criterion for the exhaustion of the shells bearing capacity is buckling. Buckling tasks are closely related to geometrically non-linear problems.

When using the stepping method of calculation, the buckling of the structure denotes the positive definiteness of the equations linearized system matrix. The condition of positive definiteness of a symmetric matrix, according to the Sylvester criterion, is the positivity of all its main minors, which is checked during the elimination of unknowns by the Gauss method [16].

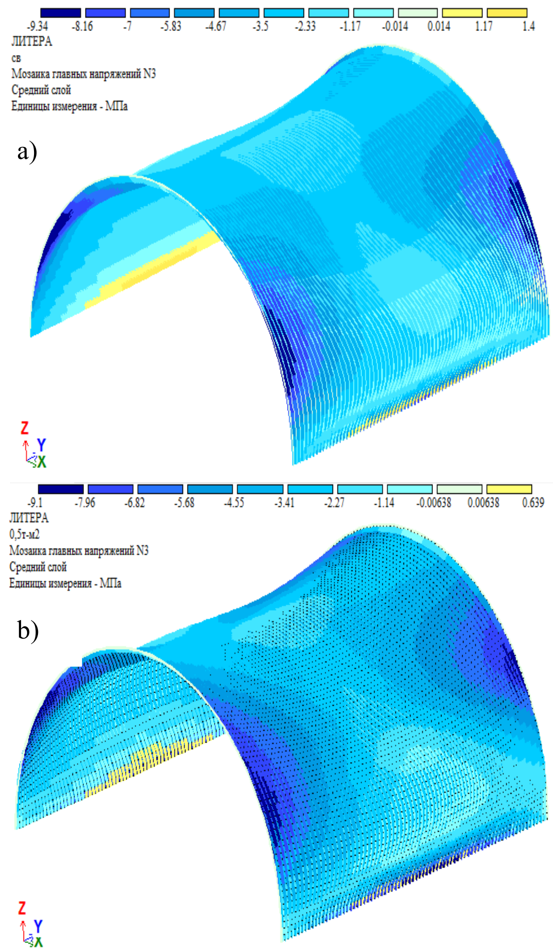

Figure 9 - Mosaic of the main stresses distribution"σ_3" MPa:
a) FE model of a ribbed continual shell;
b) FE model of a modernized specific shell (bolted)

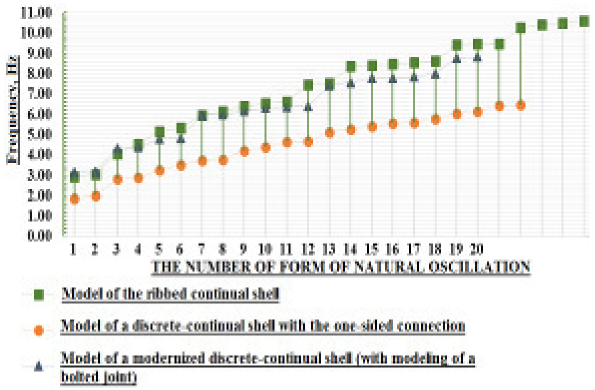

Figure 10 – Graph of the natural frequencies values, depending on the form number

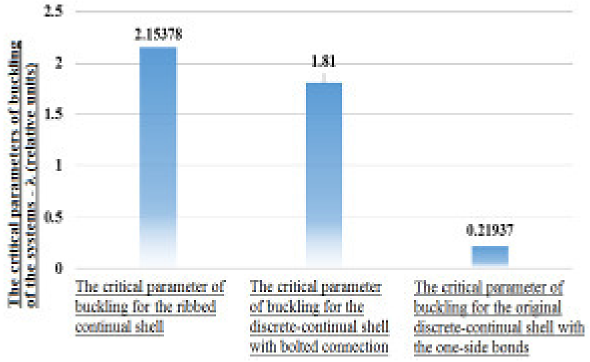

Figure 11 – Diagram of the critical parameters of the systems buckling - λ

This approach enables to research stability while simultaneously considering both geometric and physical nonlinearity. This method of stability research is called the deformed scheme stability method.

The main task of calculating the stability is to determine the critical parameter value of system buckling - λ. The physical meaning of the buckling critical parameter is that if the load on the structure is increased by a factor of λ, the system loses its stability. This version of the design stability assessment assumes that the distribution of forces/stresses is known from the solution of a linear static task and all external forces applied to the system (and as a result, internal forces/stresses) increase in proportion to one parameter λ. A buckling instability occurs when the reduction in force resistance is accompanied by the increase in displacement [17].

The main task is to determine the value of the numerical parameter λ so that at the external forces ($\lambda \times F_0$) there is a buckling. For the researched shells, the buckling factors are presented in the diagram in fig. 11.

Conclusions

After analyzing the vertical displacements of the ribbed continual shell and the modernized special type shell (with bolted connection), which are shown in Fig. 7, it is noted that the nature of the distribution and the value of the displacements have the same order. The value of tensile and compressive stresses "σ1" and "σ3" in the compared shells have almost identical distributions and the same order of value, Fig. 8-9. From the above, it can be concluded that the special type system obtained by constructive modernization (bolting installation) can be considered a discrete-continual ribbed shell while having a sufficiently adequate safety margin compared to other design models. This indicates the rationality of this approach. And the proposed modernization method enables to consider one of the possible options for preventing progressive collapse and increasing the bearing capacity of special type steel shells. As a development of this line of research, a full-scale experiment is planned to assess the impact of constructive modernization on the stress-strain state of discussed structures.

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Strength analysis of reinforced concrete flexural members at not entirely use of reinforcement resistance

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The suggestions for the engineering calculation of the reinforced concrete flexural members strength during the elastic work of steel are given. The engineering technique is developed on the basis of a nonlinear deformation model application using fractionally rational function for describing the process of concrete compressed area deformation and other prerequisites that are recommended by current norms for the reinforced concrete structures design. The analytical dependences have been obtained for determining the neutral axis depth and the calculation of the internal bending moment value perceived by the beam in the normal section. The task of determining the load-bearing capacity of the bending element is reduced to searching by the direct analytical dependences the maximum value of the bending moment that can be perceived by the beam at given strains of the most compressed rib of a cross-section.

Keywords: reinforced concrete, beam, strength, analysis.

Розрахунок несучої здатності залізобетонних згинальних елементів при неповному використанні міцності арматури

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У будівельній практиці подекуди застосовуються переармовані залізобетонні елементи, а також такі, в яких при багаторядному розташуванні арматури напруження не в усіх арматурних стержнях досягають межі текучості. Розрахунок міцності таких конструкцій є складним та трудомістким процесом, оскільки інженерної методики для такого випадку розрахунку на основі нелінійної деформаційної моделі не розроблено. Розроблення інженерної методики розрахунку несучої здатності цих залізобетонних згинальних елементів здійснено з урахуванням вимог чинних норм, розподіл напружень в бетоні стиснутої зони здійснено відповідно до діаграми «напруження – деформації» у вигляді дробово-раціональної залежності. Задача з розрахунку міцності в нормальному перерізі розглядається для балки прямокутного поперечного перерізу, армованого одиночною арматурою в момент досягнення бетоном на рівні найбільш стиснутої фібри таких значень деформацій, коли несуча здатність елемента буде максимальною. При цьому арматура в поперечному перерізі працюватиме з неповним розрахунковим опором. Отримані аналітичні залежності для обчислення висоти стиснутої зони бетону та значення внутрішнього згинального моменту, що сприймається балкою в нормальному перерізі. Розрахунок несучої здатності згинального елемента зведено до обчислення максимального значення згинального моменту, який може сприйняти балка при граничних значеннях деформацій найбільш стиснутої грані перерізу. Розглянуто також випадок розрахунку міцності згинальних елементів з багаторядним розташуванням розтягнутої арматури, при котрому напруження, які відповідають межі текучості, досягаються не в усіх рядах арматури. Застосування дробово-раціональної залежності для описання процесу деформування бетону стиснутої зони згинальних елементів дало змогу суттєво спростити та наблизити до інженерного розв'язку задачу з визначення несучої здатності елементів з неповним використанням міцності розтягнутої арматури.

Ключові слова: залізобетон, балка, міцність, розрахунок.

Introduction

Design of reinforced concrete members and structures on the basis of nonlinear deformation of materials more accurately reflects the actual work of materials and is recommended by the current norms [1, 2]. Practical methods for solving problems of the reinforcement selection, and determining the reinforced concrete elements strength have been developed on the basis of this approach. In practice there are sometimes overreinforced elements or those where, when multi-row arrangement of tensile reinforcement, stresses reach the yield strength not in all reinforcing bars. Determining the strength of such structures is a much more complicated calculation process, since the engineering methodology for such a calculation case is not developed. Even the strength calculation of rectangular section reinforced concrete beam cannot be performed without special computer programs, where the physical essence of the process is closed from the designer, which deprives him of understanding the design theoretical foundations.

Review of research sources and publications

Proposals for engineering calculations of the bending reinforced concrete elements strength are given in works [3 – 8]. In particular, in the publications [3 – 5], these proposals are implemented on the basis of the fractional-rational "stress-strain" dependence in the concrete compressed area use. In [6] examples of calculating the strength and determining the area of reinforcement on the basis of the reinforced concrete resistance concept are considered. In the article [7] methods of the problem engineering solution of determining the longitudinal reinforcement area of the bending elements is given. All of the above proposals are obtained only for elements with entire use of the reinforcement strength. In the paper [8], the problem of determining the strength of both normal and overreinforced reinforced concrete elements is proposed to be solved on the basis of the application of a uniform stresses distribution in the concrete of the compressed area. Publications [9 – 14] are devoted to the development of methods for calculating the strength of any reinforced concrete elements based on the deformation model in the general case and are proposed for implementation only with the use of modern computer programs.

Definition of non-solved aspects of the problem

Thus, in the theory of structural analysis of reinforced concrete elements a gap was created, which concerns the lack of engineering calculations of elements with reinforcement in an elastic stage.

Problem statement

Consequently, the development of an engineering method for calculating the carrying capacity of reinforced concrete bending elements with partial use of the reinforcement strength is an urgent task.

Basic material and results

The solution of the problem is carried out according to the design scheme depicted in Figure 1. In this case, the preconditions for calculating according to the norms [1, 2] are considered. In the scheme, the function-approximation of the "strain-strain" diagram in the form of fractional-rational dependence (3.4) from [1] was used in the image of the curvilinear diagram of the stresses distribution in the concrete compressed area.

The problem of calculating the strength is considered for a single reinforced beam of rectangular cross-section (Fig. 1) at the time when the most compressed concrete fibers reaching the level of such strain values, when the carrying capacity of the element is maximal. In this case, the reinforcement in the cross section works with partial design strength, i.e. $\sigma_s < f_{yd}$.

In the task, it is taken as known values: the area of the longitudinal reinforcement A_s in the cross-section, the dimensions of the cross-section of the beam $b \times h$, the physical and mechanical characteristics of the concrete f_{cd}, E_{cd}, ε_{c1} and reinforcement f_{yd}, E_s. For the considered beam it is determined by the engineering method [5], that $\xi > \xi_R$.

Unknown values are the maximum value of the resisting moment M_{Rd}, which can be perceived by the beam, and the corresponding value of strains in the most compressed concrete fibers $\varepsilon_{c(1)} = \varepsilon_{cu}$ (its level $\eta = \eta_u$), at which the beam maximizes resistance to the external load.

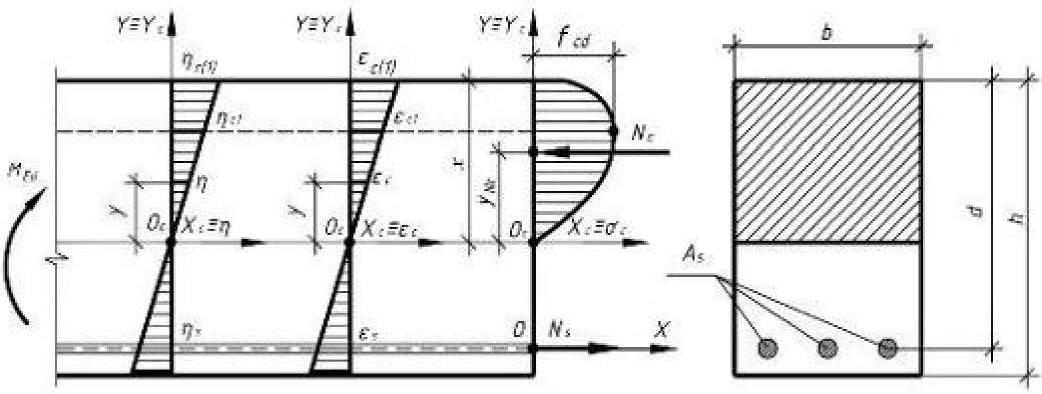

Figure 1 – Design scheme of the reinforced concrete beam

To solve the problem according to the accepted design scheme (Fig. 1) it is used:

– equation of equilibrium:

$$\sum X=0;\ N_s-N_c=0, \quad (1)$$

$$\sum M_O=0;\ M_{Ed}-N_c(d-x+y_{Nc})=0, \quad (2)$$

where N_s, N_c – resultant forces respectively in reinforcement and concrete;

d, x, y_{Nc} – the effective depth of the cross-section, the neutral axis depth, the distance from the neutral axis to the point of force N_c application;

– the "stress-strain" diagram for concrete at the axial compression according to [1]

$$\sigma_c=\frac{f_{cd}\,(k\eta-\eta^2)}{(1+(k-2)\eta)}, \quad (3)$$

where $k=1{,}05(E_{cd}\,\varepsilon_{c1,cd}\,/\,f_{cd})$, $\eta=(\varepsilon_c/\,\varepsilon_{c1,cd})<k$;

f_{cd}, E_{cd} – accordingly, the design values of concrete strength at axial compression and its elastic modulus;

– condition of strains compatibility of concrete and reinforcement

$$\varepsilon_c=\varepsilon_s; \quad (4)$$

– the "stress-strain" diagram for reinforcing steel in tension (compression)

$$\sigma_s=E_s\varepsilon_s \ \text{ at }\ 0<\varepsilon_s\le f_{yd}\,/\,E_s; \quad (5)$$

$$\sigma_s=f_{yd} \ \text{ at }\ f_{yd}\,/\,E_s<\varepsilon_s\le\varepsilon_{ud}; \quad (6)$$

– criterion for the maximum strength of the cross section of the beam element

$$M_u(\varepsilon_{cu})=M_u=\max M(\varepsilon_{c(1)})$$

or

$$M_u(\eta_u)=M_u=\max M(\eta), \quad (7)$$

in which the ultimate (characteristic) value of the compressive strain in the concrete (or its level) satisfies the condition of the extreme strength criterion of this section in the beam [2, p. 4.1.1], thereby ensuring, in the general case, the duality of the beam element strength problem solution in cross section.

To solve the problem, it is necessary first to express the components N_s, N_c, x and y_{Nc} in equations (1) and (2) functionally through σ_s, x, $\varepsilon_{c(1)}$ or through σ_s, x, $\eta_{c(1)}$.

To achieve this goal there is a need to have the law of stress distribution in the concrete of the compressed area in the form $\sigma_c=f(y,\ \eta_{c(1)},\ldots)$. Such a law describes the distribution of stresses in the concrete of the compressed area in the coordinate system $Y_cO_cX_c$ with its start on the neutral axis (Fig. 1), is obtained as follows

$$\sigma_c\left(y,\eta_{c(1),\ldots}\right)=\frac{f_{cd}\eta_{c(1)}y\left(kx-\eta_{c(1)}y\right)}{x\left(x+(k-2)\eta_{c(1)}y\right)}, \quad (8)$$

where y – the current coordinate;

x – the neutral axis depth.

Applying the dependence (8), the components of equations (1) and (2), after performing the necessary mathematical actions, are given to the expressions:

$$N_c=b\int_0^x\frac{f_{cd}\eta_{c(1)}y\left(kx-\eta_{c(1)}y\right)}{x\left(x+(k-2)\eta_{c(1)}y\right)}dy=f_{cd}bx\omega\left(\eta_{c(1)}\right); \quad (9)$$

$$y_{Nc}=S_c\,/\,N_c=x\frac{\varphi\left(\eta_{c(1)}\right)}{\omega\left(\eta_{c(1)}\right)}; \quad (10)$$

$$S_c=b\int_0^x\frac{f_{cd}\eta_{c(1)}y\left(kx-\eta_{c(1)}y\right)y}{x\left(x+(k-2)\eta_{c(1)}y\right)}dy=f_{cd}bx^2\varphi\left(\eta_{c(1)}\right); \quad (11)$$

$$\left.\begin{aligned}&\omega\left(\eta_{c(1)}\right)=\frac{(k-1)^2\left(c-\ln c-1\right)}{(k-2)^3\,\eta_{c(1)}}-\frac{\eta_{c(1)}}{2(k-2)}\ \text{при}\ k\ne2,\\&\omega\left(\eta_{c(1)}\right)=\eta_{c(1)}(1-\eta_{c(1)}/3)\ \text{при}\ k=2,\end{aligned}\right\}; \quad (12)$$

$$\left.\begin{aligned}&\varphi\left(\eta_{c(1)}\right)=\frac{(k-1)^2\left[(c-2)^2+2\ln c-1\right]}{2(k-2)^4\,\eta_{c(1)}^2}-\frac{\eta_{c(1)}}{3(k-2)}\ \text{при}\ k\ne2,\\&\varphi\left(\eta_{c(1)}\right)=\eta_{c(1)}\left(\frac{2}{3}-\frac{\eta_{c(1)}}{4}\right)\ \text{при}\ k=2,\end{aligned}\right\}, \quad (13)$$

where $\omega(\eta_{c(1)})$ – the coefficient of the stress diagram fullness in the concrete compressed area, as it can be seen from formula (9).

Formulas (12) and (13) are derived when developing the expressions (9) – (11), while the notation was made $c=1+(K-2)\eta_{c(1)}$.

After the substitution of the values of N_s, N_c, x and y_{Nc} expressed in terms of $\eta_{c(1)}$ into (1) and (2), with $\sigma_s<f_{yd}$, the latter acquire the form:

$$E_s\varepsilon_sA_s-f_{cd}bx\omega=0; \quad (14)$$

$$M_{Ed}-f_{cd}bx\omega(d-\chi\omega x)=0, \quad (15)$$

where coefficient

$$\chi=\frac{\omega-\varphi}{\omega^2}, \quad (16)$$

shows which part of the depth x the distance from the most compressed fibers to the point of application of the resultant N_c constitutes.

Based on the hypothesis application of strains linear distribution, an expression is used to determine the deformations of the tensile reinforcement

$$\varepsilon_s=\frac{\varepsilon_{c(1)}(d-x)}{x}. \quad (17)$$

After substitution (17) into the first equation of equilibrium (14) a formula for the direct determination of the neutral axis depth is obtained

$$x=-B+\sqrt{B^2+2Bd}, \quad (18)$$

where

$$B=\frac{\varepsilon_{c(1)}E_sA_s}{2f_{cd}b\omega}. \quad (19)$$

Thus, the problem of determining the strength is reduced to finding the maximum value of the bending moment that can be perceived by the beam at the predefined deformations $\varepsilon_{c(1)}$ of the most compressed rib from the second equation of equilibrium

$$M_{Rd}=f_{cd}bx\omega(d-\chi\omega x). \quad (20)$$

In practice, bending elements with multi-row arrangement of tensile reinforcement are often used. In this case, the stresses in the reinforcement of different rows may reach the yield strength or not. For this case, the design scheme acquires the form shown in Fig. 2. Equation for equilibrium for the considered design scheme (Fig. 2) is written as follows:

$$\sum X=0;\ N_{si}+N_{sj}-N_c=0, \quad (21)$$

$$\sum M_A=0;\ M_{Ed}-N_{si}(d_i-x+y_{Nc})-N_{sj}(d_j-x+y_{Nc})=0, \quad (22)$$

in which N_{si}, N_{sj} – the resultant forces in the tensile reinforcement, the stress at which $\sigma_s=f_{yd}$ and $\sigma_s<f_{yd}$, respectively;

d_i – the distance from the most compressed border of the cross-section to the i-th row of reinforcement, the stress of which reaches the yield strength ($\sigma_s=f_{yd}$);

d_j – the distance from the most compressed border of the cross-section to the j-th row of reinforcement, where the stress does not reach the yield strength ($\sigma_s<f_{yd}$).

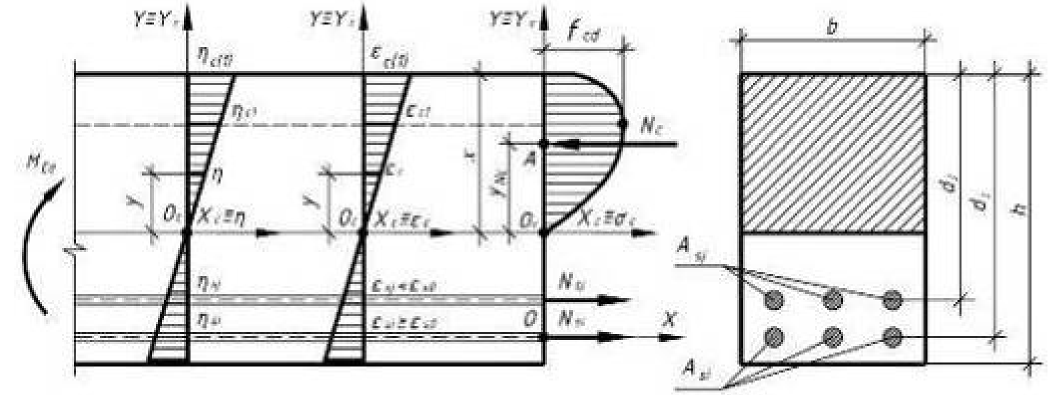

Figure 2 – Design scheme of the reinforced concrete beam with multi-row arrangement of tensile reinforcement

After substituting the values of N_{si}, N_c, x and y_{Nc} into equations (1) and (2) with $\sigma_s<f_{yd}$, the latter acquire the form:

$$E_s\sum_{i=1}^{n}\varepsilon_{si}A_{si}+f_{yd}\sum_{j=1}^{k}A_{sj}-f_{cd}bx\omega=0\,, \quad (23)$$

$$M_{Ed}-\varepsilon_{c(1)}E_s\sum_{i=1}^{n}(d_i/x-1)A_{si}(d_i-\chi\omega x)- \\ -f_{yd}\sum_{j=1}^{k}A_{sj}(d_j-\chi\omega x)=0, \quad (24)$$

where n – the number of reinforcement bars rows, where the stresses do not reach the yield strength;

k – the number of reinforcement bars rows, where the stresses reach the yield strength.

After substitution (17) into the first equation of equilibrium (23) a formula for the direct determination of the neutral axis depth is obtained

$$x=-B+\sqrt{B^2+\frac{\varepsilon_{c(1)}E_s\sum_{j=1}^{k}A_{sj}d_{sj}}{f_{cd}b\omega}}\,, \quad (25)$$

where

$$B=\frac{\varepsilon_{c(1)}E_s\sum_{i=1}^{n}A_{si}-f_{yd}\sum_{j=1}^{k}A_{sj}}{2f_{cd}b\omega}\,. \quad (26)$$

Thus, in the same way as in the previous case, the problem of determining the strength is reduced to finding the maximum value of the bending moment that can be perceived by the beam at the predefined strains $\varepsilon_{c(1)}$ of the most compressed border of cross-section from the second equation of equilibrium

$$M_{Rd}=\varepsilon_{c(1)}E_s\sum_{i=1}^{n}(d_i/x-1)A_{si}(d_i-\chi\omega x)+f_{yd}\sum_{j=1}^{k}A_{sj}(d_j-\chi\omega x). \quad (27)$$

In this case, the iterative search should begin with the value of strains $\varepsilon_{c(1)}=\eta_u\varepsilon_{c1}$, where η_u is the limiting deformation level of the most compressed fibre of the concrete in [5], which corresponds to the destruction of the element with the full use of the reinforcement strength, that is, when reinforcement reaches the stresses f_{yd}. Increment deformation at each step is $\Delta\varepsilon_{c(1)}=0{,}1\varepsilon_{cu1}$.

Typically, with $\Delta\varepsilon_{c(1)}=0{,}1\varepsilon_{cu1}$, results are obtained with sufficient accuracy, but if it is necessary to improve the accuracy of the calculation, smaller values for deformation growth at each step can be used.

The search should be carried out to the value of the strains $\varepsilon_{c(1)}$, where the value of the bending moment M_{Rd} determined from (20) or (27), depending on the calculation case, begins to decrease, thus, it is smaller than in the previous step. The largest of the obtained M_{Rd} values determines the bearing capacity in the normal section of the bending element with incomplete use of the reinforcement strength.

Conclusions

Thus, the application of fractional-rational dependence for describing the process of deforming the concrete of the bending elements compressed area enabled to simplify and approximate significantly to the engineering solution the problem of the elements bearing capacity determining with incomplete use of the tensile reinforcement strength.

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Calculation of building structures and features of its automation technology

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Recently, the issues of evaluating the quality of building structures and predicting their reliability are becoming increasingly important, especially for the construction of industrial facilities of the old fund. Powerful mathematical methods are being developed to evaluate the performance of structures and predict their reliability, but there are no automated computer systems for such analysis. At present, programs for the determined calculation of structures have been developed, which implement methods for the resistance of materials, theoretical and construction mechanics, but they do not provide an opportunity to determine and predict reliability, especially objects of the old fund. The methods of the classical reliability theory combined with the methods of statistical modeling are used in the work, which requires the use of modern IT technologies methods with the development of appropriate software systems.

Keywords: construction, reliability, automated computer systems, software complex, stress-strain state

Розрахунок будівельних конструкцій та особливості технології його автоматизації

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Останнім часом питання оцінки якості будівельних конструкцій і прогнозування їх надійності стають все більш актуальними, особливо для будівництва промислових об'єктів старого фонду. Розробляються потужні математичні методи для оцінки експлуатаційних характеристик конструкцій і прогнозування їх надійності, але немає автоматизованих комп'ютерних систем для такого аналізу. На даний час розроблені програми для детального розрахунку конструкцій, в яких реалізовані методи опору матеріалів, теоретичної та будівельної механіки, але вони не дають можливості визначати і прогнозувати надійність, особливо об'єктів старого фонду. В роботі використовуються методи класичної теорії надійності в поєднанні з методами статистичного моделювання, що вимагає використання сучасних методів IT-технологій з розробкою відповідних програмних систем. Наведено особливості моделі та алгоритмічна база задачі прогнозування міцності та надійності будівельних конструкцій. Виконано декомпозицію та модульне представлення програмної системи розрахунку конструкцій. У ракурсі задачі оцінювання напружено-деформованого стану конструкцій розроблене програмне забезпечення у вигляді драйвера прошивки для електронно-комп’ютерного пристрою лабораторного обстеження елементів конструкцій, що дає змогу визначати стан аварійності елементів конструкцій та отримувати числові значення фактично діючих навантажень на конструкції та їх елементи. Показано особливості поєднання окремих утиліт визначення параметрів надійності з метою прогнозування роботи конструкцій різних типів, які можуть працювати у аварійному та передаварійному станах.

Ключові слова: конструкція, надійність, автоматизовані комп'ютерні системи, програмний комплекс, напружено-деформований стан.

Introduction

Nowadays, there is an urgent problem of many objects with conditions appointment reconstruction. Under a large number of abandoned buildings, there is an urgent need to evaluate the current quality of structures of these objects in order to predict their strength and reliability, as well as in the development of engineering recommendations for their further exploitation. In order to assess the quality and predict the reliability of such objects, there must be a clear idea of structures current state, their ruin degree, the materials used in the construction, and the type of activity planned after the reconstruction [9].

Review of research sources and publications

One of the most powerful modern methods of calculating complex building constructions is the finite element method, which involves the overlay on the finite-element grid construction, that is, structure decomposition into elements within each, other known as functions of displacements and stresses [1-7].

The finite element method also involves a system of linear algebraic equations formation and solution, the order of which is caused by the number of unknown nodal displacements.

In this case, such concepts as the freedom of the node finite-element grid degree, considered as the vector of external loads, the supporting reactions vector. Another important factor that causes a real stressful condition is the rigidity of the structure cross section to the bend, shear and stretch. The above features of the calculation of such structures for predicting their strength and reliability determine the constant need to use the capacities of modern PCs, which in turn requires automation of settlement processes in the form of complete software complexes and systems [11-13].

Definition of unsolved aspects of the problem

The stress-strain state analysis of structures, buildings and structures is an integral part of the design process, the quality assessment and prediction of reliability in construction. Structural mechanics classifies two types of constructions in the context of the kinematic analysis, in particular by dividing the statically distinguishable and statically obscure hinged-rod systems. Calculation of the first one does not cause particular problems for engineers, but the second type systems calculation can cause considerable difficulties. It occurs to the necessity of compiling and solving, in addition to the static equilibrium basic equations of so-called additional equations of deformations or displacements compatibility.

Problem statement

Considering the necessity of the abandoned buildings reconstruction for bringing them into line with the planned activities, the assessment of their residual bearing capacity, real quality at the time of evaluation and reliability prediction during further possible exploitation becomes extremely [8, 10]. Under such terms, the actual issue is a detailed study of the structure's quality and their reliability pre-diction in possible further exploitation.

Basic material and results

According to the Poltava National Technical Yuri Kondratyuk University Department of Structural and Theoretical Mechanics several software tools that implement interrelated tasks for assessing the strength and building structures and their elements reliability were developed [8, 10].

The decomposition principle has been used as a scientific method, which uses the structure of the problem for development and enables to replace the solution of one large reliability task assessing of structures by solving a series of smaller ones in the volume of interconnected tasks.

Thus, decomposition as a separation process enables to consider the complex system of the software complex as consisting interrelated separate subsystems which, in turn, can also be divided into parts, in particular, part of the simulation factors, a part of the deterministic calculation and a part of the probabilistic construction calculation.

The output system of the developed software complex consisting of the four modules described above is located at zero level. After its division, the first level subsystems are released. This is a subsystem for describing the modules of deterministic various calculations constructions.

Among the main types of constructions, the calculation of which is realized in the utility software form are such as beam structures, reinforced beams with sprengel, frame structures, frames, arched constructions, etc. For such types of structures, solved evaluation tasks of the stress-strain state main components, based on which the parameters of the construction's reliability are evaluated as the characteristics of failure-free operation. These subsystems division and some of the other lead to the subsystem's appearance of the second level.

Software formulas of random number generator are used to calculate stochastic factors such as wind and snow load, steel constructions corrosion and tolerances of rolled various types of cross sections, crane loads in industrial shops, temperature fluctuations as well as material strength and precision of structures construction. Models are developed on the basis of mathematical interpretation, in which the description of the main dependencies is carried out on the equations of building engineering, in particular under the Hooke law, the Zhuravsky formula, the Moor formula, used the general principles of finite elements method, as well as methods of numerical integration and functions approximation.

The software package is represented by a structure that links the levels with the mandatory elements of the structure with the levels of variations of all or part of these elements. Depicted the hierarchical structure is in the form of a tree, that is, a graph without closed routes, with the arrangement of vertices at certain levels. The upper level top is the root. The levels combination and their number are determined by the requirements of the visibility and ease of received hierarchical structure perception.

As for the subsystems lower elementary level we have taken such a subsystem where the understanding of their nature and description is available to the executor, in our case, the developer. Hence, the hierarchical structure for the program is subjectively oriented, but for our system it is quite definite.

Here is an activity diagram description for the developed software complex to explain its logical structure and to describe its physical implementation. As a graphical software system main functions representation, a diagram of usage options, namely the activity diagram, is adopted that explains how these features are used.

Thus, in accordance with the activity diagram, the initial activity node "activity initial node" describes the construction design study beginning, that is, laboratory studies conducted to identify imperfections or damage.

For example, for trusses elements characteristic it is the presence of only longitudinal types of internal efforts, while in the real situation, other types of efforts can be recorded, indicating the accidental operation of the structural element. In this work a description of the utility definition of the structural elements for a farm accident rate has been given.

In the case where the reliability analysis is performed on the criterion of bearing capacity, the algorithm of determination of the loads boundary value is accordingly activated, which is further compared with the actual load on the structure. In the case when the reliability analysis is performed according to the reliability criterion, the algorithm of the probabilistic calculation of the design accordingly is activated.

Further, the send signal action occurs before the performance check of the load bearing capacity or reliability.

The result of the latter is the conclusion on the basis of which a project for the reconstruction and reinforcement of building construction elements is being developed or a design passport is issued for its further exploitation. The complex activity diagram is terminated by the activity end-node (terminal activity).

To automate the determination of possible emergency work of structures and to obtain numerical data for further numerical-iterative statistical modeling, the task iss to develop an electronic system of digital fixation of deformations and computer control of laboratory researches of building constructions elements.

In the laboratory of Poltava National Technical Yuri Kondratyuk University a developed tool for analyzing the damage to the elements of the truss farm has been applied.

For the task, a software utility was developed that enables to determine, based on the evidence of deformation sensors, whether the design is in a problem condition. That is, it works in regular mode or it is needed to be reconstructed and strengthened.

The utility algorithm is based on the basic notions of material resistance, that is, from the sensors obtained by deformation values, they pass to the values of normal stresses according to Hooke's law, after which they decompose these stresses on the values of the forces acting in the experimental section of the construction.

The decomposition procedure is based on the method of least squares, which has shown itself well in such tasks.

As it is known, the essence of this method is to minimize the squares sum of deviations between experimental data and data based on theoretical dependence.

The basis of the formula is the centrifugal compression of the material resistance:

$$\sigma = \sigma_N + \sigma_{Mx} + \sigma_{My} = \frac{N}{A} + \frac{Mx}{Jx}x + \frac{My}{Jy}y \qquad (1)$$

where N – longitudinal effort,

Mx, My – the desired bending moments relative to the x and y axes,

A – sectional area of the structure,

Jx, Jy – the moments of inertia of the cross section with respect to the x and y axes.

Figures 1-5 show the form of a software utility for entering the geometric characteristics of the section and the main form of the program.

As can be seen, this formula considers the off-center compression of the cross-section of the design in two directions - relative to the x-axis and with respect to the y-axis.

Thus, knowing the normal stress at the cross-section can be subjected to the equation and considering the value of the section's geometry and the sensor's binding relative to the center of gravity of the section, determine the components of the equation - the longitudinal force and the bending moments relative to the two axes x and y.

Several sensors can be installed in the cross section to improve the accuracy of such calculations, each of which can produce results with different accuracy. Hence, the use of the least square’s method is fairly rational and appropriate in view of one of the important properties, in particular the fact that this method tends to mitigate possible errors and inaccuracies in experimental research.

The method of least squares for the considered problem involves the formation of a system of three equations, which solves the coefficients a, b, c. The indicated coefficients represent the relation of the desired force factors to the known geometric characteristics of the cross section

$$\begin{aligned} a &= \frac{N}{A}[kN/m^2]; \\ b &= \frac{Mx}{Jx}[kN\cdot m/m^4]; \\ a &= \frac{My}{Jy}[kN\cdot m/m^4]. \end{aligned} \qquad (2)$$

As you know, the function of several arguments will have an extremum if the derivatives behind each variable are zero, so we can write down the system of equations of the least squares method for our task

$$\begin{cases} n \cdot a + b \cdot \sum_{i=1}^{n} y_i + c \cdot \sum_{i=1}^{n} x_i = \sum_{i=1}^{n} \sigma_i \\ a \cdot \sum_{i=1}^{n} x_i + b \cdot \sum_{i=1}^{n} x_i y_i + c \cdot \sum_{i=1}^{n} x_i^2 = \sum_{i=1}^{n} \sigma_i x_i \\ a \cdot \sum_{i=1}^{n} y_i + b \cdot \sum_{i=1}^{n} y_i^2 + c \cdot \sum_{i=1}^{n} x_i y_i = \sum_{i=1}^{n} \sigma_i y_i \end{cases} . \qquad (3)$$

Thus, it is possible to determine how the section of the construction works not in a project mode, which affects the probability of exhaustion of the bearing capacity of the section and the structure in general and, as a consequence, the reliability of further exploitation.

On the basis of the above mathematical substantiation of the estimation of the accident rate of building constructions elements in a laboratory a special electronic-computer device was developed, the indicators of which determine the non-project work state, which is the emergency state.

The main physical devices of the developed system include the following: a system of glued in characteristic points of cross sections for the tensonometric sensors construction, a system of servo-devices for controlling the laboratory load on a construction, an analog-digital converter and an operational amplifier. The results of deformation measurements are used to obtain the values of internal forces that act in the sections of structural elements.

The obtained values of internal efforts give the investigator-engineer an accurate picture of the stress-strain main components distribution in the sections of structural elements, on the basis of which the influence of various factors may be modeled in the future.

The following is a description of the software utility for conducting such laboratory studies.

In the design research lab, a developed utility for working with various types of constructions was introduced.

This construction cross-section is a compiled section of the two rolled steel cubes profiles, each of which has three sensors at characteristic points, which guarantees the coverage of the entire cross-section and the results objectivity.

The advantages of the developed digital system are the possibility of automation of deformation detection in a large number of cross sections structures control with high frequency and with great accuracy.

To do this, the authors have developed an electronic device based on the microprocessor ATmega328, which has been developed on the Arduino Uno/Mega processor board.

This processor is a fairly common device in cases where it is necessary to combine analog and digital devices with modern computer systems, in our case - digital sensors of deformations with a PC.

Also, for this processor it is characterized the possibility of automating the various technological processes management, in our case – the control of servo-devices for switching-on / off the pump motors of the test loads station, depending on the degree of fixed deformations of the laboratory structure.

In laboratory for testing building constructions, an electronic device with software in the form of drivers for firmware microprocessor ATmega328 based on Arduino Uno/Mega has been developed and tested.

For example, here are some screenshots of the forms of utility interaction with a user for assessing the degree of the truss rods damage.

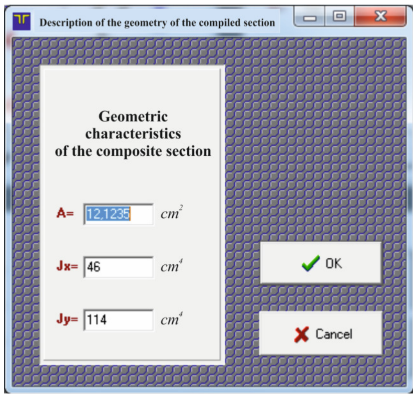

Figure 1 – Modal form for describing geometric cross section of structures

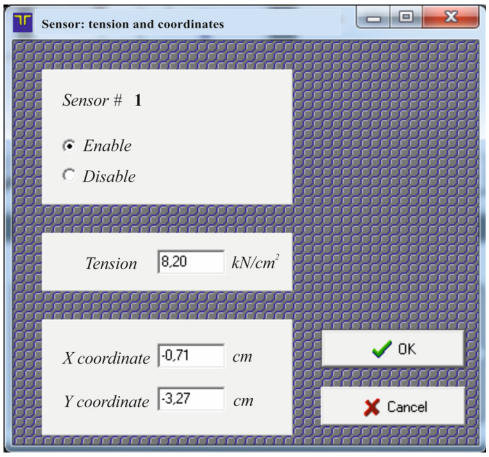

Figure 2 – Modal form for the control of the selected deformation sensor and fixed voltage by electronic-digital device

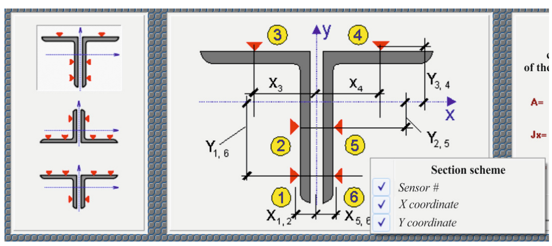

Figure 3 – Section scheme properties

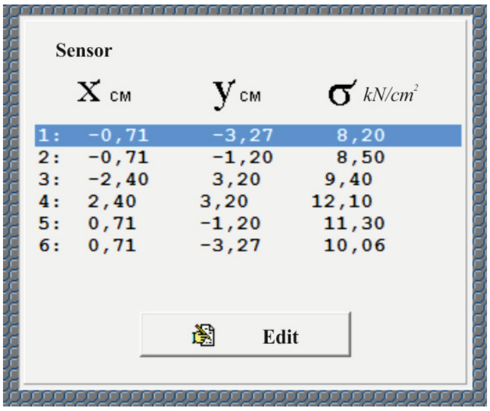

Figure 4 – Sensors data – X, Y and tension data

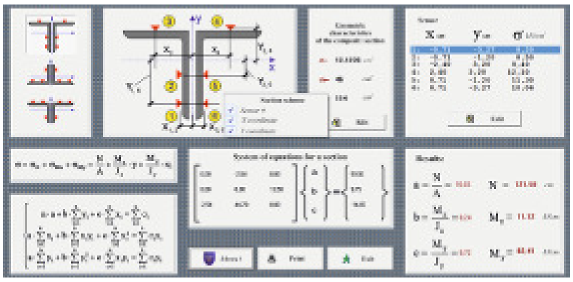

Figure 5 – The electronic computer system form for the sensors processing of structural deformation

Conclusions

The program algorithm realization peculiarities, which considered various factors of the actual work of structures and determined their stress-strain state have been presented. Software has been developed as a firmware driver for electronic structural elements of computer device laboratory testing, which enables to determine the state of structural elements emergency and obtain numerical values of actual loadings on structures and their elements. The separate utilities combination features have been presented for reliability parameters determination in order to predict the work of various structures types that can work in emergency and pre-crash states.

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Improved calculation method of reinforced concrete elements strength on inclined sections

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Application boundaries of the truss analogy and disk model methods to the strength calculation of reinforced concrete flexural elements inclined sections are established. Areas of structures failure by virtual compressed element (inclined strip) and compressed zone over dangerous inclined crack under the shear force are determined. The criterion of minimum limit force, which is perceived by the elements, is applied. Influence of concrete class, relative shear span and transverse reinforcement intensity on elements strength based on variational method in plasticity theory is specified. The data concerning the values of the transverse reinforcement coefficient at the boundaries of the failure from shear within the inclined strip and compressed zone over the dangerous crack are obtained.

Keywords: truss analogy, disk model, boundary of failure cases realization, minimum value of limit force, coefficient of transverse reinforcement

Удосконалена методика розрахунку міцності залізобетонних елементів за похилими перерізами

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Встановлені межі застосування методів фермової аналогії та дискової моделі для розрахунку міцності за похилими перерізами залізобетонних елементів, що згинаються. Визначені області реалізації руйнування залізобетонних конструкцій за умовним стиснутим елементом (похилою смугою) на дію поперечною сили та стиснутою зоною над небезпечною похилою тріщиною на основі застосуванням критерію мінімуму граничного зусилля, що сприймається елементами. Розрахункові залежності приведені до єдиної основи. Отримані дані щодо значень коефіцієнта поперечного армування елементів на межі руйнування від зрізу похилої смуги та стиснутої зони над небезпечною тріщиною. Уточнено вплив класу бетону, відносного прольоту зрізу та інтенсивності поперечного армування на міцність елементів, що підтверджується експериментальними дослідженнями. Для інтегрального оцінювання факторів впливу розглянуто задачу міцності похилої призми, завантаженої на торцях стиснутою нормальною та дотичною силами. Бетон розглядається як жорстко-пластичне тіло. Локалізація пластичної деформації в тонких шарах на поверхні руйнування є характерною для граничного стану бетону при зрізі. Аналіз отриманих варіаційний методом у теорії пластичності результатів дозволяє внести обгрунтовані зміни до визначення міцності стиснутого похилого елемента. Уточнена методика розрахунку міцності залізобетонних конструкцій за похилими перерізами на дію поперечної сили дозволяє отримати більш ефективні конструктивні рішення. Для інженерних розрахунків міцності запропонована залежність щодо визначення коефіцієнта поперечного армування ділянок конструкцій біля опор, який відповідає межі розглянутих випадків руйнування за похилим елементом та стиснутою зоною над небезпечною похилою тріщиною. Встановлена перспективність застосування теорії пластичності для вдосконалення конструктивних рішень згинальних залізобетонних елементів на основі подальшого уточнені міцності похилого елемента як складової фермової аналогії, стиснутої зони над небезпечною тріщиною і системного дослідження зрізу.

Ключові слова: фермова аналогія, дискова модель, межа реалізації випадків руйнування, мінімальне значення граничних зусиль, коефіцієнт поперечного армування

Introduction

Flexural reinforced concrete structures are widespread in practice and largely determine the construction cost. One way to optimize their design solutions is to improve calculation methods. At the same time an important place is occupied by the strength of the structures sections near the supports, which destruction on inclined sections has externally fragile avalanche character.

The theory of calculating the inclined sections strength, as well as the general theory of calculating reinforced concrete, has undergone several well-known development stages: «classical» theory, which used the basics of materials strength and considered the second stage of stress-strain state, in which the calculation was carried out at the main tensile stresses; the stage of calculating the elements by the destruction stage based on the truss analogy and the disk model; ideas deepening period about the stress-strain state of elements in the inclined cracks area.

The «classical» theory did not enable to consider the specific behavior of reinforced concrete elements in the stage of destruction. Thus, in the presence of transverse reinforcement, the actual value of the failure load exceeded the theoretical value considerably, and in its absence the calculation significantly underestimated the strength. The proposed later truss analogy considers reinforced concrete structures as trusses. They are divided into two chords – compressed one and tensile one with a constant arm of internal couple of forces along the element length, connected by an open gating of tensile braces of transverse reinforcement and compressed virtual concrete braces. The calculations provided full perception by reinforcement of the tensile force and by compressed force concrete. Based on the disk model, two schemes for the destruction of an inclined section were considered – under the bending moment and shear force action. Each scheme was described by one equilibrium equation of internal efforts and external forces. The efforts were considered to be perceived by concrete over dangerous crack and transverse reinforcement or pitch reinforcement. The method of limit equilibrium in inclined sections was the basis of the previous norms and remained normative for a long time.

During the period of in-depth research, the truss analogy method and the disk model were improved; the influence of the determining factors was specified. [1-12].

Review of research sources and publications

After enforcement of the normative documents DBN B.2.6-58:2009 [13] and DSTU B B.2.6.156: 2010 [14], the truss analogy is taken as a principle for calculation. Meanwhile, numerous experimental studies have shown the compressed inclined strip failure – the truss analogy element, as well as a concrete shear in a compressed zone over a dangerous inclined crack [1, 2, 7-10]. It should be noted that the virtual compressed element is crossed by transverse reinforcement that increases its strength, which decreases with decreasing inclination angle of the strip [13, 14]. Concerning the elements strength by shear of the compressed zone concrete, it should be noted that it is defined as the sum of the forces perceived by the compressed zone concrete and the transverse reinforcement that crosses the inclined crack [15]. With the relative shear span increase, the component of the shear effort perceived by concrete decreases. For the realization of a particular failure case, the transverse reinforcement intensity of arears near supports is crucial [7, 10].

Not solved earlier parts of the general problem are the demarcation of the above-mentioned cases of reinforced concrete elements destruction and the influences justified clarification of the determining strength factors: concrete strength, relative shear span, the transverse reinforcement intensity.

Objective of the work and research methods

It is determination of application areas of the truss analogy and disk model methods to the reinforced concrete elements strength calculation on inclined sections using the upper estimation of the efforts level and the variation method in the theory of plasticity.

Basic material and results

According to the current norms [13, 14], the force value $V_{Rd,1}$, which is perceived by the reinforced concrete element in the inclined section, is taken as smaller value between the values perceived by the concrete $V_{Rd,max}$ and the transverse reinforcement $V_{Rd,s}$.

For using qualities of concrete and reinforcement full-scale while designing reinforced concrete flexural structures it is advisable to apply the equation

$$V_{Rd,\max} = V_{Rd,s}\,, \tag{1}$$

where $V_{Rd,\max} = \dfrac{0.6 f_{cd} b_w z}{\cot\theta + \tan\theta}$, $V_{Rd,s} = \dfrac{A_{sw}}{s} z f_{ywd} \cot\theta$,

here f_{cd} – design value of concrete compressive strength, b_w – the element cross-section width, z – arm of internal couple of forces, θ – the the virtual compressed element inclination angle, A_{sw} – the cross-section area of shear reinforcement, s – step of shear reinforcement, f_{ywd} – design yield of shear reinforcement.

When $V_{Rd,s} > V_{Rd,\max}$ the transverse reinforcement is not effectively used and exceeding by the coefficient of reinforcement $\rho_w = \dfrac{A_{sw}}{b_w s}$ the value that is equal to

$$\rho_w = \frac{V_{Rd,\max}}{f_{ywd} b_w z \cot\theta}\,, \tag{2}$$

leads to reinforcement overrun.

In the case of a significant decrease in the transverse reinforcement intensity, which is characteristic for the structures designed in accordance with [15], or which have corrosive damage, the elements strength estimated according to [14] in the inclined sections is found to be much lower than that established experimentally [7, 10, 11]. It should be taken into consideration that the destruction of the areas near the supports occurs by shearing concrete in the most intense area within the compressed inclined element. The transverse reinforcement crosses this element along its entire length, that is, the tensile brace passes through the compressed brace and reinforces it.

Therefore, when establishing the application areas of the truss analogy method, the inclined compressed element strength ($V_{Rd,max}$) must be compared with the strength ($V_{Rd,2}$) determined by the disk model using the limit equilibrium conditions over the entire interval of inclination angles θ proposed in [14]. Thus, by $\theta = 45^o V_{Rd,max}$ is almost equal to the force given in [15] in an inclined concrete strip by the shear force action.

In the interval of $1 \le \cos\theta \le 2.5$ for the calculated value of the shear force should be taken smaller from the values $V_{Rd,max}$ and $V_{Rd,2}$.

Shear form of destruction is characterized for concrete and reinforced concrete elements by action of the shear forces [16, 17]. It is also implemented within an inclined strip and a compressed zone over a dangerous inclined crack [18, 19].

For comparative analysis the disk model dependences [15] are corrected to the parameters used in [14] and take the form:

$$V_{Rd,2} = V_c + V_{sw} \,, \tag{3}$$

here $V_{Rd,2}$ – shear effort perceived by the element in an inclined section, V_c and V_{sw} – efforts perceived by concrete and transverse reinforcement accordingly (fig. 1); and these components of equation (3) are equal

$$V_c = \frac{2 f_{ctd} b_w z^2}{c\eta^2} \,, \tag{4}$$

$$V_{sw} = q_{sw} c_0 = q_{sw} \sqrt{\frac{2 f_{ctd}}{\rho_w f_{ywd}} \frac{z}{\eta}} \,, \tag{5}$$

where f_{ctd} – design value of axial tensile strength of concrete, $c = 1.5 z \cot\theta$ – the inclined section projection length on the element longitudinal axis is assumed to be equal to the distance from the support to the concentrated force F or $c = \sqrt{\frac{2 f_{ctd} b_w}{q} \frac{z}{\eta}}$ by uniform distributed load q, $q_w = f_{ywd} A_{sw} / s$ – effort in the reinforcement per unit length of the element, coefficient $\eta = z / d$ (according to [14] is equal to 0,9, here d – the effective height of the cross-section of the element).

In this case, the calculation provides for the fulfillment of conditions: $z/\eta \le c_0 \le 2z/\eta$ and $c_0 \le c$.

The transverse reinforcement coefficient, which corresponds to the application boundary of calculation methods by truss analogy and disk model, is determined by the condition $V_{Rd,1} = V_{Rd,2}$.

The values of ρ_w by the class of reinforcement A400C and the corresponding value of cot θ are given in the table 1.

The dependence $V_{Rd} = \min(V_{Rd,1}, V_{Rd,2})$ on $\cot\theta$ and concrete class are given in fig. 2.

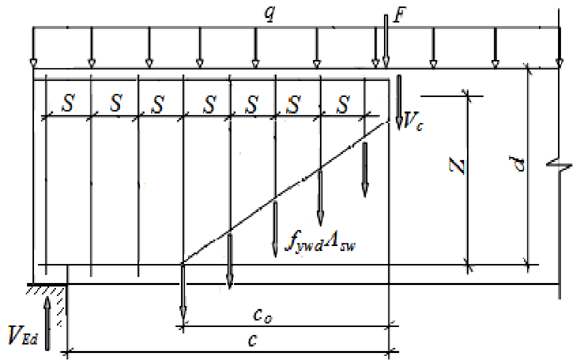


Figure 1 – Scheme of calculated forces on the shear force action on an inclined crack

By the value of cot θ, which exceeds that specified in table 1, the calculation of the shear force strength is made by the formula (3), and otherwise – by the truss analogy.

Table 1 – The values of the relative ultimate effort $\frac{V_{Rd}}{f_{cd} b_w z}$, cot θ and ρ_w at the boundaries of the failure cases at the inclined strip and the compressed zone over the dangerous crack

Class of concrete	$\frac{V_{Rd}}{f_{cd} b_w z}$	cot θ	ρ_w x10^3
C8/10	0.249	1.87	2.53
C16/20	0.272	1.57	6.29
C25/30	0.275	1.53	9.73
C32/40	0.277	1.49	12.9
C40/45	0.279	1,47	16.5
C50/60	0.279	1.47	19.8

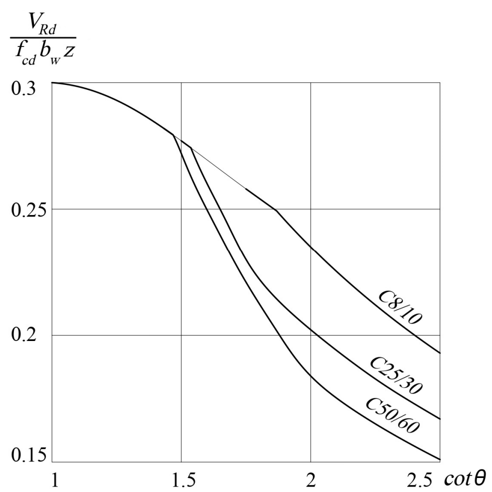


Figure 2 – The dependence of relative transverse shear effort $V_{Rd}/(f_{cd}b_wz)$ on cot θ

It should be noted that the compressed inclined strip strength, as a truss analogy virtual element, is influenced by the vertical or pitch reinforcement that crosses the strip. It increases its strength so much the greater projection strip length on the element longitudinal axis is the smaller the inclination angle to the horizontal θ is. The quantitative parameters of this influence are obtained from the strength problem solution of an inclined prism loaded by normal N and tangent T forces applied at the end surfaces of the prism as shear force components. The concrete prism collapses from shear at the surface of plastic deformation localization at an angle γ to the lateral faces. The reinforcement effect is considered by applying the forces q_w in the reinforcement per unit length of the prism lateral faces (fig. 3), which level is determined by data [20, 21], experimental studies materials [1, 2, 10] considering the ratio of reinforcement and concrete modules and the reinforcement location (the angle of its prism intersection).

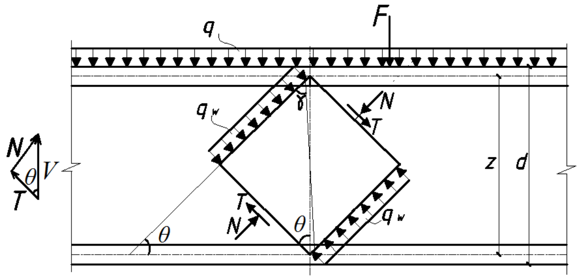


Figure 3 – The design scheme of the inclined prism

The plasticity theory is successfully used to solve the shear strength problems [22, 23]. Determination of the ultimate force perceived by the prism is carried out by a variational method using the virtual velocities principle [24, 25]. Discontinuous solutions of the prism strength problem are obtained. In this case, concrete is considered as a rigid-plastic body [26, 27]. As a plastic potential, the concrete strength condition is accepted [28]. The concrete prism strength decreases with decreasing angle of inclination θ, which is a consequence of the tangent component action T of the shear force. This decrease is offset by the transverse reinforcement effect mentioned above. Thus, the shear force, which is perceived by a virtual compressed inclined element at an interval $1 \le \cot\theta \le 2.5$, is proposed to be determined by the formula

$$V_{Rd,\max} = 0.3 f_{cd} b_w z \ . \tag{6}$$

Meanwhile, the ultimate load value by shear of the compressed zone over the dangerous inclined crack depends significantly on the transverse reinforcement intensity and the relative shear span (table 2).

The values of the transverse reinforcement coefficient at the boundaries of failure cases are given in table 3.

For better visualization, the strength calculation results are presented in fig. 4.

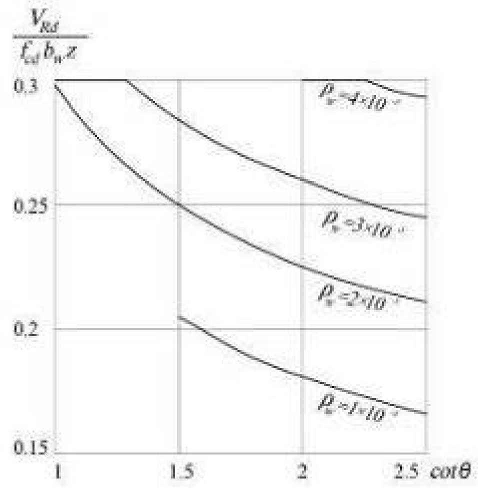
Table 2 – The value of the relative ultimate effort $\frac{V_{Rd}}{f_{cd} b_w z}$ by shear of the compressed zone over the inclined crack

ρ_w x10^3	$\frac{V_{Rd}}{f_{cd} b_w z}$ at $\cot\theta$			
	1	1.5	2	2.5
Concrete class C8/10				
1	0.225	0.205	0.181	0.173
2	0.298	0.25	0.225	0.211
3	0.333	0.284	0.26	0.245
4	0.381	0.332	0.308	0.293
Concrete class C16/20				
6	0.308	0.266	0.246	0.233
7	0.338	0.297	0.276	0.264
8	0.369	0.327	0.307	0.294
Concrete class C25/30				
8	0.282	0.243	0.224	0.212
9	0.302	0.264	0.244	0.233
10	0.323	0.284	0.265	0.253
11	0.344	0.305	0.286	0.274
12	0.365	0.326	0.306	0.295
Concrete class C32/40				
12	0.297	0.262	0.244	0.234
14	0.329	0.294	0.276	0.266
16	0.361	0.326	0.308	0.298
Concrete class C40/50				
16	0.304	0.271	0.255	0.245
18	0.33	0.297	0.28	0.27
20	0.356	0.322	0.305	0.296
Concrete class C50/60				
16	0.27	0.237	0.22	0.211
18	0.292	0.258	0.242	0.232
20	0.313	0.28	0.263	0.253
22	0.334	0.301	0.284	0.274
24	0.356	0.322	0.306	0.296

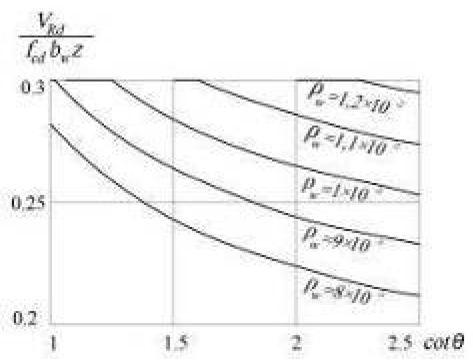
Table 3 – The value of the transverse reinforcement coefficient ρ_w at the boundaries of the destruction on the inclined strip and the compressed zone over the dangerous crack

$\rho_w \times 10^3$ at $\cot\theta$			
1	1,5	2	2,5
2.04	3.45	3.87	4.12
5.75	7.1	7.78	8.18
8.88	10.8	11.7	12.2
12.2	14.4	18.2	21.9
15.7	18.2	19.5	20.9
18.8	21.9	23.5	14.4

a)



b)



c)

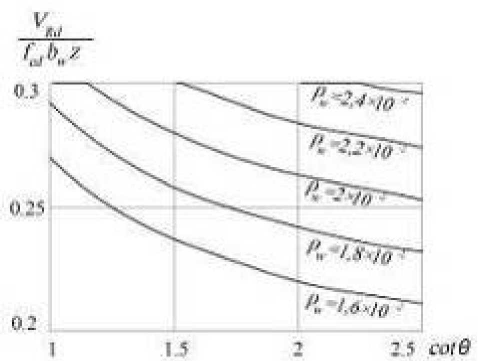


Figure 4 – Dependence of relative shear effort V_{Rd} / ($f_{cd}b_w$z) on cot θ at concrete class C8/10 (a), C25/30 (b), C50/60 (c)

To select the calculation method for the reinforced concrete element on the inclined sections for the shear force action when solving the problem of checking the strength (for the given concrete classes and reinforcement, the rectangular cross-section dimensions, the percentage of the transverse reinforcement ρ_w, the parameter *z* and the compressive element inclination angle *θ*) the ultimate value of the transverse reinforcement coefficient is established

$$\rho_{wR} = \frac{(1-2.5\chi)}{75\chi}\frac{f_{cd}}{f_{ywd}}\cot^{\alpha}\theta, \qquad (7)$$

where $\chi = \frac{f_{ctd}}{f_{cd}}$, $\alpha = \chi(1+75\chi)$.

If $\rho_w < \rho_{wR}$ the force V_{Rd} is equal to the sum of the forces in the compressed zone concrete V_c and in the transverse reinforcement V_{sw}, which are calculated by formulas (4 and 5) and determine the element strength by the inclined crack in the force action V_{Ed}.

In this case, if $V_{Ed} > V_c + V_{sw}$ the flexural element failure occurs by shearing the compressed zone concrete and achieving stresses in the transverse reinforcement, which crosses the inclined crack, the yield strength $A_j = s_k b_{mon}$.

If $\rho_w \geq \rho_{wR}$ – the force V_{Rd} is equal to $V_{Rd,\max}$, which is calculated by the formula (6) and determines the inclined element strength (providing $V_{Ed} > V_{Rd,\max}$ the element destruction occurs from the shear within the inclined strip.

Further improvement of flexural reinforced concrete elements structural decisions and structures creation of equal strength by inclined and normal sections are largely connected with increase of calculation accuracy of their strength on inclined sections based on the systematic shear study as a destruction form and considering the influence of all the determining factors.

Conclusions

1. Analysis of reinforced concrete elements experimental data and results of strength calculations on inclined sections necessitates the improvement of their calculation method.
2. The boundary of elements failure cases realization from concrete shear is established within the inclined strip and the compressed zone over the dangerous crack.
3. Based on the inclined prism shear problem solution by variation method in the plasticity theory, the influence of concrete class, relative shear span and reinforcement intensity are specified.
4. Areas of truss analogy method application and a disk model are determined and the method of calculating the reinforced concrete elements strength on inclined sections is improved.
5. For engineering calculation, the dependence of the transverse reinforcement coefficient ρ_{wR} at the elements destruction boundaries along the inclined strip and the compressed zone over the dangerous crack on the strength characteristics f_{cd}, f_{ctd}, f_{ywd} and the inclination angle *θ* is proposed.

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Experimantal researches of the achievements of a current burdening course plates

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The article presents the results of the experimental study of the peculiarities of the work under the calculation load of planar superstructural plates of the developed system of non-beating overlap. Great attention is paid to the construction of bearing structures prototype. The design of auxiliary equipment is described, which enabled to test the investigating U-block in conditions that imitate real resistance. The method of performing experimental research with indication of methods and means of measuring geometrical and physical parameters that characterize the stress-strain state and bearing capacity of the test plate is given. The conducted investigations enabled to determine the nature of deformation and destruction of the superstructure slabs as a separate element in the developed system of non-beating ceiling. Attention is drawn to the fact that the achievement of the bearing capacity is not accompanied by the process of destruction, but is characterized by significant movements of the supporting plots in the vertical plane.

Keywords: displacement, bearing capacity, deformed state, experimental research, unbounded overlap

Експериментальні дослідження надколонної плити збірного безбалкового перекриття

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У статті наведено результати експериментального дослідження особливостей роботи під розрахунковим навантаженням плоских надколонних плит розробленої системи безбалкового перекриття. Приділено велику увагу конструюванню дослідних зразків несучої конструкції. Особливістю плити, що досліджується є скошені майданчики спирання по зовнішньому периметру для міжколонних та прольотних плит. Описано конструкцію допоміжного устаткування, яке дозволило випробовувати досліджувальну надколонну плиту в умовах, що імітують реальне спирання. Описано конструкцію та технологію виготовлення додаткового устаткування, що імітацує спирання надколонної плити на колонну у вигляді сталезалізобетонного стакану квадратного перерізу. А також опорну конструкція, що імітує спирання міжколоних та прольотних плит. Дослідні конструкції плити і допоміжного устаткування виготовлено на обладнанні та в умовах діючого заводу залізобетонних виробів. Наведено методику виконання експериментальних досліджень із вказуванням методів і засобів вимірювання геометричних та фізичних параметрів, що характеризують напружено-деформований стан та несучу здатність дослідної плити. Проведені дослідження дали змогу встановити характер деформування і руйнування надколонних плит як окремого елемента у розробленій системі безбалкового перекриття. Аналіз характеру деформування свідчить, що деформації розтягу на верхній грані дослідної плити наростають швидше ніж деформації стиснення (нижньої грані). При цьому відбувається зменшення величини стискаючих деформацій до місця спирання дослідних зразків на опорну раму. Це підтверджує припущення про передачу навантаження від міжколонних та прольотних плит на надколонну за принципом «лінійного шарніру». Звернута увага на те, що досягнення несучої здатності не супроводжується процесом руйнування, а характеризується суттєвими переміщеннями опорних консольних ділянок у вертикальній площині.

Ключові слова: переміщення, несуча здатність, деформований стан, експериментальне дослідження, безбалкове перекриття

Introduction

Modern tasks of construction development raise new requirements for the production of building structures and their modernization on the basis of scientific and technological progress. The significant effect when introducing new upgraded constructive solutions can be achieved at the expense of the optimal combination of physical and technical parameters of elements under the scheme "design-material-technology". According to this scheme, the main part of the economy is formed, firstly, due to the wider use of the existing potential of prefabricated reinforced concrete structures, in particular the use of round-bottomed slabs, and secondly, the use of advanced steel reinforced concrete structures that combine the advantages of steel and concrete and enable to reduce structural height of the frame elements.

At present, the bulky, non-skidding and non-rigid overhead structures are widely used in construction. Such constructive systems provide the possibility of constructing buildings of arbitrary configuration in terms of different volumetric-planning solutions. A further step in the modification of prefabricated and prefabricated monolithic frames of buildings and structures is a combination of prefabricated round-bottomed slabs, their modifications and a steel-loose-binder non-white frame. The tasks are aimed at finding rational parameters of such structures, studying their durability and deformability, and implementing the results in construction are feasible and relevant.

Review of research sources and publications

Along with existing types, new progressive structures from steel reinforced concrete have been developed, which enable to reduce costs during installation of structures, to refuse the arrangement of shuttering and additional racks and to increase the mounting speed. Sufficiently widely considered are various systems of non-white floorings in work [1,7-8,10]. All of them have advantages, but they are not defective either. Material and labour costs for their installation are significant.

The system of non-white roofing with modified multi-hollow slabs [2-3,5-6] has been developed by the authors. It is capable of providing optimum with regard to the overall length and strength of the joints of the plate layout and does not require large material and labour costs for the installation and assembly of joints between individual slabs. It is achieved by the use of intercalary, interlayer and flying slabs in prefabricated reinforced concrete bezel, with slabs around the perimeter having slanted lateral faces forming a platform for the adjacency of the adjacent slabs [4,9]. The general stiffness of the overlap is achieved by welding between the mortar parts, which are provided on all plates. For the implementation of the practice of building such overlapping systems, the necessary proposals for their design and, in particular, the calculation of the strength of individual elements and the whole system as a whole.

Definition of unsolved aspects of the problem

At the moment, there is practically no data on the work of individual boards as part of the unbroken overlap. As a prototype, a full-size superstructure slab of non-white floor was selected, which enabled to obtain the most complete information about the object of research.

Problem statemen

The purpose of conducting experimental research is to establish a valid stress-strain state and determine the bearing capacity of individual structural elements of the developed flat non-white floor.

Basic material and results

To reveal the features of the superstructure plates work under the influence of external load, a program of their experimental research was developed (Fig. 1). According to the adopted program of experimental research, a series of prototypes was manufactured. For samples, concrete structures and reinforcing rods were used, which were in the presence of Svetlovodsk ferro-concrete products factory. The thickness of the previous samples of the plates is 220 mm. For precise installation in the design position of samples, on their surfaces were made special markings. Experimental samples were filled with concrete of class C25 / 30 for durability.

Designs of samples and technology of their manufacturing.

For the production of prototype superclone boards, an individual formwork was made (Fig. 2). The overall dimensions of the slab in the plan were 1200×1200 mm. In the middle of the slab there is a hole with the dimensions of 410×410 mm to allow for its installation after the installation of the column. This hole is framed by a "glass" of steel sheets in thickness of 8 mm. On the contour of the plate there are projections with slanted surfaces for the possibility of interlocking and flying slabs on them.

Technique for conducting an experiment.

From each cup, concrete was cemented with 3 standard cubes and 3 standard prisms. Prisms and cubes were made in metal collapsible formwork. For the purpose of creating for prisms and cubes the same conditions for concrete hardening in the test plates, the deck was executed at the age of 28 days.

The method of conducting experimental research includes the production of additional equipment, a choice of power equipment and measuring instruments.

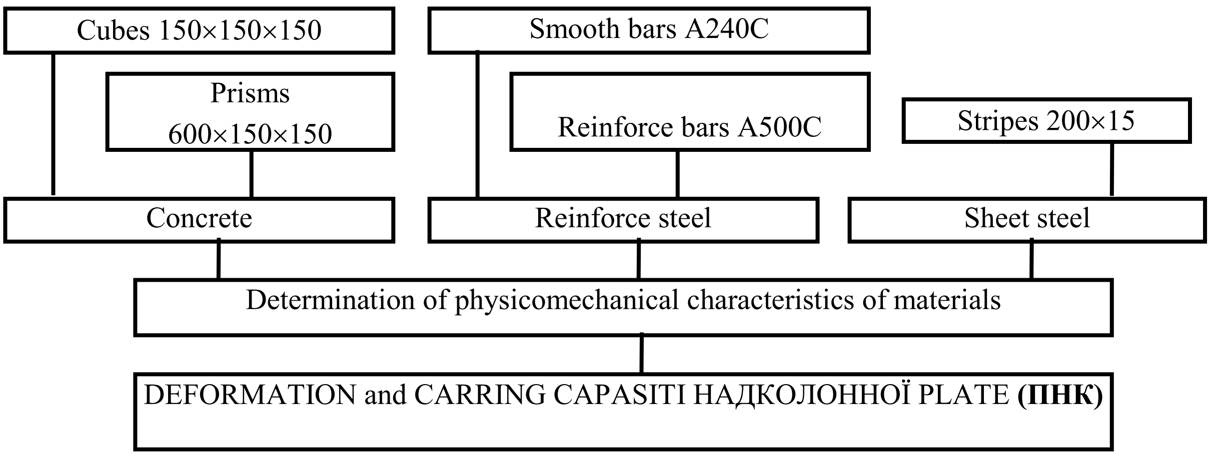

Figure 1 – The program of experimental researches of the superconducting slabs of the base slab system

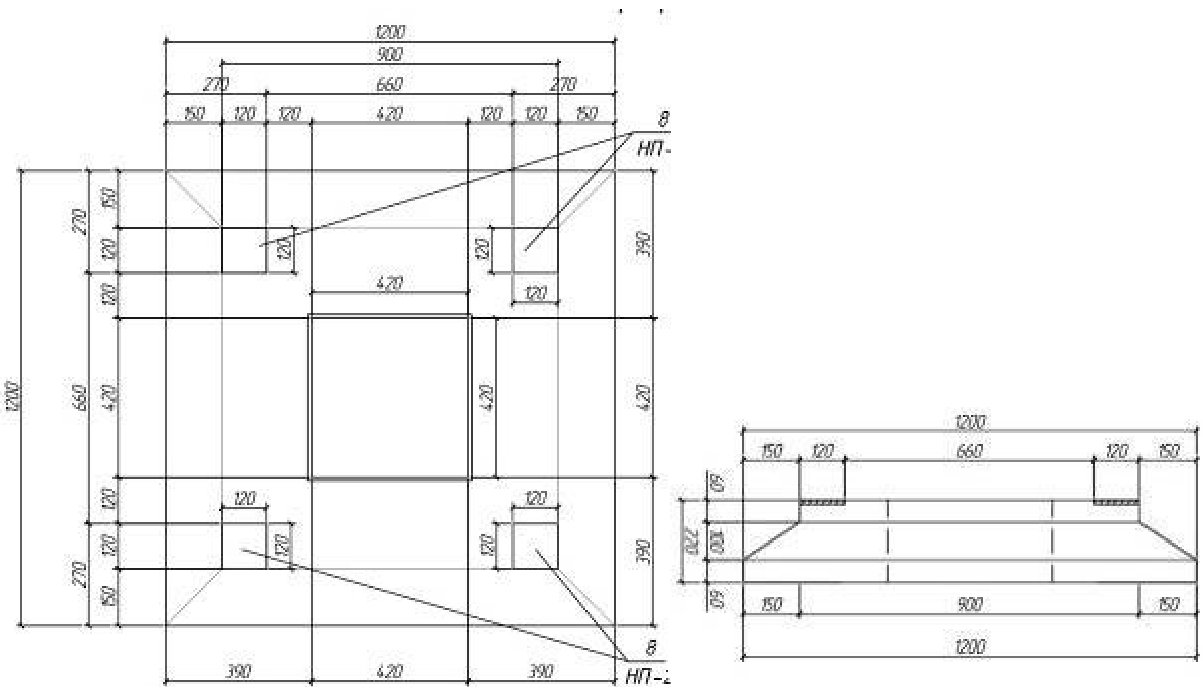

Figure 2 – Geometric dimensions of U-shaped slabs (PNCs)

Fig. 3 shows the assembled design of the support for PNK overhead pillars.

An additional equipment was designed and manufactured for the superconductor coil simulation (Fig. 3). It is a steel-concrete square cross-section with a steel console around the perimeter. Such a construction imitates a part of the column with the place of the superstructure coil. The column part is formed by four segments of equal-angle angles ∟ 125×10 with a length of 400 mm. The interior space between the corners is filled with concrete. From the bottom, the corners are combined with a steel plate 400×400×12 mm. At a distance of 180 mm from the upper corner of the corners around the perimeter fixed console of steel twill ∟ 125×10. The connection of steel elements is made using electric welding.

Figure 4 shows an experimental sample located in a pilot plant. The samples were tested after the concrete has reached the design strength, but not earlier than for 28 days. Testing of samples took place at a specially constructed facility at the laboratory of the Department of Structures on Metal, Wood and Plastics of Poltava National Technical Yuri Kondratyuk University. The equipment consists of a hydraulic installation with a capacity of 300 kN and a bearing beam. The latter through a system of two strains is fixed from the vertical displacements by connecting with the force floor of the laboratory. The efforts on the PNC plate were transmitted through a pre-installed jack.

The loading was carried out in steps of 0.1 from the predicted theoretical calculation of the destructive load N. Each load was maintained for at least 5 minutes. At all stages, relative and absolute deformations were measured. Measurement of deformations was carried out by two methods: with the help of watch-type indicators with a price of a section of 0.01 mm on the basis of 200 mm and an electron-tonal method (Fig. 5 – 6). The base of the electric resonator was 50 mm. For strain-gauge tests an automatic gauge of deformations of GNP-8 was used, the accuracy of which is 1×10^{-5}.

In all samples, for the determination of relative deformations on the upper part, hourglass indicators with a price of 0,01 mm divisions are installed on the basis of 200 mm parallel longitudinal axis between the supporting surfaces. Indicators are fixed using specially made brackets. The brackets were screwed into previously welded N8 screws (Fig. 4).

The locations of the electro-tensile resistors were sealed to a mirror luster, degreased with acetone and based on glue BF-2. After 24 hours gluing of the electro-tensile resistors was performed.

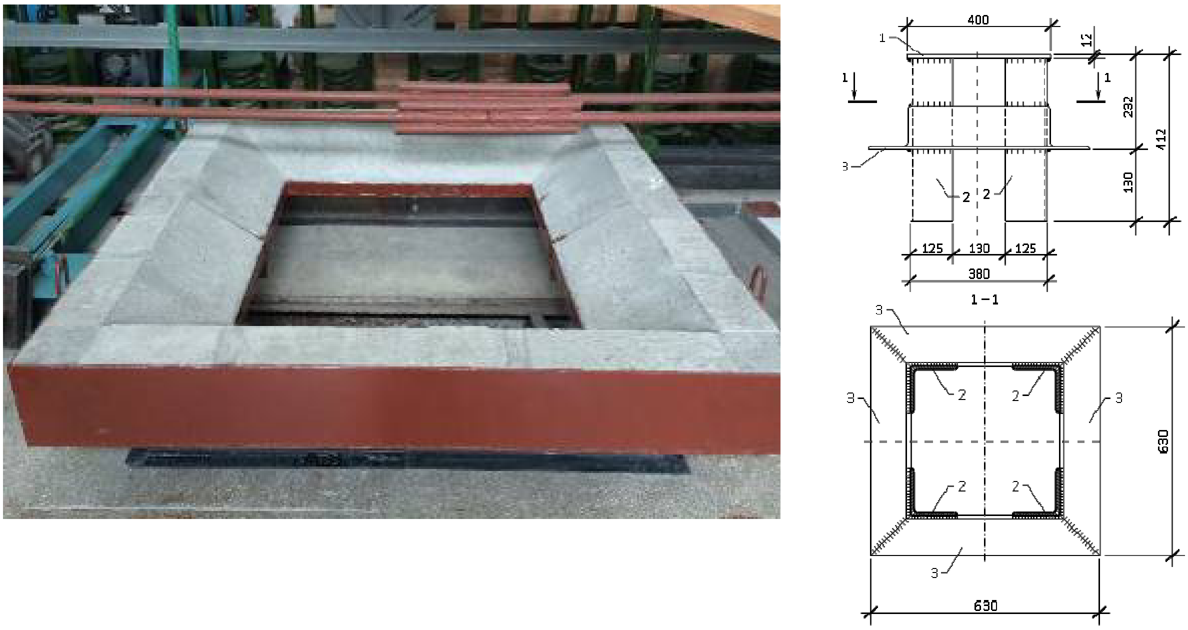

Figure 3 – The support and device imitating the location of the backbone on the PNA:
1 – steel plate 400×400×12; 2 and 3 – a corner 125×10

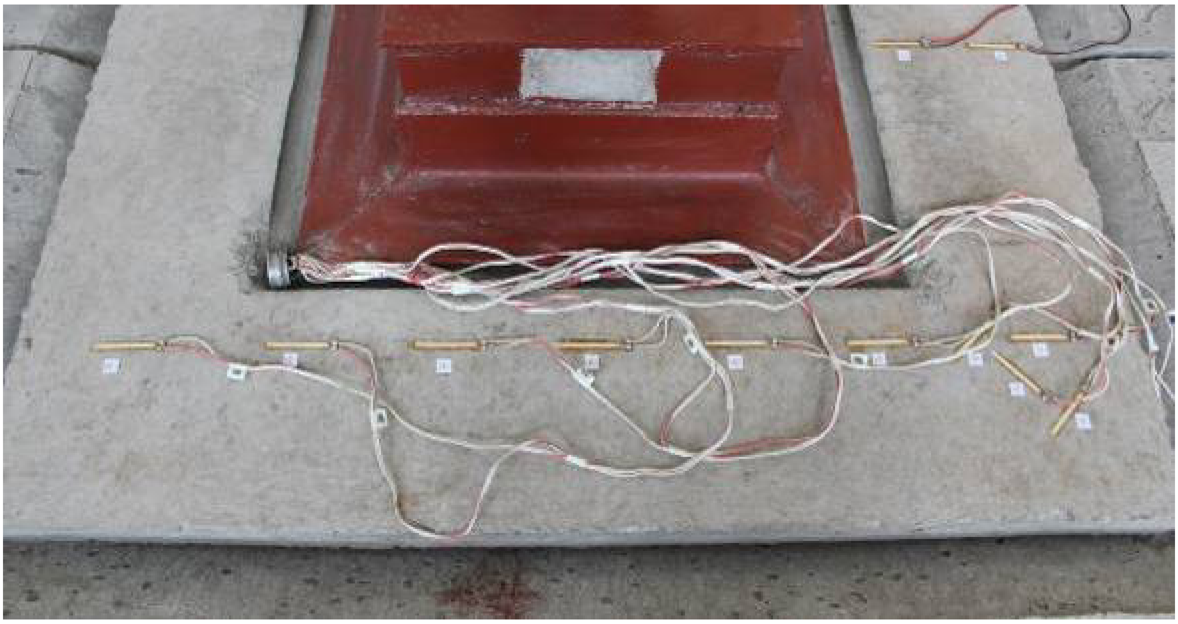

Figure 4 – The layout of the measuring devices on the prototype samples of the PNA series

Figure 5 – Electro-thrust resistors on PNC plates

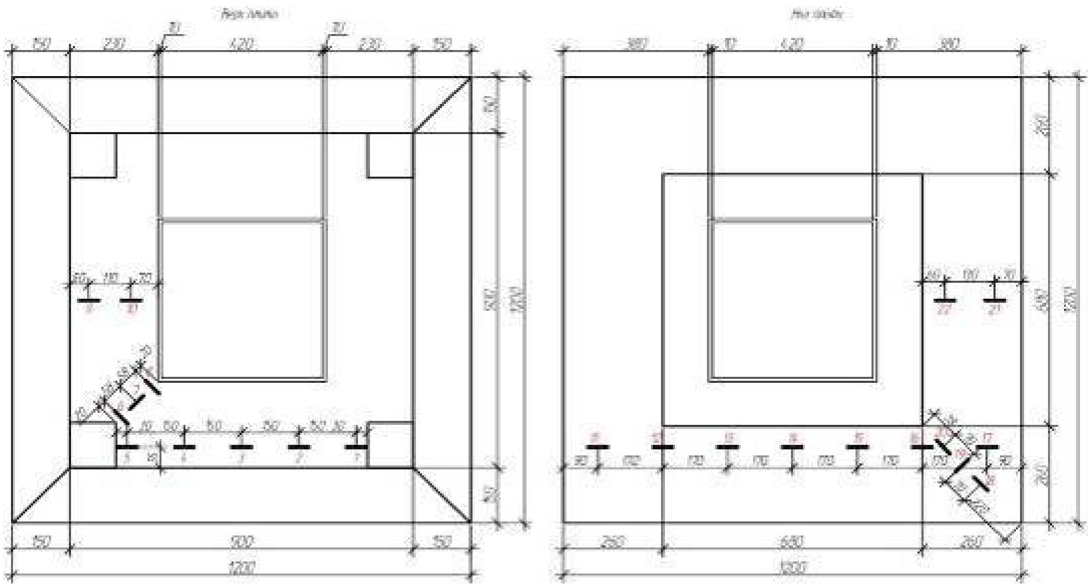

Figure 6 – Scheme of arrangement of electro-tensile resistors on PNA plates

Results of experimental researches and their analysis.

Figure 7 shows graphs showing the change in relative deformations of the most stretched (points 1-5) and the most compressed (points 11-17) of the fibers of the PNA plate.

Measured values of deformations according to the parameters of the electric resonators, the chains of which are located in the cross section between the supporting surfaces. According to the graphs given above, it can be seen that the tensile deformations on the upper edge of the PNA plate grow faster than the deformation of the compression (lower face). In this case, the attenuation of compressive (11-17) deformations occurs to the point of reference of the prototype to the support frame. This confirms the assumption of transferring the load from interconnect (PMK) and flying (software) slabs based on the principle of "linear hinge".

Although no apparent destruction of the experimental prototype of the superconductor slab was detected, attention should be paid to the avalanche-like increase in stretching strains at the location of the electrotensor resistor # 3. With a total load of 210 kN, this electroplating resistor is out of operation. At this moment, the transverse cracks opened up intensively. After removing the external load of their banks turned to the place, but completely the crack was not closed. The distribution of formed cracks can be seen in Figure 9. The depth of crack opening was half the height of the cross section of the PNC plate.

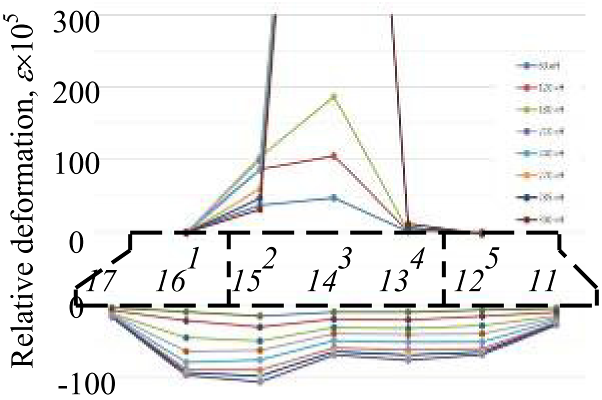

Figure 7 – Distribution of relative deformations on the upper (1-5) and lower (11-17) surfaces of a sample of a series of PNA depending on the value of the payload

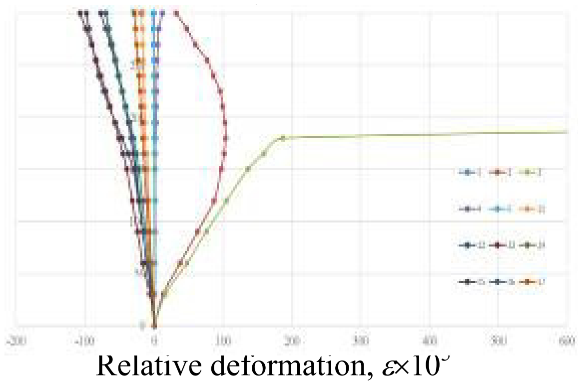

Figure 8 – Distribution of relative deformations for electro-tensor resistors 1-5 and 11-17 surface samples of a series of PNA depending on the value of the payload

Figure 9 – Distribution of cracks in the upper stretched surface of a sample of PNA series after unloading

Interesting is the distribution of deformations at the edges of the PNC plate. The values of compression (t. 11 and t. 17) and stretching (t.1 and t. 5) deformations throughout the loading are close to zero values. But in this place the load is transferred in the form of a reference reaction. Therefore, it can be concluded that the decisive efforts to calculate the supporting part of the PNC plate are the magnitude of the cut-off force.

Figure 8 shows the graphs of relative deformations in the electric resonators 1 - 5 and 11 - 17 from the magnitude of the external imposition. On these charts, the line belonging to the electro-tensor resistor No. 3 is most distinct. It is located in a stretched zone, interspersed with the passage. So, when the value reached 190 kN load, there was a phenomenon of "fluidity". One can make a conclusion that in this place work reinforcement has reached the limit of fluidity. The confirmation of this is the reduction of deformations in the electric resonators No.2 and No.4. Considering the peculiarities of the PNA experimental plate reinforcement, the redistribution of internal forces from the reinforcement linear elements is possible on the ring.

Figure 10 shows graphs showing the change in the relative deformations of the stretched (point 7) and compressed (points 18 and 20) of the fibers of the PNA plate. These points are located in a diagonal section. It should be noted that the growth of tensile strains (point 7) is more intense than compression deformations (points 18 and 20). With an external force of 210 kN, the electric resonator, fixed at point 7, was torn and out of order. But at the same time the design of the experimental plate of the PNC continued to perceive the external force, which continued to grow.

The distribution of deformations on the sloping section of the knot and on the adjoining sites does not change significantly when the external load is increased. The maximum values of compression effort do not exceed the value of 50×10^{-5}. It indicates that the internal bending moment, which is a reaction to the effect of the external load, does not lead to any destruction of the reinforced concrete in this area. And the value of transverse forces remains decisive.

Figure 11 shows the graphs of the relative deformations in the electro-tensor resistors 7, 18 and 20 from the magnitude of the external imposition. On these charts, the line belonging to the electro-tensor resistor No. 7 is most distinct. It is located in a stretched zone, diagonally between the outer and inner angles of the PNA plate. So when the external forces reached the value of 210 kN, deformations began to intensively increase and reached a critical value for concrete of 200×10^{-5}. Although there was no such significant change on the compressed face.

A grid of cracks in this zone has a complex picture in the form of cross-curved lines. It should be noted that there is the presence of a long crack, which has crossed the backs of the design diagonal from the outer to the inner angles. Its appearance indicates the need to make changes in the design of the reinforcing frame of the PNC plate.

Figure 12 shows graphs where the change in relative deformations of stretched (points 9 and 10) and compressed (points 21 and 22) of PNA fiber fibers are shown. These points are located on an average cross-section, which intersects the inner square hole. Points 9 and point 22 are located along one normal to the horizontal sides of the structure. It is obvious that in this place the general picture of the deformed state is preserved - deformation of the tension is more intensive than the deformation of compression. But the absolute value of the deformation on the surface of the design does not reach the critical value, which corresponds to the strength of the concrete.

This area is perpendicular to the reference face. It can be explained by the fact that the deformed state of the minorities than in the extreme sections parallel to the plane of the deflections. Figure 13 shows the graphs of relative deformations in the electric resonators 9, 10, 21, and 22 of the external imposition magnitude. On these charts the line is the most distinct.

The maximum external force applied to the experimental extracobble plate (PNC) was 300 kN. Clearly, the destruction of the plate did not take place, and the cracks that appeared on the stretched surface of the concrete after the unloading were almost closed, leaving only filament marks. The general nature of the crack propagation is shown in Figure 14. All the cracks were transverse, and inclined cracks were not detected.

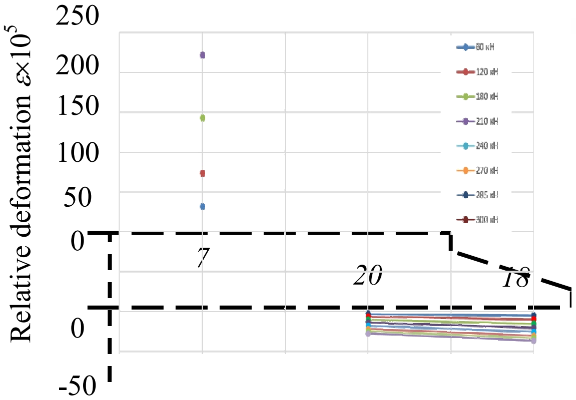

Figure 10 – Distribution of relative deformations on the upper (7) and lower (18, 20) surfaces of a sample of a series of PNA depending on the value of the payload

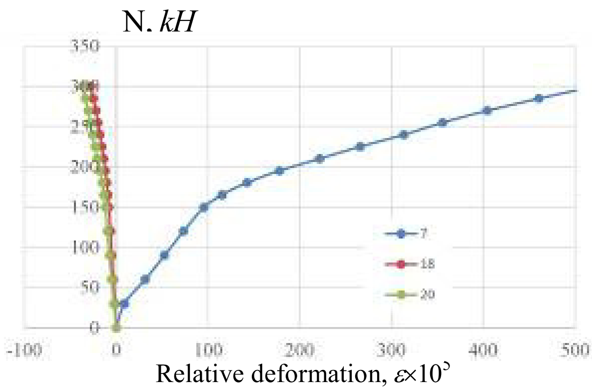

Figure 11 – Distribution of relative deformations for electro-tensor resistors 7 and 18, 20 surface samples of a series of PNA depending on the value of the payload

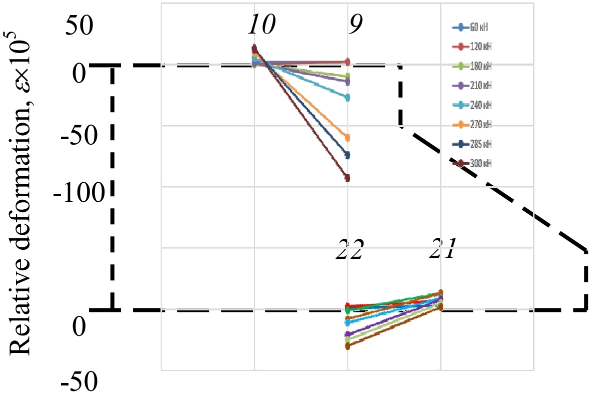

Figure 12 – Distribution of relative deformations on the upper (9-10) and lower (21-22) surfaces of a sample of a series of PNA depending on the value of the payload

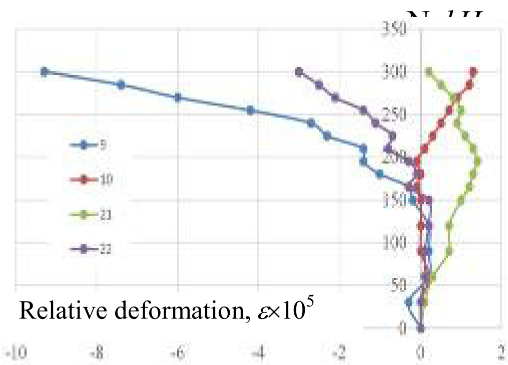

Figure 13 – Distribution of relative deformations for the 9 -10 and 21-22 surface of the electro-conductors of a sample of a series of PNA depending on the value of the payload

Figure 14 – Cracks in the concrete surface of a sample of PNA series

Conclusions

According to the results of the research experimental part, the following conclusions can be drawn:

1. The program of experimental research is developed considering the possibility of using the material base of the existing production of building structures. That enabled to design and make prototypes in the natural size of the real bearing structures of flat non-white floor. Materials of constructions (steel and concrete) are applied in real bearing structures. The test equipment is certified.

2. The maximum aggregate load transferred to the superconducting plate of the PNA series was 300.00 kN. The test sample could not be destroyed. The boundary condition for such a design is the achievement of the yield strength by the extended fittings in individual locations. Therefore, as a bearing capacity, an effort equal to 195.00 kN is taken.

3. The advantage of the proposed separate plates of the developed system of non-stop overlap is that none of the tested structures was destroyed during experimental studies. As the boundary state it is necessary to consider the states of the second group.

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Accidents features in construction

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This work aims to highlight the problem of accidents in the construction industry. On the basis of collected and processed information the accidents classification in the building is presented, depending on the building erection stage. The article gives a detailed description and analysis of such cases in the construction. The collection of information on accidents was carried out using various information sources: Internet resources, literary sources, scientific works and information from world-wide journalistic services. The findings are presented over the last ten years and cover worldwide construction incidents. The material is systematized and presented in the table form. The study results are the created accidents classification with the probability of their occurrence, which can be used later in the design of buildings and structures, in order to predict the various types of accidents in the construction.

Keywords: buildings and structures, building structures, failures, building accidents, destruction of structures

Особливості аварій у будівництві

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Розглянуто проблеми аварій будівель і споруд, відмов конструкцій; зазначено про недосконалості ведення статистики аварій та методології обробки даних про них. Проаналізовано характерні особливості обробки статистичних даних та результатів центрів експертиз, які було опубліковано вченими різних країм за останні роки. Особлива увага звертається на детальний опис і аналіз наймасштабніших аварій у будівництві за минулі роки. Досліджено найбільш розповсюджені випадки руйнування будівель та споруд. Проаналізовано приклади аварій будівель та споруд під час реконструкції, під час будівництва об’єктів та по причині великого віку об’єкту. Визначено найбільш імовірні прогнози можливої аварії (наприклад, пошкодження, вихід із ладу, руйнування будинку, будівлі, споруди, лінійного об’єкту інженерно-транспортної інфраструктури або їх частин), що сталася з техногенних або природних причин для підрахунку можливих матеріальних збитків і (або) соціальних втрат від відмови об’єкту. Виявлено та обґрунтовано необхідність класифікації аварій у будівництві залежно від етапу зведення будівлі. Розкрито підхід до опису аварії, виходячи з її вірогідності. Наведено приклади ситуацій, у яких аварія являлася вірогідною. Представлено дані досліджень, отримані за останні десять років, що охоплюють будівельні інциденти по всьому світу. Класифікація аварій приведена за типом їх виникнення; матеріал систематизовано та представлено у вигляді таблиці. Дані таблиці вміщують у собі опис аварії, її причини, місце та час виникнення. На основі проведеного дослідження запропоновано класифікацію за ймовірністю виникнення аварій будівель та споруд, для неї представлений детальний опис. Класифікацію створено із метою можливості моделювання різних типів вірогідних аварій, які можуть виникнути на об’єкті, для їх подальшого уникнення, а також з метою подальшого викоритання при проектуванні будівельних об’єктів.

Ключові слова: будівлі і споруди, будівельні конструкції,відмови, аварії будівель, руйнування конструкцій.

Introduction

The problem of buildings and structures accidents remains relevant in modern conditions. Cases of buildings collapses with significant economic losses and human victims make it more closely to work on this issue. That is why, in this article, attention is paid to accounting for accidents in buildings and structures in recent years and to create an appropriate classification based on the collected data.

Review of research sources and publications

Several publications are devoted to construction accidents, including the monograph B.I. Belyaeva [1], M.M. Laschenko [2], M.M. Sakhnovskii [3], O.M. Shkineva [4] and many others. Quite detailed material of accident statistics was presented by A.V. Perelmuter in the table form of steel structures accidents causes [5]. Also noteworthy are publications by K.I. Yeremin with references to this subject [6, 7].

Speaking about building accidents, firstly is necessary to consider the reasons of structures failure, among which, except cases of excessive casual load, the accidental magnitude of the load capacity (inadequacy of safety ultimate factor), there are many others (unexplored constructions, errors in design, manufacturing and installation, violation operating rules, etc.) [5]. Also during construction often enough do not adhere to those or other norms and requirements for construction work, which in turn can lead to fatal mistakes, at the cost of which can become human life.

On the example of India, we can give the figures for the statistics of the victims as a result of construction accidents. Data from the National Crime Records Bureau (NCRB) indicates that a total of 38363 people lost their lives due to collapse of various structures between 2001 and 2015. Most people lost their lives because of the collapse of residential houses. Uttar Pradesh recorded the highest number of deaths (5690) during this period [31].

Against this background, it is worth noting that the methods for calculating structures, for example, from bricks, are characterized by a high degree of idealization of their real properties and working conditions under the action of explosive and other loads. Thus, the imperfection of the calculation methods is compensated by increased safety factors [20].

The analysis of publications on the estimation of the accident rate of construction objects shows that the statistics of accidents are not perfect. This is not only about the lack of well-documented failures and accidents, but also about the imperfection of the methodology for processing data on them [5]. In our time, despite the great opportunities in the issues of publicity and the press, it is difficult to obtain objective information of accidents, as the construction market is a commercial struggle between construction companies. As a result, many accidents are deliberately silent, and in the future, such incidents are not publicized. Also, at the current stage of construction development in Ukraine, the question arose about the justification in the state building codes of the people number issue who are constantly on the site and are at risk of accidents.

Speaking about accident statistics, general information is provided annually by the city expert center, which is recognized consultant number 1 in the field of manufacturing in Russia (according to the RA Expert's ratings in 2012). The company «MCE-North», part of the international holding, is provides technical expertise (technical diagnosis) of buildings, structures and equipment [8].

Definition of unsolved aspects of the problem

The question about building accidents is raised during long time. Even there are official organizations of various levels for consideration of accidents that have arisen in construction. But then their own statistics and analysis nowadays are imperfect. From this it can be concluded that this topic requires more attention for furthering study and systematization, which will allow to predict the accident, take the necessary measures, and thus to exclude its possibility.

Problem statement

To analyze building accidents on the materials of modern publications, scientific works, Internet resources and mass media, to create an appropriate classification of buildings and structures accidents

Basic material and results

1. Reasons of building collapses

In considering the accidents statistics in buildings and structures, it is advisable to get acquainted with the work of statistical centers that provide open access information for the relevant period [32].

Thus, according to experts on the technical buildings diagnostics of the Companies Group, the City Center of Expertise (ISE), in 60% of collapse cases occur through a combination of violations committed at different construction and operation stages.

Over 2013, the buildings collapses in Russia killed 57 people and 67 people were injured.

Failure to observe the technology of construction and installation work (including safety rules) accounts for 53% of the reasons for the collapse (in 2012 – 50.68%). Rejections, low quality of building materials – 5% (in 2012 – 1.36%). As a result of violations of conditions (including terms) of the buildings operation, about 38% of decompositions occur (in 2012 – 46,57%). Mistakes made during designing – 4% (in 2012 – 1,36%).

Furthermore, large collapses and those that caused people to suffer were taken in this statistics.

Speaking about the building accidents that took place between May 2014 and May 2015, according to experts from the technical diagnostics of the buildings of the Companies Group, the City Expert Center (CEE), 76% of collapse cases occur through a set of violations committed at different construction and exploitation stages.

For the year, in result of the buildings collapsed in Russia, 43 people were died and 144 people were injured. Non-compliance with the construction technology and installation work (including safety rules) accounts for 45% of the causes of decompositions (in 2013 – 53%).

Rejections, low quality construction materials – 5% (in 2013 – 5%). As a result of conditions violations (including terms) of the buildings operation, about 31% of decompositions occur (in 2013 – 38%). Mistakes made during designing – 19% (in 2013 – 4%).

Without claiming the full problem coverage the as a whole, possible to distinguish the most widespread cases of buildings and structures accidents, namely: errors of engineers in the calculations; negligence of builders during construction an object, improper operation or incorrect reconstruction, cases of which have increased significantly over the past few years.

Accidents should also be classified according to the class of consequences, in accordance with the National Standard of Ukraine [9]. Taking into account the research carried out, the most widespread buildings accident can be considered objects with a consequences class of CC2, in particular residential buildings with the people number who are constantly in the building, up to 400 people.

But it should be noted that the most significant accidents, with hundreds of victims and colossal consequences, occurred in the buildings of the consequences of CC3. These include shopping centers, sports arenas, industrial enterprises and entertainment complexes.

Materials on accidents were sought out through the information network, world news and modern scientific publications, which considered these issues in particular. Based on the received material, classification tables were created for the types of accidents that have occurred in recent years. It should be noted that the information is constantly updated depending on the incidents occurring at the given time.

As a result of the study, a table was created showing examples of accidents and structures in recent years with available information on their destruction, the location and incident causes, as well as the number of victims.

Figure 1 – Building collapse in Hong Kong, 2010

Table 1 − Accidents during the buildings reconstruction

Description of accident	Reasons of accident	City, country / The date	Number of the victimsof the accident
A four-storey office building that was under reconstruction. Slab floor was collapsed.	Unauthorized planning of premises on the first and second floors.	Krasnoyarsk, Russia, June 15, 2009	3
Five-storey house of dormitory. Collapse of two entrances.	Deal of deterioration	Astrakhan, Russia July 22, 2009	2
Four-story building, during the reconstruction period. The three floors are destroyed.	Deal of overlappings deterioration	Prague, Czech Republic 10.02.2009	-
A dwelling house built more than half a century ago.	Repairs	Hong Kong, China January 29, 2010	5
The building adjoining the hotel "Kharkiv".	During the reconstruction period	Kharkov, Ukraine March 16, 2010	-
Eight-story house. Overlapping was collapsed.	During the reconstruction period	St. Petersburg, Russia, September 01, 2010	A few
Three-story house.	Repair work, which resulted in violations of bearing structures.	Dumyat, Egypt 01.02.2012	35
Five-story house.	Illegal construction in violation of safety rules.	Sian, China June 26, 2011	7

2. Accidents during the buildings reconstruction

Speaking about accidents during the buildings and structures reconstruction (table 1), it should be noted that the incidents of such accidents have increased significantly over the past few years. The works in many cases are carried out incorrectly, poor quality materials are used, negligence during reconstruction is also excluded.

On September 1, 2010, in St. Petersburg, on Ligovsky Prospekt, 145, the ceilings of the eight-story building was collapsed. The crash began from the roof, and ended in the very bottom [10].

Quite often, the accident objects are those buildings that are under reconstruction. For example, on March 5, 2003 in Moscow, Russia, the construction of a multifunctional shopping center collapsed when dismantling brick diaphragms (pylons) that were located around staircase cells [11].

3. Building accidents at a stage of construction and acceptance in operation

For a more detailed study of this issue, the information collection and analysis on accidents in the stage of construction was also carried out. Materials are obtained through a variety of information sources, Internet resources and the media. In the process of work also was familiarized with scientific works on the accidents statistics and their typing. Based on the information received, the crash of new buildings was thoroughly analyzed and systematized in the table form. The list of accidents covers the worldwide timeline between 2003 and 2016 (Table 2).

On February 23, 2015 in Chernyakhovsk, Russia, the wall of an unfinished building collapsed, whose construction was suspended for a significant period. As a result of the incident, a 11-year-old boy was killed, during the collapse a plate fell on the boy. The unfinished building was in private ownership, after reviewing the event place, a decision was made to initiate a criminal case [12].

The accident carries not only significant economic losses, but also can take human lives. Thus, in India, 71 people died, including 25 children, as a result of the building collapse that was in the construction process. According to the Indian television channel NDTV, the tragedy occurred near the city of Mumbai on April 6, 2013. A seven-storey residential building construction was carried out illegally, in the absence of the necessary documentation, which would confirm the work safety on the site. According to law enforcement officials, despite the fact that the building was erected illegally, and its construction is not completed, the four floors have already been settled by the inhabitants. The probable disaster cause was the poor construction and building materials quality. The building collapse caused the destruction of the entire structure. Eyewitnesses say that the seven-story building has developed in 3 – 4 seconds, like a card house [13].

The trend was confirmed in December 2012 in Waghol City, where 13 people died as a result of the unfinished building collaps, and earlier in September, a building collapsed in Pune, Maharashtra, resulting in the deaths of six people [14]. On July 29, 2016, a part of the building that was in the construction phase collapsed in Pune, India. As a result of the incident, nine workers died.

Such an accident occurs throughout the world. For example, on March 29, 2013, in the city of Dar es Salaam, Tanzania, a 12-story unfinished building collapsed, killing 36 people. In relation to owners and construction contractors there is a criminal proceeding, in which nine people have already been arrested [13].

Not an exception to this and more developed countries, in particular, Russia. So, on August 15, 2015 in the Moscow downtown, a new building collapsed. As a result of the ceilings collapse between the first and second floors, two people were injured [15].

In Surgut, on March 6, 2014, a new building collapsed (Fig. 2). The ceiling collapsed between the fourth and fifth floors. Under the rubble, the saviors found three people, two of them died. Despite this, the media did not report any information regarding the opening of criminal proceedings, or the commencement of the investigating commission work at the incident scene [16].

Unfortunately, a country like Egypt has, in our time, also received a reputation like India in construction terms. Accidents before the time of acceptance buildings in operation in Egypt are not uncommon. Builders most often do not adhere to construction standards, exceeding the permissible number of floors or saving on the quality of materials. Sometimes construction is conducted at all without the permission of state bodies and departments [13].

In November 2012, in Alexandria, Egypt, 10 people were killed when the high-rise building under construction collapsed. Late in the evening, the eleven-story building collapsed into neighboring buildings. All the dead and wounded - the inhabitants of these houses [13].

Figure 2 – Building collapse in Surgut, 2014

An example is the accident that took place on January 16, 2013, in Alexandria, Egypt, where an eight-story dwelling house collapsed. The saviors freed 25 bodies from the rubble, 15 wounded were found. As the Alexandria governor said, the construction was carried out without the necessary documents, the municipal authorities did not issue a building company a building license [17].

Table 2 – Accidents of buildings and constructions at the stage of construction

Font Size	Description of accident	City, country	Date	Number of the victimsof the accidents
1	Collapse of the shopping center structures	Moscow, Russia	5.03.2003	-
2	The destruction of an unfinished 13-storeyd building	Shanghai, China	27.06.2009	1 person
3	The collapse of the unfinished building	Burundi	10.07.2009	14 died, more than 40 were injured
4	The collapse of the unfinished construction, which was almost ready for delivery	Dubai, United Arab Emirates	16.08.2009	-
5	The destruction of the 4-storeyd building shopping center. The exact cause of failure is unknown	Istanbul, Turkey	27.04.2009	-
6	Collapse of the hotel that was under construction process	Baku, Azerbaijan	28.04.2009	3 people
7	Collapse of the4-storeyed building that was under construction. Caused by poor quality of the construction materials.	Xi'an, China	02.10.2010	10 people were injured
8	Building collapsedduring the construction	Puna, India	September, 2012	6 people died
9	Building collapse	Alexandria, Egypt	November, 2012	10 people died
10	Unfinished house collapse	Vahholy, India	December 2012	13 people were died
11	The accident when constructing of a residential house. Reasons were the illegal construction, negligence, failure to comply with standards	Taganrog, Russia	13.12.2012	5 people died, 14 were injured
12	Destruction of 8-storeyed building. The reasons were failure to comply with standards, the illegal construction	Alexandria, Egypt	16.01.2013	25 people died, 15 were injured
13	The destruction of the 12-storeyed unfinished building	Dar Es Salaam, Tanzania	29.03.2013	36 people died
14	7-storeyed residential building collapse. Causes are negligence, the illegal construction	Mumbai, India	6.04.2013	71 people died
16	The destruction of the unfinished facility walls, whose construction was suspended. The reason was the frozen construction	Chrniakhovsk, Russia	23.02.2015	11-year-old boy died
17	Newly-built floors collapse of a building in the city center	Moscow, Russia	15.08.2015	2 people were injured
18	Destroyed building during construction	Tel Aviv, Israel	5.09.2016	2 people were injured
19	The collapse of the ceiling of an unfinished residential building	Ural, Russia	5.09.2016	1 person was injured
20	The collapse of the unfinished construction	Saransk, Russia	13.11.2017	2 people died, 3 people were injured
21	The collapse of the unfinished construction of the mall	Sumy, Ukraine	13.02.2013	-
22	Building collapse during the construction. The collapse of the newly-built floor construction	Kiev, Ukraine	19.11.2017	-

The acuteness of the illuminated problem can be clearly imagined if you explore the global information network. Only in one day around the world there were buildings collapses during their construction, as a result, many people were killed and injured.

For example, at 13:00 on September 5, 2016, the Israeli police press service announced a building collapse in Tel Aviv (Fig. 3) that was in the construction phase, leaving two people dead and five more missing. The mobile crane, which drove on the multi-storey car park roof on Ha-Barzel Street in the Tel-Aviv district of Ramatha-Khayal, dropped off the building part that could not bear the weight of a huge machine [19].

Figure 3 – Building collapse in Tel-Aviv, 2016

On the same day, at 17 o'clock, the press service of RIA VistaNews reported on the collapse of an unfinished residential building in the Urals (Fig. 4), resulting in serious injury to one of the workers. The incident took place in the Sverdlovsk region. According to preliminary information, the workers carried out the building structures dismantling of an unfinished dwelling house. During these works, the one floor overlap could not withstand the load and collapsed on the worker. At the moment, the commission operates on the scene of the accident, which, as a matter of urgency, must provide a legal assessment of the incident [17].

Figure 4 – The destruction building during construction, Ural, Russia, September 2016

4. Accidents due to the large age of buildings

During researching and analyzing the buildings and structures accidents, it is not impossible to avoid accidents that occurred due to the facility large age, or as a result of failure to perform timely repairs in buildings that need it.

A good example of inappropriate care for buildings can be the historical significance construction – the Cadet Corps, Poltava, Ukraine (Fig. 5).

This building was built in 1840, is currently inactive and is in a dilapidated state. The building reconstruction is not carried out, therefore the building is in a miserable condition, which in the future may lead to another accident in the construction industry. Moreover, such cases are not isolated, and unfortunately, are quite common in the Ukraine territory.

We give additional examples of this type accidents. Namely, in January 2010 in Tbilisi, Georgia, there were just two accidents. At first, the carrier wall of a residential three-story building collapsed, a day later - carrying two-story structures. In both cases, the buildings were in a emergency state. It should be noted that emergency measures were not carried out before the collapse. Fortunately, there are no victims [7].

Figure 5 – The appearance of the Cadet Corps in Poltava at present, and in the XIX century

On October 26, 2010, a residential building was partially destroyed in the Kirov region, Sovetsky.

The load-bearing wall collapsed, followed by stairs marches and inter-floor overlays. The pre-war building needed major repairs, the means for repairs were allocated slowly. People were not affected by the accident [21].

Also on the basis of the processed material a table was created describing the accidents and structures requiring repair work (Table 3).

The problem of studying accidents in buildings and structures is incomplete information about certain accidents. In the finding process in the various sources of necessary information, it has to be repeatedly encountered with the illuminated problem incompleteness.

Table 3 − Accidents of buildings and structures requiring repair work

Description of accident	City, country	The date	Number of the victimsof the accidents
Collapsed bearing wall of a residential three-story building. The building was in an emergency. No collision preventive measures were taken.	Tbilisi, Georgia	January, 2010	-
The wreck of the emergency wings is destroyed. The building was declared emergency. The inhabitants were evicted.	Odessa, Ukraine	March 21, 2010	
Partially demolished dwelling house. The load-bearing brick wall collapsed, followed by stairs and blanking. The building needed major repairs.	Sovetsk, Russia	November 26, 2010	-
Collapse of a three-story building that was in an emergency. The destruction occurred due to repairs that were carried out in the neighborhood.	Barletta, Italy	October 3, 2011	4
The seven-story building, which was in an emergency, was destroyed.	Luxor, Egypt	February 11, 2011	15 died, 20 were injured
Collapsed unoccupied emergency facility located near low-rise buildings.	Alexandria, Egypt	14 July, 2012	15
A five-story building collapsed. The cause of the accident was the cracking of the old building, formed as a result of heavy rains.	Beirut, Lebanon	15 January, 2012	27 died, 12 were injured

5. The largest destruction of buildings and structures in the world

Cases of large-scale accidents, which resulted in the investigation and compiled the relevant conclusions, undoubtedly cover the problem under consideration, but there are also such accidents that are suppressed for various reasons, one of which is the commercial struggle in the market between housing and construction companies. Sometimes the accidents coverage is reported incomplete, with inaccurate information, or unidentified the incident causes. All these factors affect the processing information quality. Therefore, it is also advisable to mention accidents, the causes of which have not been established, but which resulted in between one to ten injured.

This includes the a five-story dwelling house collapse on August 4, 2011 in the Pakistani port city of Karachi, where 29 people died [23] and a three-story dwelling house collapse on October 25, 2009 in Palma de Mallorca, Spain. Under the building rubble the five people died and two were injured in various severity degrees [6].

On June 10, 2012 in Lutsk, Ukraine, a five-story residential building collapsed (Fig. 6) – the bearing walls from the first to the fifth floor between the first and second entrances were destroyed. Rescuers have rescued from the building 18 people. As a result of the tragedy, two people were killed and one was injured [22].

In areas with difficult climatic conditions, as a rule, there is a high probability of a building accident, therefore, the requirements for the facilities construction in these areas are set more stringent. However, it is difficult to prevent the building of possible floods or other cataclysms.

Such accidents also include accidents that occurred due to design failures, such as in January 1978 in the city of Harford, Connecticut, USA, due to overloading with snow in the city center, where a hockey match was conducted during the day, overnight collapsed on the night from a height of 30 m a sports arena measuring 92 by 110 m (Fig. 7). The investigation revealed errors in the calculations of designers [23].

Figure 6 – Building collapse, Lutsk, Ukraine, 2012

Figure 7 – The fall of a sports arena in the city of Hartford (USA), 1978

Also such an accident occurs due to untimely repairs, such as January 15, 2012 in Beirut, Lebanon, where a five-story building collapsed. The reason for the steel collapse were cracks formed as a result of torrential rains. 27 people were killed, 12 wounded [10].

It is impossible to ignore the most serious accidents over the past two decades, which resulted in dozens, but not hundreds of casualties and thousands of wounded. These include the Sampoong shopping center collapse in Seoul (South Korea). On June 9, 1995, one of South Korea's largest buildings - the largest supermarket in Seoul, Sampoong, collapsed. Under the building ruins, 502 people died, 937 were injured and serious injuries. According to the investigation, it was discovered that a building whose collapse lasted only 20 seconds collapsed due to a number of reasons, the main of which were violations of building codes (Fig. 8).

Figure 8 – Collapse of the Sampoong shopping center, Seoul, South Korea, 1995

One of the reasons for the collapse building was the center's leadership decision to put on the roof three huge industrial air conditioners. In 1993, they were placed on a roof on special pallets, thus adding a load on a so weakened central part of the building (Fig. 9) [24].

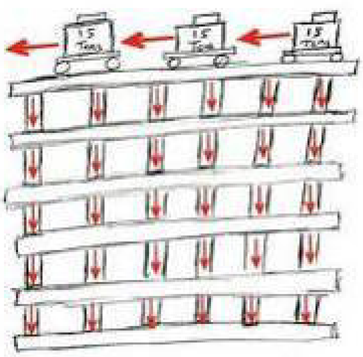

Figure 9 – The scheme of placement on the roof of three huge industrial air conditioners

Large-scale accidents in the construction industry cannot be attributed to the destruction of the shopping center «Maxima» in Riga (Fig. 10), which happened on the evening of November 21, 2013 in the district of Zolitude. Approximately at 5:45 pm, the roof and the supermarket walls deformed, numerous customers and workers were locked inside. At 18:00, one of the center walls fell and the roof over the ticket offices fell. At noon on November 23, the number of deaths reached 52 people: 51 Latvians and one Armenian citizen. The Latvian police put forward three versions of the disaster: 1) violation of the design; 2) violation of the rules of construction; 3) storage on the roof of building materials [25].

Figure 10 – The collapse of the Maxima Shopping Center in Riga, 2013

The record number of dead and wounded in the last decade has been recorded in 2013, when in Sawar (Bangladesh) on April 24, a complex containing a bank branch, a shopping center with lots of stores and five sewing factories was destroyed (Fig. 11). On May 9, the death raised to 953 people, more than a thousand people were injured.

Figure 11 – Collapse of the complex in the city of Savar (Bangladesh), 2013

On May 3, Interior Ministry experts have established the building collapse reasons: a strong vibration from powerful electric generators. Four giant generators were installed in the building in violation of all the rules, and when they re-started after the electricity was switched off for some time, their vibration, together with the vibration of thousands of machines, led to the collapse of the building [26].

The most massive accident in the past few years during which the roof collapse has occurred, Transworld Park, a sports and entertainment complex in the Yasenevo district (Moscow), opened in June 2002, which collapsed on February 14, 2004, can undoubtedly be considered [27].

6. Proposals for buildings accidents classification

Studying the accidents statistics and the characteristics in construction, a number of eminent researchers have been trying for decades to create a unified, well-founded classification of this type. But the goal set before the scientists is so unrestricted in the study, as in the implementation methods.

For the most part, accident statistics are currently being conducted in the most obvious way, namely, the accident information collection in tabular form, with the indicated reasons, the injuries number and the incident date. If the accidents collection covers the international territory, the general table is supplemented with information about the country where the incident occurred. Such a collecting information method can be defined as a general one. It allows you to summarize all the processed data from various resources and sources, on the basis of which the accident rate charts can be constructed depending on the selected indicators: crashes by type of building, destroyed structures, places (countries or cities) or number of victims people

The next part of the information statistical processing is a more detailed resulting general table breakdown by the objects type that have been destroyed. For example, the accidents types can be divided into three components: the buildings and structures destruction at the construction stage, in the objects reconstruction and the accident due to the large building age. Classification is precisely on these grounds due to the high repeatability level during the study of this issue, which implies that the probability of such an accident occurrence is highest.

On the research conducted basis, graphs and charts are created, which reflect the results obtained, which are already making final conclusions.

An example of generalized data processing is the annual accidents statistics, created by the Russian company «City Center of Expertise». The peculiarity of this company's work 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 can be read by anyone. At the same time, official statistics, which is conducted by state authorities, do not have access to ordinary citizens. On this basis, there is a need to address the work transparency issue of the Commissions investigating accidents in buildings and structures.

The possibility of providing public information can be a significant step in addressing accidents that occurred during the construction phase, as the publicity of incidents and work results carried out by the special commission will be a major impetus for the elimination of accidents certain types.

In addition, the data statistical processing on accidents building objects, makes it pay attention to the high-rise buildings problem, which are decommissioned, but not later dismantled. The authorities often do not pay attention to their accident rate and the destruction highest probability. The result of long-term dismantling, and in most cases, its complete absence, can become human life.

If for some time the accident was considered as a probabilistic event, which has no regularities and whose results cannot be predicted, then at present scientists have made a tangible breakthrough in this field of knowledge. With the introduction of such concepts as economic and non-economic consequences, the development and implementation of possible losses calculations, depending on the design failure.

The approach to the accident description can be considered with its probability. That is, an accident may be probable, impossible or accidental (Fig. 12).

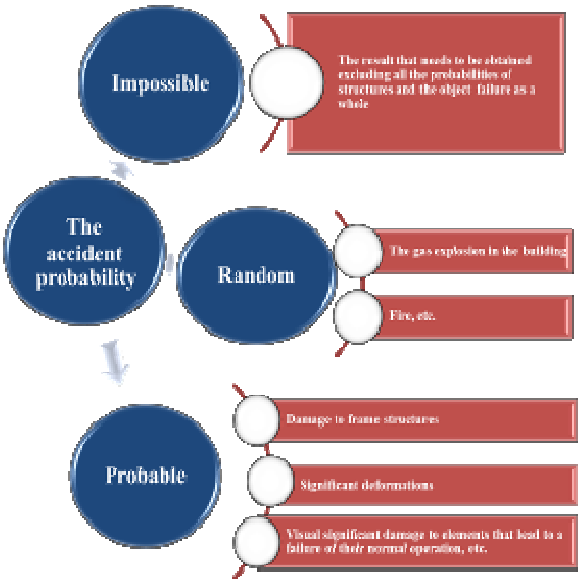

Figure 12 – Classification of accidents on the probability of their occurrence

These are three fundamental features that make it possible to differentiate the event and its progressiveness. That is, there is a certain antinomy of concepts: chaos, irregular series of events - in this case, the objects construction and their exploitation - acquires a regular order only when we narrow the range of statistical selection. Thus, moving from macro to micro-research, we create more complex statistics, which includes a clear understanding of the probability, impossibility or chance of an event.

Here are examples of situations in which the accident was probable. In this case, this is an accident near the city of Mumbai, April 6, 2016 [13]. The collapse of a seven-storey residential building provoked a number of reasons, such as a violation of building codes, negligence in the construction, illegal construction works. The probability of emergence of an emergency situation was the maximum in this case.

Accidental accidents include the explosion of gas in a residential building in Brussels, which took place on March 18, 2017, resulting in the loss of one person [28]. One building collapsed completely, from the other only the facade remained. Or the fire that occurred on February 21, 2015, in the OAU, where the tallest Fakel building fired [29]. No one was hurt.

The result of the accidents analysis that occurred in construction should be the impossibility of an accident. A striking example of working out the past years' experience, the implementation of necessary improvements and the various accidents types prevention is the modern complex «Federation», which consists of two skyscrapers of 324 meters high (Fig. 13) [30].

Figure 13 – Modern complex «Federation», Moscow, Russia

The building is equipped with cutting-edge technology, and is the highest in Europe and the strongest in the world. The hard frame "Federation" is designed in such a way that the output from the work of one element does not affect the normal work of the entire design. The experience of past years with the problems of fire safety and explosive environment introduced the latest high-tech designs. This facility serves as a vivid example of effective work on building mistakes.

Conclusions

The result of the research is the classification of accidents of buildings and structures based on the collected and processed material. Thus, the paper presents an attempt to generalize accidents by their type (period of operation of the building), as well as the proposed classification for the probability of an accident. For the appropriate calculation of the frame of the projected building, it is necessary to simulate a number of probable and probable accidents with their thorough elaboration. The result of this design is to reduce the probability of an accident, which is why the most vulnerable skeleton locations (depending on a variety of factors) are subject to reinforcement and careful work. Further investigation of accidents in construction allows using the classification presented to predict and eliminate potential emergencies for buildings of different types and destinations.

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Survivability and failure risks of steel frame structures: conceptual framework

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The article considers the concept of "survivability" of steel frame structures and defines its features. In the design of steel frames there is a need to reserve the main load-bearing structures to prevent progressive destruction. With the possible destruction of any individual element, the entire object or its most critical part must remain operational. The degree of damage to the system in case of failure of an individual element is determined. The main prerequisites for prevention of destruction in emergency situations, in particular, the calculation of the increase in carrying capacity. The approaches to determining the risks of failure and strengthening of steel frame elements are considered.

Keywords: survivability, destruction, redundancy, risk, damage.

Живучість і ризики відмови сталевих рамних конструкцій: понятійний апарат

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У статті проводиться аналіз роботи сталевих статично невизначених рам використовуючи моделі з максимально наближеними до реальних конструкцій діючими за нормами навантаженнями при можливих відмовах окремих елементів. Розглядається визначення поняття «живучість» сталевих рамних конструкцій. При проектуванні сталевих рам існує необхідність резервування основних несучих конструкцій для запобігання прогресуючих руйнувань. При можливому руйнуванні будь-якого окремого елемента весь об’єкт або його найвідповідальніша частина повинна зберігати працездатність. Визначається ступінь пошкодження системи при відмові окремого елемента. Визначені головні передумови запобіганню руйнуванню при аварійних ситуаціях, зокрема, розрахунок величини збільшення несучої здатності. Представлені умови граничних станів при розрахунках живучості багатоповерхових будівель. Приводяться конструктивні заходи для забезпечення стійкості каркасів. Представлені системи діафрагм жорсткості висотних будівель. Проведені розрахунки ряду сталевих рам. Результати показують, що поодинокі відмови елементів конструкцій ведуть до руйнування ряду перетинів. Це унеможливлює розгляд лавиноподібного прогресуючого руйнування. Аналізуються підходи до визначення ризиків при відмовах і підсиленні елементів сталевих рам. Представлені межі нормативного ризику аварії. Розраховується гранично - допустимі ризики відмови конструкції. Визначається фактичний ризик аварії і рівень достатньої конструкційної безпеки об'єкта. Ресурс об’єкту можливо подовжувати підсиленням елементів що відмовили, але в межах гранично допустимого ризику. Підсилення виконується обмежену кількість разів з урахуванням амортизації. Вартість робіт по обстеженню та підсиленню визначають залежно від ризиків можливих втрат при відмові (аварії) та в порівнянні з вартістю об’єкту.

Ключові слова: живучість, руйнування, резервування, ризик, пошкодження.

Introduction

European and Ukrainian practices for solving survivability problems require detailed study and effective solutions. One of the reasons is the lack of a common calculation method in the design of buildings and as a consequence there is an imperfect regulatory framework. There are a number of documents in the regulatory framework of Ukraine. Some of these standards indicate the need to calculate the survivability [1, 2] and are used to perform most of the calculations in the design. These documents are advisory. The need to ensure survivability in technical systems requires the development of analysis and evaluation of mechanisms methods and means of its provision for each specific class of systems. In foreign norms as comparative characteristics for the calculation of the vertical element refusal such as columns or pylons, engineers-designers are offered a very specific restriction of the collapse - 70 m^2 or 15 % of the area of the floor.

Review of research sources and publications

General concepts of risks and survivability of complex systems including building structures are presented in a number of scientific papers [3-6]. The work [3] presents a novel classification framework for severe global catastrophic risk scenarios. Extending beyond existing work that identifies individual risk scenarios, authors propose analyze global catastrophic risks along three dimensions: the critical systems affected, global spread mechanisms, and prevention and mitigation failures. The classification highlights areas of convergence between risk scenarios, which supports prioritization of particular research and of policy interventions. It also points to potential knowledge gaps regarding catastrophic risks, and provides an interdisciplinary structure for mapping and tracking the multitude of factors that could contribute to global catastrophic risks. The paper [4] is introduced the concept of system survivability under attack in analogy with system reliability. Authors limit consideration to the discrete case and define a component/system survivability to be the probability that the system/component continues functioning upon attack.

The differences between the suggested concept of system survivability and the traditional one of system reliability are defined. Most often, the survivability follows a Bernoulli distribution for which the survival probability is derived based on the system configuration. Authors develop results for series, parallel, series-parallel, parallel-series and k-out-of-n systems. It also provided the expected number of attacks for each system configuration based on the particular attack strategy both for single and multiple attacks. Scientists illustrate the process through a real application. According to [5] extreme events often cause local damage to building structures and pose a serious threat when one or more vertical load-bearing components fail, leading to the progressive collapse of the entire structure or a large part of it. Since the beginning of the 21st century there has been growing interest in the risks associated with extreme events. The accent is now on achieving resilient buildings that can remain operational after such an event, especially when they form part of critical infrastructures, being occupied by a large number of people, or are open to the public.

This paper [5] presents an ambitious review that describes all the main advances that have taken place since the beginning of the 21st century in the field of progressive collapse and robustness of buildings. Widely diverse aspects are dealt with, including: a collection of conceptual definitions, bibliometric details, the present situation and evolution of codes and design recommendations, quantification of robustness, assessing the risk of progressive collapse, experimental tests, numerical modeling, and research needs. The work [6] determines the strongest determinant of the destruction or endurance; some other factors such as inundation height, depth of the building parallel to the tsunami direction and opening ratio have also been considered as the factors supporting the survival. This paper investigates Sendai sewage purification center which survived the tsunami in the context of its endurance.

The issue of survivability and risks of steel frame structures devoted works [7-12]. The paper [7] presents a numerical model for analyzing steel frame structures subject to localized damage caused by blast load and subsequently investigating their survivability under fire attack. The proposed numerical method adopts a mixed-element approach for modeling large-scale framework and it is proven to be sufficiently accurate for capturing the detailed behaviour of member and frame instability associated with the effects of high-strain rate and fire temperature. Design implications related to the use of various numerical models for separate assessment of blast and fire resistance of steel structures and their components are discussed. Fire–blast interaction diagrams are generated to determine the fire resistance of columns considering the initial damage caused by the blast loads.

A multi-storey steel building frame is analyzed so that the complex interaction effects of blast and fire can be understood and quantified. The frame is found to be vulnerable, as it possesses little fire resistance due to the deformation of key structural elements caused by the high blast load. The paper [8] presents results of an investigation into the effect of span length on progressive collapse behaviour of seismically designed steel moment resisting frames which face losing one of their columns in the first story. Towards this aim, several nonlinear static and dynamic analyses were performed for three frames designed for a high seismic zone considering various span lengths. The analysis results revealed that beams and columns of the studied frames had adequate strength to survive one column loss in the first story. However, in order to determine the residual strength of the frame, a series of nonlinear static analyses called pushdown analyses were performed. It was shown that by decreasing the span length to half, the strength of the studied frames increases 1.91 times based on the performance-based analysis perspective. Besides, results of nonlinear static analyses revealed that by increasing the applied loads, the investigated structures are more susceptible to progressive collapse when they lose an internal column. Three frames have been analyzed in [9] with capacity design concepts taking into account shear capacity, flexural capacity and contribution from floor reinforcement to beams. Maximum inter-story drift ratios obtained from time-history analyses are plotted against ground motion intensities. Results are statistically interpreted to develop cumulative distribution functions for frames. Fragility curves are plotted for damage states of conventional structures. Fragility curves thus drawn are used to estimate the expected annual loss (EAL) of low rise RC frames using quadruple integral formula based on probabilistic financial risk assessment framework. Depending on the extent of damage, the fire resistance rating of the structure could be significantly reduced.

The paper [10] is devoted to obtaining some quantitative information about this topic, with reference to steel moment-resisting frames, even if the adopted methodology could also be extended to either different structural types or structural materials. As a first step, a simplified modeling of earthquake-induced structural damage, based on the superposition of geometrical and mechanical effects, is proposed. Then, a wide numerical analysis is performed with reference to a single-bay single-storey frame structure, allowing the main parameters affecting the problem to be identified. Finally, two multi-storey plane frames, designed in accordance with methods specified by Eurocodes, are analyzed as a case study.

In [11] a numerical procedure has been developed to model the sequences of failure which can occur within steel beam-to-column connections under fire conditions. In this procedure two recent developments, a static–dynamic solution process and a general component-based connection element, have been combined within the software in order to track the sequence of local failures of the connections which lead to structural progressive collapse in fire. In particular the procedure developed can be used to investigate the structural behaviour in fire, particularly the ductility and fracture of different parts of the steel-to-steel connections, and the influence of the connections on the progressive collapse resistance of steel frames in fire.

In the component-based connection model, a connection is represented as an assembly of “bolt-rows” composed of components representing different zones of mechanical behaviour whose stiffness, strength, ductility and fracture under changing temperatures can be adequately represented for global modelling. The potential numerical instabilities induced by fractures of individual connection’s components can be overcome by the use of alternate static and dynamic analyses. The transfer of data between the static and dynamic analyses enables a seamless alternation between these two procedures to take place. Accuracy and stability of the calculations can be ensured in the dynamic phase, provided that the time steps are set sufficiently small. This procedure has the capacity of tracking the local failures sequence (fractures of connection components, detachment and motion of disengaging beams, etc.) which lead to final collapse.

Following an illustrative case study of a two-bay by two-storey frame, the effect of ductility of connections on the collapse resistance of steel frames in fire is demonstrated in two case studies of a generic multi-storey frame. It is shown that the analytical process is an effective tool in tackling the numerical problems associated with the complex structural interactions and discontinuous failures which can affect a steel or composite frame in fire, potentially leading to progressive collapse. It can be seen that both tensile and compressive ductility in the connections make a contribution to the fire resistance of the beams. Preventing the detachment of steel beams in fire can be achieved by inducing greater ductility into their connections. Combined with appropriate component-based connection models, this procedure can be adopted in performance-based fire-resistant design to assess the ductility requirements of steel connections. Detailed finite element modelling of key elements is necessary to improve the robustness assessment of structures subjected to a coupled effect of fire and blast loads.

The paper [12] presents a method for a realistic multi-hazard approach by studying the residual load bearing capacity of steel columns under fire conditions and followed by an explosion. The approach adopts the use of a material constitutive law able to take into account both the strain rate sensitivity and the thermal softening. Explicit nonlinear dynamic analyses are performed using the explicit commercial code. Results show that the residual load bearing capacity is influenced by the stand-off distance. The time of fire loading at which an explosion is triggered is a critical parameter as well. High strain rates in the typical blast range are numerically obtained as a consequence of explosions in the close proximity. A comparison with the Eurocode approach is also reported. The results can be of great interest to establish the initial conditions that could potentially lead to the onset of progressive collapse in steel framed structures subjected to a combined effect of fire and blast loadings.

A static push-down analysis [13] is conducted experimentally using a 1/3 scale one-story bare steel moment frame substructure in this study. The objectives of this test include: investigating the behavior of bare steel moment frame under column loss scenario; validating the computational models developed for the purpose of investigating progressive collapse of steel frame structures. The contributions of collapse resisting mechanisms including flexural action and catenary action to the robustness of the system as the increase of the vertical displacement of the center column are quantified. The test results reveal that flexural action plays an important role in resisting progressive collapse along the entire loading process. However, the catenary action becomes the primary collapse resisting mechanism in the final stage of loading. Dynamic responses of the test specimen are estimated using energy-based method. It is shown the test specimen behaves elastically subjected to sudden loss of the center column and therefore progressive collapse will not occur. The dynamic increase factor is also estimated on the basis of the testing results. The analysis results suggest that catenary action has a great impact on the value of the dynamic increase factor under large deformation conditions.

At the same time, the problem of ensuring the survivability of structures in emergency situations has been studied for a long time [14]. Substantial research, conducted since about 1990, So, in [15] gives a General analysis of this problem. As a result of these studies, certain recommendations have been made for certain types of structures concerning the establishment of emergency parameters and constructive measures to prevent "progressive" destruction.

Definition of unsolved aspects of the problem

In the literature, not enough disclosed questions on the formulation of the term "survivability", not presented a single algorithm for calculating the survivability of building structures. Also, the literature does not take into account the dynamic components of the load on steel redundant frame.

Problem statement

The main problem in the work is the use of analytical methods in the study of approaches to determining the survivability of steel statically indeterminate frames in case of failure of individual elements. One of the tasks is to consider common approaches to assessing the risks of failure of structures and their corresponding strengthening.

Basic material and results

In the course of the study, the concept of "survivability" was defined. This property of the object to maintain limited working capacity under influences not provided for by the operating conditions, in the presence of some defects and damages, as well as the failure of some components of the object. As a rule, all parts of the object and the object as a whole should be calculated taking into account the limit States of the first and second groups. When considering emergency design situations, it is allowed to calculate only the main load-bearing structures of category A1 according to the limit States of the first group.

The technical system has the ability of survivability thanks to the built-in internal and external means of ensuring survivability (means of performance control, means of emergency protection). Survivability as an internal property of the system can be manifested in large external influences that are not provided for by the conditions of normal operation and under normal operating conditions, when there are failures of elements caused by operational defects, aging and other factors.

The main bearing structures of the objects of the classes of consequences (responsibility) CC3 and CC2 should be designed so that in an emergency the probability of avalanche (progressive) destruction, incomparably greater than the initial structural damage, is sufficiently small.

According to the source, the survivability of building structures is defined as the preservation of the bearing capacity or performance of structures in case of failure of one or more elements. Under the survivability of the building is understood to exclude the collapse of the entire building or its part with the sudden destruction of individual elements of the carrier system from the action of explosive waves or strikes when hitting vehicles, falling aircraft and other similar cases. There are two types of collapse: progressive collapse of the building and the loss of the overall stability of the building. Safety of building structures has led to the study of the properties of survivability - ensuring the stability of buildings and structures to emergency actions, to progressive collapse [16].

The most common is the definition of survivability properties as the system ability to adapt to emergency situations, to resist harmful effects, while performing its target function by changing the structure and behavior of the system. Depending on the degree of complexity of the organization and the class of systems, as well as the level of analysis, the property of survivability can be manifested (and, accordingly, quantified) by the same indicators that characterize the stability, strength, reliability, adaptability and others. According to the main positions of the theory of systems at the solution of a question in probabilistic statement the level of its survivability raises. This is done by improving the reliability of the system.

Survivability models can be stochastic, within the framework of the modern mathematical theory of reliability, or deterministic, within the framework of catastrophe mechanics. The probabilistic model describing the survivability of the system is called "load-strength" ("load - bearing capacity", strength model). Under the influence of an external load, the "strength" of the system gradually decreases until the system fails. External loads are described by a random function. For the rational justification of the damage magnitude which the construction is steady to the last, the necessary theory of risk, this enables to associate a probability of damage certain value occurrence and damage which may cause failure.

It is considered the survivability of building structures with possible destruction. According to [17], survivability is understood as the property of an object, which consists in its ability to resist the development of critical failures from defects and damages in the installed system of maintenance and repair, or the property of an object to maintain limited performance under influences not provided for by the operating conditions, or the property of an object to maintain limited performance in the presence of defects or damages of a certain type, as well as in the failure of some components."

There is no generally accepted term "structural survivability". Under the "survivability of the structure" is proposed to understand its property to maintain the overall bearing capacity at local destruction caused by natural and man-made impacts, at least for some time. This problem is directly related to ensuring the stability of structures of buildings and structures of "progressive" collapse in beyond design basis emergency damage and local structural damage. When designing critical structures, it is necessary to develop a system of preventive safety measures that reduce the emergency impacts risks. In addition, it is necessary to identify the "key" elements of the supporting structure which failure inevitably entails avalanche-like structure destruction, and to ensure the ability of such elements to perceive emergency effects without destruction.

Justification of the structures ability to withstand "progressive" destruction is carried out on the basis of calculation. The most accurate nonlinear calculation of structures considers the actual operation of the material and the system as a whole. Calculation of structures for resistance to "progressive" destruction is proposed as follows. At the first stage, the design is calculated at the operational stage (or in several installation and operational stages, considering the physical and geometric nonlinearity. At the second stage, the scheme is calculated with the elements removed from work. The calculation is also carried out considering the physical and geometric nonlinearity. If it turns out that some elements of the model do not meet the condition of strength (that is, they are destroyed), the calculation continues in the same way in the next stage without such elements. The calculation is completed by complete destruction of the carrier system.

However, it should be noted that in most cases, to prevent "progressive" destruction of the structure, it is necessary to provide the carrying capacity of all its elements in the initial emergency damage. In these cases, the calculation is stopped at the calculation first stage and the calculation second stage and "progressive" destruction process modeling is not necessary. The proposed method of calculation, in fact, is a computer simulation of a critical situation and enables to trace the adaptation of the structure to the new situation on the basis of changes in the design scheme. The designer on the basis of this calculation is able to identify a number of constructive measures to prevent this type of destruction.

The example of calculation of a high-rise building at local destruction caused by removal of an average column is given. This calculation enables to ensure the stability of the building structure to "progressive" destruction in case of emergency failure of building frame one of the columns. This can be done by a small increase in the percentage of reinforcement. According to the linear-elastic calculation, the number of longitudinal reinforcement of crossbars required for the perception of emergency action and the loads applied to its moment is about 3.5 times higher than the number of reinforcement necessary to ensure the bearing capacity of crossbars at design loads and impacts.

As a result of the two-stage calculation of the frame, taking into account the geometric and physical nonlinearity of the necessary reinforcement of the crossbars, it turned out to be 29% less. One-stage nonlinear calculation showed results similar to the results of two-stage calculation, but the required number of reinforcement bars was 10% more. Thus, a careful calculation analysis of the load-bearing system of the building allows to reveal additional reserves of its load-bearing capacity and with certain structural measures that require some increase in material consumption, it is possible to ensure the stability of the building to "progressive" destruction. In addition, it is possible to reduce the material intensity of the bearing structures of the building by taking into account the beyond-design emergency effects of those structures that in the design state of the building, with minor deformations, are not load-bearing, and with significant deformations of the bearing system due to emergency exposure, can be included in the work on the perception of the existing loads on the building.

Therefore, the sustainability of the constructions to the "progressive" destruction is part of the General problem of survivability of the structure. The problem of fire resistance of load-bearing structures, as well as the problem of meeting the requirements of seismic resistance, even in the case of construction of critical structures in areas with weak seismic activity, adjoins here. Consider the concept of" survivability " for high-rise buildings. High-rise buildings are buildings with an increased level of responsibility, so ensuring their reliable survivability is a priority. The survivability of a high-rise building is provided by a number of factors: the right choice of the design scheme, measures against progressive collapse, special techniques, fire resistance, seismicity, the use of appropriate materials and structures.

In high-rise construction, both traditional structural systems (frame, frame, cross-wall) and special ones used only in the construction of high-rise buildings (trunk, box, "pipe in pipe" and their combination) are used. The highest survivability of a high-rise building is provided by the cross-wall system. In addition, this system allows to achieve significant savings in materials of load-bearing structures [20]. This does not mean that only the above-mentioned structural systems should be used for all high-rise buildings. This problem should be solved individually in each case, depending on the whole complex of architectural, structural, installation and operational tasks.

To ensure the necessary survivability of a high-rise building, it is necessary to take into account the probability of local destruction of its supporting structures, which should not lead to a progressive collapse of the building. The calculation of the stability of the building must be made on a special combination of loads, taking into account the following schemes of local destruction: the destruction of two intersecting walls of one floor in a circle of 80 m2; failure of columns (pylons) with the walls adjacent to them, on the same area of local destruction; the collapse of the overlap of one floor on the above area. In some cases other schemes of local destructions can be accepted. In high-rise buildings are dominated by monolithic and precast-monolithic reinforced concrete floors, which are connected with other load-bearing structures should provide for the perception of the weight of half the span of the overlap.

Consider the concept of "survivability" of buildings and structures. There are measures to ensure survivability in emergency situations that should be recorded in the project documentation and known to the personnel responsible for the operation of the facility, as well as provided with appropriate instructions for supervision and maintenance of structures.

The building structure and substrate should meet the following requirements: to accept without damages and deformations unacceptable impacts arising during construction and within the prescribed period of operation; have sufficient capacity to perform under conditions of normal use during the entire installed life, namely, their operational parameters (displacements, vibrations, etc.) with a given probability should not exceed the established regulatory or project documentation limits, and their durability should be such that deterioration of materials and structures as a result of rot, corrosion, abrasion and other forms of physical deterioration did not lead to an unacceptably high probability of failure; to have sufficient survivability against local destruction and in compliance with the standards of emergency situations (fires, explosions and the like), excluding the progressive collapse phenomenon, when the overall damage is much larger than the initial perturbation that caused them.

The operating conditions components corresponding to the normal operation of the object effect depending on the equipment operation, atmospheric influences and others. Hazardous impacts should be considered throughout the construction and operation of the facility. The spatial unevenness and frequency of these impacts should be considered in the assessment of impacts. If hazards cannot be accurately predicted, it is advisable to consider them for safety reasons [16].

The structural safety position of a construction object imposes restrictions on the amount of the actual risk of the buildings, structures and structures accident. To the main part of the situation applies to the area of admissible values, the accident risk which boundaries are regulatory and limits the risk of accidents (Fig. 1). As long as the object accident actual risk remains within that area, the level of structural safety is considered sufficient.

The main purpose of the provisions introduction on the accident risk magnitude is to ensure the construction projects maximum possible safe resource and service life. The Figure 1 shows the whole set of standard accident risk values (R_n, R_{ma} i R_m). Therefore, if the risk of accident inherent in the object before its commissioning, normative (R_n), prevention of gross errors in the operation of the object, the safe resource (T_S) and service life (T_L) of this object is the greatest possible values, depending on the building structural type. In the presence of the provision, a principal opportunity is provided through planned examinations, during which the actual risk (R_a) is measured and changes associated with aging and wear are detected, and through preventive measures (strengthening, repair, etc.) that reduce the accumulated risk amount and cyclically increase the object safe resource (Fig. 1).

The object durability most significant indicators are its safe resource. If at the end of a safe resource, repair and restoration measures to reduce the risk of an accident at the facility are not carried out, then the value (T_L-T_S) is the time of the dangerous existence of the facility. However, during this period of life, the resistance of the object overload is reduced and (T_L-T_S) resource use can lead to an accident, and hence to losses that are disproportionately higher than the cost of preventive measures. Position on the accident actual risk magnitude plays the role of the regulatory framework in the implementation procedures of technical regulation the accident risk for the purpose of extending the safe service life of building objects. At the same time, the greatest effect is achieved through the regulation of the accident risk at the early stages of the object life cycle - the design and construction stages - designated in the law on technical regulation as declaration and certification.

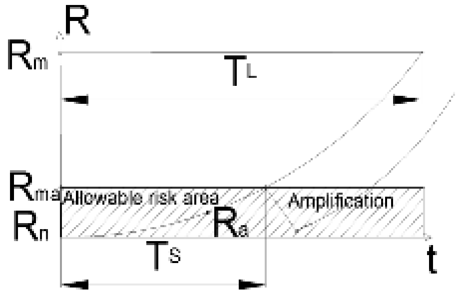

Figure 1 – Possible risk of accident and the resource object

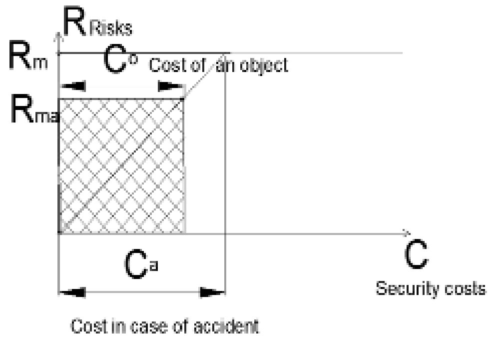

Figure 2 – Definition of safe risk taking into account the cost of the object in conventional units

Based on the economic feasibility, when deciding on the implementation of strengthening structures, cost in case of accident C_a (loss) and cost of an object C_o should be analyzed. Therefore, the inequality should be fulfilled

$$C_a \le C_o \,. \tag{1}$$

In case of structures failure (accident), the risk of loss should not exceed the failure:

$$R \le R_{ma} \approx C_o \,, \tag{2}$$

where R – the potential risk of failure of the structure; R_{ma} – maximum allowable risk.

Conclusions

As a result of the study, it has been found that the survivability of steel statically indeterminate frames can be increased by improving the reliability of both individual elements and the system as a whole. The resource of the object can be extended by strengthening the failed elements. But it can be done within the maximum permissible risk. It has been proved that the cost of inspection and strengthening is determined depending on the possible losses risk in case of failure (accident) and in comparison with the object cost.

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Probability of brick structures destruction

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The results of calculations of construction brick destruction probability determination at central and off-center compression are given; the determination of the brick structures destruction probability, caused by the exhaustion of the masonry strength on local compression (crushing); determination of the brick structures destruction probability associated with the exhaustion of the masonry strength on the displacement (cut); determination of the brick structures destruction probability associated with the exhaustion of the masonry strength on the fold, the stretch; determination of the brick structures destruction probability on the crack opening. The value of the security characteristic for each case has been determined and the comparison with the normative values has been made.

Keywords: limit state of the system; integral model of reliability estimation; the Monte Carlo method; refusal model; system reliability; system "building - base"; brick construction.

Ймовірність руйнування цегляних конструкцій

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У статті розроблено методологію розрахунків, яка би дозволила позволила виконати ймовірність руйнування цегляних конструкцій. Наведено результати розрахунків визначення ймовірності руйнування конструкцій з цегли при центральному та позацентровому стиску; визначення ймовірності руйнування конструкцій з цегли, пов’язаного з вичерпанням міцності кладки на місцевий стиск (зминання); визначення ймовірності руйнування цегляних конструкцій, пов’язаного з вичерпанням міцності кладки на зсув (зріз); визначення ймовірності цегляних конструкцій, пов’язаного з вичерпання міцності кладки на згин, на розтяг; визначення ймовірності руйнування цегляних конструкцій на розкриття тріщин. При розрахунках вважаються відомими закони розподілу випадкових величин навантажень і впливів – для постійного, корисного, снігового, вітрового навантажень, температурних впливів та впливів від нерівномірних осідань. Випадкові значення максимальних розрахункових напруг у перерізах цегляних конструкцій знаходять в залежності від випадкових зусиль, що виникають у конструкціях від випадкових впливів. При розв'язанні даної задачі статистичної динаміки використовується функція граничного стану, рішення якої проводиться методом статистичних випробувань (Монте-Карло). Імовірнісна оцінка міцності конструкцій з цегли здійснюється на основі виконання розрахунків по визначенню ймовірності руйнування за умови вичерпання міцності цегляної кладки. Визначено значення характеристики безпеки для кожного випадку та виконано порівняння з нормативними значеннями. Показано, що значення ймовірності руйнування цегляних конструкцій за базовий строк служби знаходиться в діапазоні 1×10^{-5}… 1×10^{-4}, що відповідає мінімальним значенням надійності, рекомендованим чинними нормами та Єврокодами. Розходження між запропонованим підходом та нормативною методикою складають не більше 8% - 10%.

Ключові слова: граничний стан системи; інтегральна модель оцінювання надійності; метод Монте-Карло; модель відмови; надійність системи; система «будівля – основа»; цегляна конструкція.

Introduction

In Ukraine, brick house construction is a significant part of the total housing fund. The number of brick buildings that are in critical condition is constantly increasing, The major repairs, reinforcement of structures are demanded. The accidents causes analysis in brick structures of the «building-base» system indicates the need to control and manage the reliability and operational suitability at all stages of the life cycle.

Review of research sources and publications

Today there is an obvious need for new science-intensive models for assessing the reliability of building structures both at the level of design and operation, and in normative regulation.

It is considered that the constructions, designed in accordance with the current rules, have sufficient level of safety, the quantitative security parameters are not set. The reliability assessment remains at the designer's level of intuition.

Only since 2009, with the introduction of DBN B.1.2-14:2009 [1], the assessment of the building elements reliability as the main criterion for the level of safety, has become obligatory in the design. However, the practical implementation of the provisions [1] is slowed down by the lack of the corresponding apparatus in the design standards – for almost 10 years of the DBN's effect, the Ukrainian norms of the construction industry did not lead (with a small exception) not only algorithms for determining reliability at the design stage, but also recommendations for the choice of materials and loads statistical models, which is fundamentally important. The security issue of the "building-base" system is also problematic. For many years, the scientific works in this direction were aimed exclusively at the development of models of load-deformed state analysis; the probability side of the problem for the most part is open.

In the Eurocodes system, a number of normative documents has been developed that regulate the probabilistic approach to building constructions and structures [2 - 3]. Problems of reliability and safety assessment of building structures are considered in the works of O. Weinberg [4], V. Rayzer [5], A. Perelmuttr [6], O. Lichov [7], A. Lantuch-Lyaschenko [8, 9], S. Pichugin [10], N.P. Hoej [11], R. Sêco e Pinto [12], A.U. Ebenuwa [13] and many other domestic and foreign scientists.

It should be noted that in the Ukrainian scientific sphere of residential and industrial construction there are currently no studies devoted to the stochastic modeling of the «building-base» system during the operation – researches that enable to determine the resource of the system at the random time of the life cycle.

Definition of unsolved aspects of the problem

The given analysis of the problem state enables to state that the research devoted to the scientific search for models of reliability and safety assessment of the "building-base" system is relevant, corresponds to the needs of the present in society, opens the way for a realistic assessment of the technical condition of residential and industrial buildings, has a significant socio-economic effect.

Problem statement

One of the parts of the research cycle is the development of a calculation methodology that enables to determine the probability of the brick structures destruction.

Basic material and results

The article is devoted to determining the probability of brick structures destruction. In accordance with the new normative documents, the author has proposed the following algorithms: determination of the brick structures destruction probability at central and off-center compression; determination of the structures made of brick destruction probability, associated with the exhaustion of the masonry strength on the local compression (crushing); determination of the brick structures destruction probability associated with the exhaustion of the masonry strength on the displacement (cut); determination of brick structures destruction probability associated with the exhaustion of the masonry strength on the fold, the stretch; calculation for opening cracks.

An algorithm for determining the brick ctructures destruction probability. Execution of the calculation is reduced to the determination of the destruction probability of the most dangerous cross section in the brick structure of those or other influences. In this case, it is necessary to determine the value of maximum tensions in the structural cross sections, which are due to the current loads, which can be represented by probabilistic and deterministic quantities.

The laws of the distribution of random quantities of loads and influences are known (Table 1) [14].

Random values of the maximum calculated tensions in cross sections of brick structures are based on random efforts that arise in designs from random influences. While solving the problem of statistical dynamics, the function of the boundary state is used, which solution is made by the method of statistical tests (Monte-Carlo).

In order to estimate the probability of the boundary state occurrence of a particular type, the problem of the reliability theory is solved with involving the apparatus of probability theory. At this case, the sequence of solving this problem is as follows:

– the condition of the brick element (construction) strength is considered;

– the parameters of the random variables distribution, which are the initial data, are determined;

– the solution to the problem of the probability assessment of the brick structures destruction is made.

Probabilistic assessment of brick structures strength is carried out on the basis of calculations that determine the probability of destruction on the condition of strength exhaustion of the brickwork. [15].

Table 1 – Existing tensions and influences, accepted in the model of brick structures failure

Type of load	The designation	Units	Average value	Variation coefficient V_x	The distribution law
Constant load	G	kN	G	0,1	Normal
Payload (50 years)	Q	kN/m^2	$0,6Q$	0,35	Gumbel
Snow load (50 years)	S	kN/m^2	$0,7S_0$	0,5	Gumbel
Wind load	W	kN/m^2	$0,75W_0$	0,35	Gumbel
Temperature impact	T	C^o	T	0,15	Normal
Uneven settlement impact (effort in the foundation)	D	kPa	P	0,59	Normal

To do this, it is necessary to determine the distribution of the tensions and the boundary resistance of the brickwork on the compression, local compression (crushing), stretching (axial and bending) and on the cut (on the bound and unbound cross sections).

An algorithm for determining the brick structures destruction probability on the compression by Monte-Carlo method is proposed as follows.

1. N statistical tests are performed.
2. Random probabilities of calculated loads are assigned: from construction own weight P_G, from snow P_S, payload P_Q and loads from uneven subsidences P_D.
3. According to the known values P_G, P_S, P_Q, P_D the quantiles of loads are determined, from own weight G, snow S, payload Q, uneven subsidences D.
4. The random value of the vertical load N_{Ed} . is determined
5. The random probabilities of the strength of the brickwork for compression P_{fd}. are determined.
6. By the value of P_{fd} the quantiles f_d and the value of the vertical strength of the wall N_{Rd} are determined.
7. The value of the boundary state function Y at the central compression can be written by the formula:

$$Y = N_{Rd}\,(f_d\,,\, b,\, t) - F\,(\gamma_{constr},\, q,\, S_0,\, A',\, E,\, a,\, b) - \\ - F_1\,(\gamma_{constr},\, q) \geq 0 \qquad (1)$$

with off-central compression:

$$Y = f_d \cdot A_c\,(b,\, t,\, e_0) - N(\gamma_{constr},\, q,\, S_0,\, F,\, F_1,\, e_0) \geq 0; \qquad (2)$$

8. The condition $Y \geq 0$ is checked.

Graphs of the vertical load distribution function N_{Ed} and the vertical strength of the wall N_{Rd} and graphs of distribution probability density of vertical load N_{Ed} and vertical strength of the wall N_{Rd} are shown on the fig. 1, 2.

In equations (1) and (2) F, F_1 are the load values from the above floors of the building and the load on the current floor, respectively; b, t – geometric dimensions of the brick structure, e_0 – eccentricity, γ_{constr} – specific gravity of structures, S_0 – characteristic value of snow load, E – the absolute value of deformation; A' – cargo area, A_c – area of the compressed zone of the cross section; N – applied load.

In the general case, all input parameters of the equations (1, 2) are random variables. However, some values, such as: the geometric dimensions of the cross section, the size of the structure can be considered as deterministic quantities. In accordance with the rules of the brickwork arrangement, the allowable deviations in the thickness of building structures are: for walls ± 15 mm, for pillars ± 10 mm, for the thickness of the solder seams -2…+3 mm. Compared with the size of the construction, this is a maximum of 5% of the cross section of the construction, therefore, the variability of the structure geometric dimensions can be neglected.

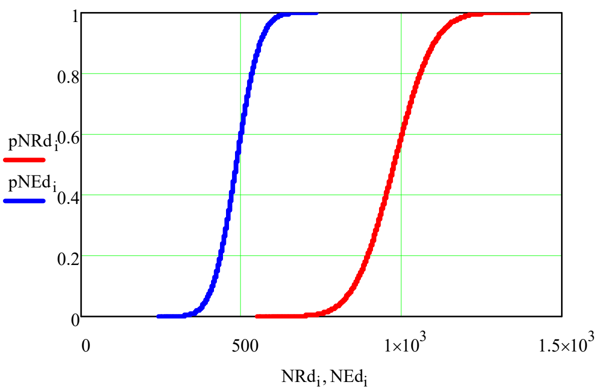

Figure 1 – Vertical load distribution N_{Ed} and vertical strength of the wall N_{Rd} functions

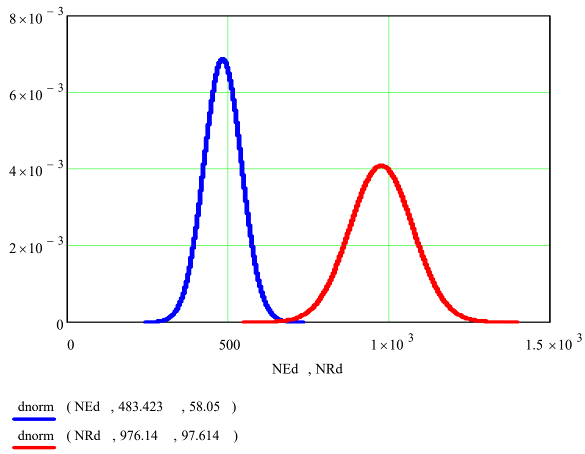

Figure 2 –Density of the probability of vertical load distribution N_{Ed} and vertical strength of the wall N_{Rd}

After performing all N tests, the probability of the brick structure element destruction during the estimated lifetime is calculated as the ratio of the number of tests n, for which $Y<0$, to all tests N. Consider that the number of tests N must be large enough to more accurately determine the value of Y, in this case, the number of tests has been taken as $N = 1\times10^5$.

Calculations have been made to determine the probability of the brickwork destruction; the following probability values for the various cases given in Table 2 are obtained.

Table 2 – Calculations results of brick structures destruction probability

Name of the quantities	Value	β	Difference, %
Probability of the brick structure destruction on the central compression	1×10^{-5}	4,27	+10%
Probability of the brick structure destruction on the off-central compression	$1{,}2\times10^{-4}$	3,7	-3,3
The probability of the brick structure destruction on local compression (crushing)	4×10^{-4}	3,51	-8,26
Probability of the brick structure destruction on the displacement (cut)	$1{,}8\times10^{-4}$	3,66	-3,825
Probability of the brick structure destruction to the fold on the bound cross section	$1{,}2\times10^{-4}$	3,7	-2,3
Probability of the brick structure destruction on the cracks opening	$1{,}4\times10^{-4}$	3,692	-2,48
Recommended minimum values by DSTU-N B V.1.2-13:2008 (EN 1990:2002, IDN) [16], **ISO 2394-1998** [17]	$\mathbf{1\times10^{-4}}$	**3,8**	

Conclusions

In the framework of reliability theory, a methodology for determining the probability of brick structures destruction as a result of achieving each of the possible boundary states has been proposed; algorithms and computer programs that are designed to solve such probabilistic problems by the Monte-Carlo method have been developed.

Calculations have been made to determine the probability of the brickwork destruction. The following probability values for different cases are obtained: Table 2.

The value of the probability of the of brick structures destruction for the base lifetime is in the range $1\times10^{-5}\ldots1\times10^{-4}$, which meets the minimum reliability values recommended by the applicable norms and Eurocodes.

The difference between the proposed approach and the normative method is no more than 8% - 10%.

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Improvement of settlement calculations of building foundations by increasing the reliability of determining soil compressibility indices

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Ways to improve the methods of calculating the foundations bases' settlements by increasing the reliability of determining the soil compressibility indices are substantiated. The complex approach to refinement of the buildings bases' settlements calculation by the layer summation method is investigated by accounting for the soil deformation modulus variability in the full pressure range perceived by the base at loading; soil strength coefficient β_Z; soil deformation anisotropy by elastic orthotropic model; tendencies to magnitude variation in the soil deformation modulus in depth of the body under the foundations and within the artificial bases built with the soil compaction. There was also proved the possibility of increasing the accuracy of the predicting method for the buildings' foundations base settling using the soil compression index and accounting for the pressure effect on the soil deformation parameters in depth of the compressible strata.

Keywords: settlement, method of layer-by-layer summing up, reliability, soil compression test, soil porosity coefficient, soil deformation modulus, soil compressibility index, anisotropy.

Удосконалення розрахунку осідань основ будівель підвищенням достовірності визначення показників стисливості ґрунту

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Проаналізовано сучасні підходи щодо удосконалення аналітичних методів розрахунку осідань основ будівель і споруд. Встановлено, що більшість з них мають свої певні області раціонального застосування стосовно ґрунтових умов, обладнання з визначення властивостей стисливості ґрунтів, видів і розмірів фундаментів чи штучних основ, задач проектування і не носять комплексного характеру. Обґрунтовано шляхи удосконалення методів розрахунку осідань основ фундаментів підвищенням достовірності визначення показників стисливості ґрунту. Розглянуто комплексний підхід до уточнення розрахунку осідань основ будівель методом пошарового підсумовування шляхом урахування: змінності модуля деформації ґрунту в усьому діапазоні тиску, який сприймає основа при навантаженні; коефіцієнта β_Z за міцністю ґрунту; деформаційної анізотропії ґрунтів за пружною ортотропною моделлю; закономірностей зміни величин модуля деформації ґрунту за глибиною масиву під фундаментами і в межах штучних основ, що зводяться з ущільненням ґрунту. Також доведено можливість підвищення точності методики прогнозування осідання основи фундаментів будівель використанням в ній показника стискання ґрунту й урахуванням впливу тиску на деформаційні параметри ґрунту за глибиною стисливої товщі. Подано приклади застосування аналітичних методик розрахунку осідань основ будівель.

Ключові слова: осідання, метод пошарового підсумовування, достовірність, компресійне випробування ґрунту, коефіцієнт пористості ґрунту, модуль деформації ґрунту, показник стискання ґрунту, анізотропія.

Introduction

Predicting the deformation magnitude of «Soil base – foundation – building (structure)» system is a complex, but at the same time a priority task when designing the foundations of buildings and structures. The reliability and cost-effectiveness depend on the reliability of the deformation calculation. Most of the abnormal deformations of buildings and structures are caused by errors in determining the compressibility parameters of the soil base.

Prof. Dalmatov considered the settlement of each foundation base to be the sum of five components [1]:

1) settlement due to the compaction of the natural structure soils at the increasing stresses from the foundations' weight;

2) settlement associated with decompression of the upper soil strata that lies below the bottom of a foundation ditch, due to the reduction of stresses during excavation;

3) settlement due to the soil squeezing (extrusion) from beneath the foundation caused by the progression of plastic deformations;

4) settlement of the disruption, which progresses due to the soil compressibility increase at its natural structure distortion during execution;

5) settlement caused by changes in the stress state or deformation of the soil base during the building (structure) operation.

In geotechnical practice, usually, only the first two components of deformation are considered. The buildings' foundations settlements are calculated using analytical methods (layer summation method (LSM), etc.) [2, 3] and finite element modeling (FEM) [2]. Particularly, in the LSM, recommended by the building codes for practical calculations, the settlement of the foundations' bases is determined using the calculated scheme of linear-deformed half-space by expression

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

where $\beta = 0.8$ is the coefficient taking into account the lateral extension of the soil;

$\sigma_{zp,i}$ is the average value of the additional stress in the i^{th} elementary soil layer;

h_i, E_i are respectively, the thickness and deformation modulus of the i^{th} layer of soil;

n is the number of elementary layers within the compressed strata under the foundation footing.

This method, with all its versatility and clarity, is, however, based on a number of assumptions [2 - 5], in particular: the soil is a continuous, isotropic, linearly deformed body; settlement is caused by the vertical stress action only, and other components are not considered; lateral expansion of the soil at the base is impossible; stresses are determined under the center of the foundation footing; when determining stresses, differences in the compressibility of individual layers are neglected [4]; the foundation has no rigidity; deformations are analyzed only within the compressed strata; lateral expansion of the soil is taken into account by the parameter $\beta = 0.8$ regardless of the type and condition of the soil, etc.

Comparisons show that the calculated values of settlements of the foundations of buildings are sometimes up to several times different from the values of stabilized settlements of full-scale objects [6]. The defining characteristics that affect the accuracy of the prediction of the base's settlements are the parameters of compressibility, in particular, the modulus of deformation of each soil layer within the compressed strata [1 - 6].

Review of research sources and publications

Geotechnicians have experimentally and theoretically substantiated a number of original techniques aimed at improving (increasing the reliability) of analytical calculations of the bases' settlements of the building foundations (whereas the LSM is taken as a base) by:

– determination of the soil deformation modulus magnitude and the compressive strata, with regards to the footing area of the large-scale foundations [7];

– introduction of the correction (increasing) coefficient (Agishev, I.A.) m_k to the values of soil deformation modules obtained from compression tests in the pressure interval of $\sigma = 0.1 – 0.2$ MPa (m_k value at liquidity index of $0.5 < I_L < 1$ for sandy loam, sandy clay and clay varies between 2 and 6 based on soil type and its porosity index e) [8]. However, the results of long geodetic surveys of the settlement of buildings and structures on loose [9] and wetted (degraded) soils [10, 11] showed that it is more correct to use the results of compression tests without increasing coefficient m_k;

– considering the variability of the soil deformation modulus over the depth of the compressible strata [12];

– considering the deformation anisotropy of soils (most often according to the ratio of soil deformation modules in the vertical and horizontal directions within the model of elastic orthotropic environment) [13, 14];

– accounting for the structural strength of soils when assigning values of soil compressibility characteristics and compressive strata capacity [15, 16];

– considering the variability of the soil deformation modulus over the full pressure range, which is perceived by the base at loading [17 - 20], ie the deformation modulus is a function of the stresses in the massif and the corresponding changes in the porosity coefficient. The parameters of the soil model can be set by interpreting the logarithmic function of the soil compression test data according to the speed of pressure transfer during the operation of the bases [17 - 19]. The disadvantage of the odometer is the low accuracy of measurements, because the friction forces of the sample behind the walls of the ring reduce by 10 - 50% (depending on humidity, type of soil, conditions of the experiment) the pressure applied to the sample, especially with increasing stresses in the soil.

This leads to a false increase in the actual magnitude of the deformation modulus [21]. Therefore, in order to avoid the high soil friction force behind the walls of the ring, especially at high pressure, and to ensure the sampling of the undisturbed structure, the «Ring for soil testing in one-dimensional deformation» is used [17];

– the use of so-called soil compression index (which reflects the relative change in the porosity coefficient of soil compression and more correctly than the standard parameters of compressibility characterizes the deformation properties of the base at specific stress values) and taking into account the effect of pressure on the deformation parameters of the soil and its porosity variation over the compressed strata depth [22, 23];

– application of probabilistic calculations of bases taking into account the parameters of the heterogeneity of soil massifs. According to this approach to determining the bases' settlement, in particular, it is established that there is a probability of linear and nonlinear stages of the base deformation when the pressure under the foundation footing exceeds the calculated soil resistance under the deterministic approach. This effect is caused by the heterogeneity of the physical and mechanical parameters of natural and compacted soils and the random nature of loads and influences on the foundations [24, 25];

– the use of soil deformation parameters obtained in three-axis compression devices, where it is possible to take into account the effect of changing the modulus of deformation as a function of horizontal stresses, which enables interpreting the obtained parameters with respect to over compacted soils, structural strength of strata and other specific properties of rocks, and accounting for elastic-plastic nature of a soil [26].

Definition of unsolved aspects of the problem

However, it should be noted that the original methods analyzed are fairly correct in themselves, but most of them have their specific areas of rational application for specific soil conditions, laboratory or field equipment to determine the characteristics of soil compression, types, and sizes of foundations or artificial basis, design problems, etc., and, most importantly, they are not complex yet.

Problem statement

Therefore, as the purpose of the work presented the improvement of analytical methods for calculating the settlements of the foundations' bases of buildings and structures was adopted utilizing a comprehensive account of the reliability increasing methods of determining the soil compressibility parameters.

Basic material and results

Consider a comprehensive approach to clarify the calculation of the buildings' foundations settlement by LSM taking into account:

1) the variability of soil deformation modulus in the full pressure range, which perceives the base during loading;

2) coefficient β_Z of soil strength;

3) deformation anisotropy of soils by elastic orthotropic model;

4) regularities of the soil deformation modulus change in depth of the massif under the foundations and within the artificial bases, erected with soil compaction.

Table 1 provides typical examples of determining the compressibility characteristics of clayey soil, depending on the standard degrees of pressure for different densities of loam. For example, according to the data of compression tests according to the standard method of DSTU B B.2.1-4-96 specimens of loess semi-solid loam with a natural humidity of 0.24 and a coefficient of porosity of 0.86 in the range of stresses 0 - 2.7 MPa an increase in the value of the deformation modulus from 3 to 22 MPa was established [17].

Table 1 – Variability of the deformation modulus values in the clay loam test

Place of soil sampling	Porosity coefficient e	Wetness w, %	Soil deformation module E, MPa, in vertical pressure intervals σ, MPa			
			0 - 0.05	0.05 - 0.1	0.1 - 0.2	0.2 - 0.3
in the massif of natural structure	1.45	25	0.765	1.06	1.19	0.53
	1.27	18	1.83	1.35	1.03	0.81
	0.74	20	5.43	4.85	9.52	6.25
	0.67	16	5.95	5.21	10.4	6.40
	0.56	15	5.56	6.49	7.81	13.0
in the ground bed	0.48	16	17.0	10.5	12.2	16.2
	0.46	14	24.1	11.4	14.0	22.8
for in-situ piles in drilled wells	0.60	19	13.2	9.7	12.7	9.80

Considering the variability of the soil deformation module, it is advisable to make the basis of the known relationship between deformations and stresses, which has the best statistics in power form

$$\Delta h = b(\sigma_i / \sigma_0)^a \qquad (2)$$

Concerning tests in expression (2) Δh is the specimen deformation under stress σ_i when conditional stabilization is achieved (this is a strain rate of 6.25 mm/h). Empirical coefficients: a – dimensionless value, which varies within narrow limits (for clay soils of Poltava a = 0.6 – 1.5). The coefficient b has a linear dimension that corresponds to the

Δh value and ranges in a much larger range from a few units to several tens. b value is closely related to the soil porosity coefficient e_0. By expression (2) the equation of the compression curve is corrected

$$e_\sigma = e_0 - \frac{b(\sigma_i/\sigma_0)^2}{h}(1+e_0) \quad (3)$$

where e_0 and e_σ respectively, the soil porosity coefficients at $\sigma = 0$ and $\sigma = \sigma_i$, and h is the height of the specimen.

At any pressure interval, based on (2) and classical expressions of soil mechanics, the coefficient of compressibility is

$$m_0 = \frac{\left[e_0 - \frac{b(\sigma_н/\sigma_0)}{h}(1+e_0)\right]}{\sigma_к - \sigma_н} - \frac{\left[e_0 - \frac{b(\sigma_к/\sigma_0)}{h}(1+e_0)\right]}{\sigma_к - \sigma_н} \quad (4)$$

where σ_H and σ_K are the vertical stresses at the beginning and end of the interval. After simplification we have

$$m_0 = \frac{b(1+e_0)\left[(\sigma_к/\sigma_0)^a - (\sigma_н/\sigma_0)^a\right]}{h(\sigma_к - \sigma_н)} \quad (5)$$

The given coefficient of compressibility is equal

$$m_v = \frac{b\left[(\sigma_к/\sigma_0)^a - (\sigma_н/\sigma_0)^a\right]}{h(\sigma_к - \sigma_н)} \quad (6)$$

Finally, the formula for determining the modulus of deformation looks like

$$E = \frac{\beta_z \cdot h \cdot (\sigma_к - \sigma_н)}{b\left[(\sigma_к/\sigma_0)^a - (\sigma_н/\sigma_0)^a\right]} \quad (7)$$

where β_Z is the coefficient which takes into account the absence of transverse soil expansion in the compression device (not to be confused with β coefficient in expression (1)) and which is calculated according to DSTU B B.2.1-4-96 depending on the transverse deformation coefficient (Poisson coefficient) ν. In the absence of research data DSTU B B.2.1-4-96 enables adopting ν depending on the type and condition of the soil.

β_Z coefficient depends on indicators of the physical and mechanical properties of the cohesive soil. It can be determined using formula [17]

$$\beta_z = \frac{0.5 \cdot \sigma_1 \cdot (1 - \sin\varphi_{II}) - c_{II} \cdot \cos\varphi_{II}}{\sigma_1 - c_{II} \cdot \cos\varphi_{II}} \quad (8)$$

where σ_1 is vertical stress acting under the foundation footing for conditions b = 0; φ_{II}, c_{II} are internal friction angle and specific adhesion for water-saturated bonded soil.

Then, for the LSM, the base settlement will be

$$S = \beta \cdot \sum_{i=1}^{n} \frac{(\sigma_n + \sigma_k) \cdot 0.5 \cdot h_i}{\beta_z \cdot h \cdot (\sigma_n - \sigma_k)} \cdot b \cdot \left[\sigma_n^a - \sigma_k^a\right] \quad (9)$$

Equation (9) is final and, given expressions (7) and (8), enables us to improve the calculation of the settlements of the foundation's bases of buildings.

Now, using expression (9), having the magnitudes of the additional stresses at the boundary of the elementary layers z, into which the compressible strata H_C is divided, it is sufficient to simply take into account the variability of the modulus of deformation by the nature of the additional pressure plot. Of course, if there are different layers of soil within the compressive thickness, the parameters a and b should be set separately for each layer.

Here is an example of the calculation: for individual foundations of different sizes with a depth of laying $d = 2$ m, under the footing of which the average pressure is $p = 250$ kPa. The base is a clayey loam that has the strength indicators of $\varphi_{II} = 22^o$ and $c_{II} = 15$ kPa in the water-saturated state. Table 2 contains the averaging data of six long-term compression tests that were performed until the standard conditional stabilization rate ($v = 6.24 \cdot 10^{-4}$ mm/h) was achieved. Sample height is $h = 35$ mm. The processing of the results by expression (2) gave the following parameters of the compression curve with a high correlation coefficient $r = 0.998$: $b = 5.87$; $a = 1.146$; $\Delta h = 5.87 \cdot \sigma^{1.146}$.

In the pressure intervals from 0.1 to 0.2 MPa and 0.2 to 0.3 MPa with $\beta = 0.5$ compression modulus, respectively: $E = 3.4$ and 3.2 MPa. These values are usually used to calculate the settlement by DBN B.2.1-10-2009. If we consider the subsidence of (9) accounting for the variability of E and β_Z depending on the magnitudes of the additional pressure, then we have the data contained in table 3.

Therefore, such an algorithm for analytical precipitation determination is recommended.

1. Compression testing of clay soil samples with the obligatory fulfillment of deformation stabilization conditions.
2. Approximation of compression results by dependence (2) and determination of parameters a and b.
3. Testing of samples of clay soil for displacement and determination of strength and further critical pressure.
4. Calculation of parameters of lateral extension β_Z and ν.
5. Calculation of additional pressure according to the DBN scheme B.2.1-10-2009.
6. Determination of the values of the modulus of deformation considering the curve of additional pressure within the compressive strata.
7. Settlement calculation based on the variability of lateral expansion parameters and deformation modules. Therefore, in comparison with the traditional method of using the modulus of general deformation, expression (7) together with the refined β_Z coefficient allows increasing the value E. This increase, for water-saturated clayey soil, gives grounds for limiting the use of coefficients m_k, which are taken into account in the transition from the compression module to the stamp. In each case, it is possible to account for specific indicators of the physical and mechanical properties of the soil, rather than constant values, which are assigned only by the plasticity number.

Numerous experimental data [14] of primary (natural) and secondary (induced, i.e., after compaction or consolidation of soil) soil anisotropy substantiate the need to account for this effect to clarify the calculation of sediment bases. Some fairly typical examples of soil deformation anisotropy from the authors' practice are shown in Table 4.

Table 2 – The results of the compression test of clay loam

	Vertical pressure σ, MPa						
	0.00	0.05	0.10	0.15	0.20	0.25	0.30
Deformation Δh, mm	0.00	0.18	0.45	0.67	0.96	1.20	1.40
Porosity index e	0.843	0.833	0.821	0.808	0.794	0.780	0.765

Table 3 – Calculation of settlements of the bases of individual square foundations by (9)

z, m													
	B	1.5 m			2.1 m			2.7 m			3.3 m		
	σ_{zp}	216.0 kPa			216.0 kPa			216.0 kPa			216.0 kPa		
	σ_1	231.3 kPa			223.8 kPa			216.4 kPa			209.0 kPa		
	β_z	0.803			0.805			0.808			0.811		
		σ^c, kPa	E, MPa	S, cm	σ^c, kPa	E, MPa	S, cm	σ^c, kPa	E, MPa	S, cm	σ^c, kPa	E, MPa	S, cm
0		168	5.43	2.5	187	5.36	2.8	196	5.33	2.9	203	5.23	3.11
1		83.7	6.02	1.11	118	5.58	1.69	147	5.60	2.1	160	5.52	2.32
2		35.3	6.82	0.37	60.3	6.32	0.76	85.2	6.03	0.71	107	5.86	1.46
3		22.2	7.3	0.13	33.9	6.87	0.39	51.5	6.48	0.64	69	6.24	0.88
4		$\sum S = 4.11$ (7.1)*			21.3	8.05	0.12	33.5	6.91	0.39	47	6.60	0.57
5					$\sum S = 5.76$ (10.2)*			25.1	7.10	0.14	33	6.94	0.38
6								$\sum S = 6.88$ (13.1)*			26	7.18	0.14

$\sum S = 8.86$ (15.6)* – in the brackets settling on the recommendations of the DBN B.2.1-10-2009

Table 4 – Soil deformation anisotropy coefficients

Type of soil	Place of sampling	Porosity ratio e	Wetness w, %	The deformation module E_-, МПа	Coefficient $n_{E\perp}$
loess loams	natural massif	1.10 1.07 0.87 0.83 – 0.96 0.825	16.5 22.5 27.5 13 – 20 19	1.5 3.7 5.4 2.8 5.6 – 6.0	0.93 0.91 0.87 0.7 – 0.9 0.7 – 0.9
clay loam	bulk foundation * bulk foundation **	1.0 – 1.05 0.8 – 0.86	24 25	1.9 2.8	0.75 0.86
clay loam	ground bed	0.44 – 0.57	14 – 21	13.7 – 18.7	0.62 – 0.89
loess loams	under the foundation footing***	1.04 0.85	22.5 27.5	3.9 6.9	0.92 0.82
clay loam	for in-situ piles in drilled wells	0.60	19	12.7	0.76

* – drop-out time of about 40 years; ** – a break time of more than 10 years;
*** – lifetime of about 100 years at the ratio of the average pressure under the foundation footing to the calculated soil resistance p/R ≈ 100%

The anisotropy coefficients were determined by the formula

$$n_{E\perp} = E_{\perp}/E_{-} \; , \tag{10}$$

where is $E\_$ deformation of the soil in the case of orientation of the rings when selected at an angle α = 0º relative to the horizontal plane; $E_{\perp}$ is the same but α = 90º.

It is proposed to consider the deformation anisotropy of soil soils by determining the additional pressure in formula (1) by the expression

$$\sigma'_{zp,i} = \sigma_{zp,i} / \sqrt{n_{E\perp}} \; . \tag{11}$$

In this case, for the conditions of calculating the sediment of a separate foundation, discussed above, at $n_{E\perp} = 0.8$ the settling value will increase by about 10%.

The regularities of changing the values of the soil deformation modulus at the depth of the compacted zone, the use of which increases the accuracy of calculating the settlement of the foundations, which are reduced with compaction of its bases, can be rationally obtained within the first stage of modeling using the complex “PRIZ-Pile” [17 – 19].

The following is also another comprehensive approach to improving the settlement calculation of the base of MPP foundations on the N_{pw} soil compression index. This indicator reflects the relative change in the porosity ratio of the sample in compression tests [22]. Soil compressibility depends on its initial porosity and, accordingly, the initial porosity ratio. Therefore, to exclude the effect of the porosity of individual samples on the compressibility characteristic of its size, it is advisable to determine it as the relative decrease in the porosity coefficient when compressing the soil sample by expression

$$N^i_{pw} = (e^i_0 - e^i_p) / e^i_0 = \Delta e^i_p / e^i_0 \qquad (12)$$

where N_{pw}^i is the index of compression of the i[th] sample;

e^i_0, e^i_p are the coefficients of porosity of the i[th] soil sample, respectively initial and after application of pressure p;

Δe^i_p is a decrease in the coefficient of porosity of the i[th] sample after applying pressure p.

N_{pw} parameter is called indicator of soil compression. It is defined as the average of the compression indices of the samples N_{pw}^i

$$N_{pw} = \sum_{i=1}^{n} \frac{e^i_0 - e^i_p}{e^i_0 \cdot n} = \sum_{i=1}^{n} \frac{\Delta e^i_p}{e^i_o \cdot n} = \sum_{i=1}^{n} \frac{N^i_{pw}}{n} \qquad (13)$$

where N_{pw} is an indicator of soil compaction of certain humidity w from the stress p.

Index of N_{pw} shows that it's determined at specific values of humidity w and the stress p. The compression ratio of the soil reflects the relative decrease in its porosity ratio during compression.

Settlement of the soil layer is expressed by expression

$$e_p = e_o - \Delta h / h (1 + e_o) \qquad (14)$$

where e_0 and e_p are respectively porosity the soil coefficients: initial and under the pressure p.

$$\Delta h/h\ (1 + e_0) = e_0 - e_p \qquad (15)$$

then the settlement Δh of the soil layer h is equal to

$$\Delta h = (e_0 - e_p)/(1 + e_0) \cdot h \qquad (16)$$

From formula (12), the reduction of the porosity coefficient of compression is

$$e_0 - e_p = N_{pw} \cdot e_0 \qquad (17)$$

Substituting (17) into formula (16), we have

$$\Delta h = h\ (N_{pw} \cdot e_0)/(1 + e_0) \qquad (18)$$

Substituting Δh by the thickness of the i[th] layer of soil S_i of thickness h_i, we obtain

$$S_i = N_{pw}^i \cdot h_i \cdot e_0 / (1 + e_0) \qquad (19)$$

From where the sum of the settlements of the individual layers can be described by the following expression

$$S = \sum_{i=1}^{n} S_i = \beta \sum_{i=1}^{n} N_{pw}^i\ h_i\ (e_0^i)/(1 + e_0^i) \qquad (20)$$

Where S_i is subsidence of the soil base from loading of the foundation; coefficient $\beta=0.8$; N_{pw}^i is the index of compression of the i[th] layer of soil; h_i is the thickness of the i[th] layer of soil.

The calculation of (20), expressed in terms of N_{pw} the subsidence of the LSM, takes into account the change in the stress state at the depth of the base from the load and the effect of soil porosity on its compression. The subsidence of the substrate was determined by two deformation parameters (the modulus of deformation and the compression ratio of the soil) obtained in the standard K-1 compression device and the CLES device (compressibility with a lateral extension of the soil) [22].

Comparison of the results of long geodetic surveys of settlements of a field object (section of a six-story building) and calculated by the index of compression of the soil and the modulus of deformation. The site belongs to the Poltava Forest Plateau. Groundwater level - 5 m from the daily surface. The following engineering-geological elements (EGE) are distinguished:

EGE-1 – bulk soil and soil-vegetation layer (1.4 m thick);

EGE-2 – loamy forest, solid, highly porous, subsidence (3.1 m);

EGE-3 – loamy forest, fluid-plastic (5.6 m);

EGE-4 – loam is rigid plastic (4,8 m);

EGE-5 is a loamy forest, microplastic (17 m deep).

Soil physical and mechanical characteristics are given in Table 5.

The average pressure under the sole of the tape foundations on a natural basis was p = 180 kPa at the calculated soil resistance R = 200 kPa. In the process of erection and operation of the building by Professor M.L. Zotsenko organized a geodetic survey of its sediments [19].

The calculations of the bases settlement of the foundations are made by the MPP (fig. 1) through 1) the soil deformation module; 2) the compression ratio of the soil, which is defined in each layer at the appropriate pressure at depth, taking into account the initial coefficient of porosity of this layer (fig. 2). Comparison of the calculated values of settlements of the foundation of the building by two methods with the data of field observations is shown in table 6.

Comparing the results of the calculations by the two methods with the data of the field observations, it was found that the subsidence of the basis determined by the new method is closer to the actual values than to the normative ones. The amount of subsidence of the soil by the soil compression index by 36% exceeds the value calculated by the modulus of deformation. At the same time, the subsidence of the base, determined by the soil compression index, is 13.8% higher than the value obtained from geodetic observations, and the settlement calculated through the deformation module of the soil is 26% less than the observation data.

Table 5 – The value of the physical and mechanical soil properties on the site

Soil characteristics	EGE numbers, value of characteristics				
	1	2	3	4	5
Humidity on the verge of fluidity	-	0,36	0,28	0,36	0,29
Humidity on the verge of plasticity	-	0,21	0,18	0,21	0,19
Number of plasticity	-	0,15	0,10	0,15	0,10
Humidity is natural	0,16	0,20	0,26	0,28	0,25
Fluidity index	-	-0,07	0,80	0,47	0,60
Specific gravity of soil, kN / m^3	15,00	16,60	18,66	18,47	18,79
Specific gravity of dry soil, kN / m^3	-	13,83	14,81	14,43	15,03
Porosity ratio	-	0,92	0,78	0,84	0,76
Soil specific gravity,γ_{II}, kN / m^3	-	16,46	18,52	18,33	18,62
Specific cohesion, c_{II}, kPa	-	11	8	16	9
The angle of internal friction, φ_{II}, deg.	-	25	28	26	28
The deformation module, E, MPa	-	6	8	11	8

Table 6 – The results comparison of the foundations' bases settlements of the house section with the data from field observations calculated by the two methods

№ EGE	Initial coefficient of soil porosity e_0	Soil deformation module *E*, MPa	Base settlement *S*, m, determined through the deformation module *E*	Soil compression ratio index N_{pw}	Base settlement *S*, m, determined through the N_{pw}	Base settlement *S*, m, determined by geodetic observations
2	0,92	6,0	0,088	7,62	0,138	0,119
3	0,78	8,0		6,45		
4	0,84	11,0		5,61		

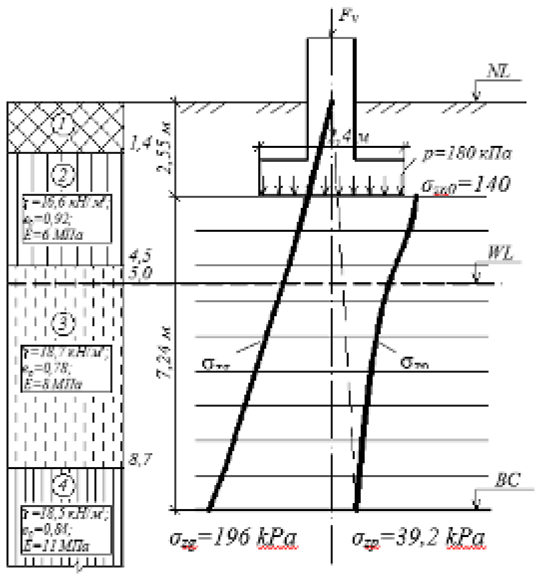

Figure 1 – The scheme to determine the settlement of the foundations' bases by the method of layer summation by the soil deformation modulus

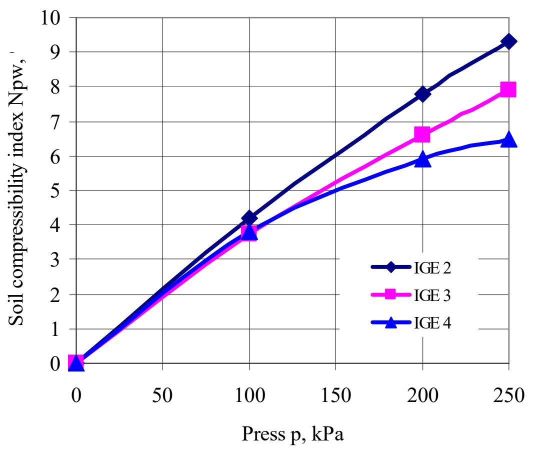

Figure 2 – Determination of soil compression indexes N_{pw} to calculate the base settlement

Conclusion

Thus, it is possible to clarify the calculation of the base settlements by LSM by a complex consideration: variability of the soil deformation module in the full pressure range, which is perceived by the base at loading; soil strength coefficient; deformation anisotropy of soils; regularities of change in the size of the soil deformation modulus in depth of the array under the foundations and within the artificial bases, which are reduced to soil compaction.

The developed compression index reflects the relative change in the porosity coefficient of soil compression, and more correctly than the standard parameters of compressibility characterizes the deformation properties of the substrate at specific stress values, which vary in depth of the compressive strata under the foundation, taking into account the effect of porosity on the compression. The accuracy of the method of determining the settlement of the foundations is enhanced using soil compression index. The settlement of the foundations, determined through this indicator, up to 27% exceeds, calculated through the module of soil deformation.

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Numerical simulation of hard airdrome coatings stress-strain state when interacting with weak ground base

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The technique and its numerical realization of the hard airdrome coatings specified calculations with the consideration of their interaction with heterogeneous multilayer weak soil half-space is proposed. Numerical simulation of the hard airdrome coatings stress-strain state is carried out on the basis of the relations of the elasticity nonlinear theory with the help of the finite element method momentary scheme. The task of calculating the actual concrete coating on a rigid artificial basis of the International Airport "Odesa" runway is solved. A finite-element model for calculating hard surfaces with the use of a momentary finite element scheme and a universal spatial shell finite element has been developed.

Keywords: finite element method, weak ground base, rigid airdrome coverage, numerical simulation, stress-strain state

Чисельне моделювання напружено-деформованого стану жорстких аеродромних покриттів при взаємодії зі слабкою ґрунтовою основою

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Запропоновано методику і її числову реалізацію уточнених розрахунків жорстких аеродромних покриттів з урахуванням їх взаємодії з неоднорідним багатошаровим слабким ґрунтовим півпростором. Чисельне моделювання напружено-деформованого стану жорстких аеродромних покриттів здійснено на основі співвідношень нелінійної теорії пружності за допомогою моментної схеми методу скінченних елементів. Розв'язана задача розрахунку реального бетонного покриття на жорсткій штучній основі злітно-посадкової смуги Міжнародного аеропорту «Одеса». Розроблено скінченно-елементну модель для розрахунку жорстких покриттів з використанням моментної схеми скінченних елементів і універсального просторового оболонкового скінченного елемента. Чисельний розрахунок напружено-деформованого стану жорстких аеродромних покриттів при взаємодії зі слабкою ґрунтовою основою виконаний на дію навантаження від літака B 767 -300. Розрахункова схема покриття побудована так, щоб було включене колісне навантаження від всього шасі повітряного судна з урахуванням того, що основна опора літака розміщувалася б на середній плиті розрахункового фрагмента. Аналіз отриманих чисельних результатів показує, що значення погонного згинального моменту під лівим нижнім колесом основної опори літака B767-300 досягає величини -99,724 кН·м/м, що відповідає напруженню в нижньому волокні умовної плити 950,88 кН·м/м^2. Розглянута конструкція жорсткого аеродромного покриття задовольняє умовам граничного стану та забезпечена міцність на можливі допустимі напруження. Розрахунок покриттів при колісному впливі крупно фюзеляжних повітряних суден при наявності включень слабких шарів ґрунту необхідно виконувати тільки на основі чисельних досліджень. Максимальне значення прогину при постійному коефіцієнті постелі становить 0,41 мм, а при змінному – 0,75 мм.

Ключові слова: метод скінченних елементів, слабка ґрунтова основа, жорсткі аеродромні покриття, чисельне моделювання, напружено-деформований стан

Introduction

The calculation and design of rigid airfield coverings have been carried out in accordance with the normative document [1]. However, modern literary sources [4, 9] suggest that the existing norms of calculation are clearly obsolete, since they do not consider the parameters of heavy-duty large body aircraft of type B 767-300 and others and the presence of weak layers of soil in the bases.

The basis of the existing regulations for calculating the rigid airfield coverage is the analytical relationship between the calculation of a continuous inseparable plate on elastic basis with the use of the direct proportionality hypothesis, with very close consideration of the transition to finite-size plates in the presence of cross-stitches and jump joint.

Review of research sources and publications

Since 2015, the volume of overhaul and new building of aerodrome elements is rapidly increasing in Ukraine (in the years 2015-2030, such airports in Boryspil, Odesa, Vinnitsa, Chernivtsi, Kherson, and Nikolaev are planned to be reconstructed).

These problems in airfield construction can be solved only by applying modern numerical methods for calculating the stress-strain state of hard airfield coverings before the construction or reconstruction of real aerodromes.

The forest soils, which occupy almost 75% of the territory of Ukraine, are in stressed state under the influence of wheel loading or their own weight of coverings and soil, they can sink and can reduce their structural strength when soaked [3, 9-11, 13-17]. In order to solve this problem and to improve the calculation of rigid airfield coverings in the presence of weak soil active layer of weak soil and soil heterogeneity inclusions, [4-6] modeling of the soil basis was proposed by a discrete non-homogeneous half-space, considering the discrete-local zones of state equations for anisotropic material , equivalent to real weak soils.

In the framework of existing regulatory documents, considering modern scientific researches, it is possible to consider the interaction of aerodrome coatings with inhomogeneous soil half-space using the functions of the variable coefficient of subgrade resistance [3; 4]. Such approach is consistent with the problem solution of research of rigid airfield coverings which mainly work on the bend and distribute the load on sufficiently large plane of soil half-space. These coatings should be considered as thin plates using discrete finite element models (FEM).

It is proposed to approximate the function of the variable coefficient of subgrade resistance by a trigonometric spline function in the local coordinate system on the median surface of the plate.

Problem statement

The purpose of the study is to test the strength of the actual airfield coverage using numerical methods.

Basic material and results

Numerical simulation of the stress-strain state of the rigid aerodrome coating in interaction with weak ground base is performed on the example of the International Airport "Odessa". The engineering-geological section with the largest thickness of weak soils is used as the source data.

In this case, geological data are selected from the most unfavorable conditions, which are shown in Figure 1.

The proposed coverage of the International Airport "Odessa" runways consists of the following constructive layers:

– top layer – high-strength concrete grade B40

cement concrete Btb 4,8/60,
R_{bn} = 29 MPa,
$R_{btn\ axial\ tension}$ = 2,1 МПа,
$R_{btb\ bending}$ = 4,1 MPa,
$E_b = 3{,}53\cdot10^4$ МПа,
ρ = 2500 kg/m^3,
ν_1 = 0,22;

– the bottom layer is a leached concrete grade B7,5

cement concrete Btb 4,8/60,
R_{bn} = 5,5 MPa,
$R_{btn\ axial\ tension}$ = 0,7 MPa,
$R_{btb\ bending}$ = 1,5 MPa,
$E_b = 1{,}6\cdot10^4$ MPa,
ρ = 2490 kg/m^3,
ν_1 = 0,23.

The artificial foundation is represented by such constructive layers:

– ground cement M75

R_{bn} = 3,5 MPa,
$R_{btn\ axial\ tension}$ = 0,55 MPa,
$R_{btb\ bending}$ = 1,3 MPa,
$E_b = 3{,}53\cdot10^4$ MPa,
ρ = 2500 kg/m^3,
ν_1 = 0,22;

– crushed stone from natural stone, enclosed by a method of decomposition with limiting compressive strength 100 MPa

$E_п = 4{,}5\cdot10^2$ MPa,
$K_s = 4{,}5\cdot10^2$ MH/м^3,
$E_b=3{,}0\cdot10^3$ kgp/sм^2;

– synthetic gauze

K_{se} = 16 kgp/sм^2,
$h_{тек.}$=20,0 sм.

Soil foundation has the following characteristics:

– layer from the element EGE-1(E_b = 672,2 kgp/sm^2, h_1 = 140 sм);

– layer EGE -2 (E_b = 672,2 kgp/sм^2, h_2 = 170 sm);

– layer EGE -3 (E_b = 525,15 kgp/sм^2, h_3 = 250 sm);

– layer EGE -4 (E_b = 266, 86 kgp/sм^2, h_4 = 90 sm).

Equivalent coefficient of subgrade resistance [7]:

K_{se} = 55,285 MH/м^3

In accordance with [1], this ground base belongs to the weak group.

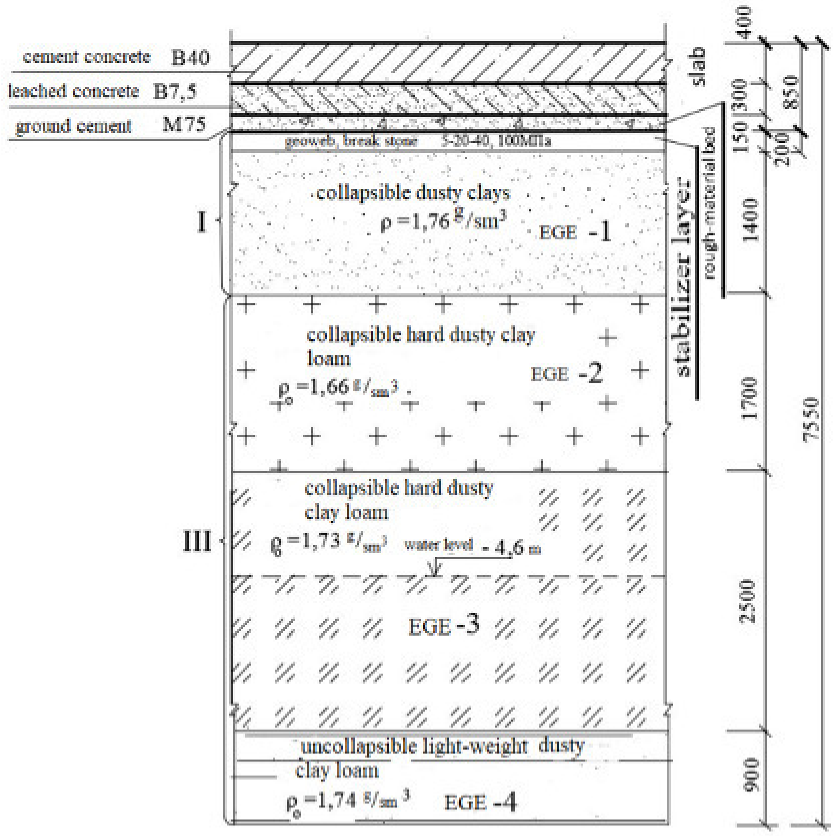

Figure 1 – Aerodrome coating construction consideing the active layer of soil foundation

Aerodrome plates, reducing the average pressure under the cover at the expense of a large area, sharply increase the depth of the compressed layer of the soil half-space, that is, attracting to the work of deep, but weak, strongly compressed layers of water saturated dust-loamy soils.

For the implementation of a refined numerical calculation of the coverage under the wheel load of the aircraft chassis one of the most versatile schemes of the FEM is used - the moment scheme of finite elements (MSFE) [5].

In this technique, a simplified mathematical model of elastic basis is used with the assumption of the proportionality between the positive plate deflection and the reaction of the base and depends on the coordinate of the pplate (node) median surface pointwhere the base deflection and reaction must be calculated

$$q\,(x^2, x^3) = -c\,(x^2, x^3)\;\omega\,, \qquad (1)$$

where $q\,(x^2, x^3)$ is the function of the proportionality factor (subgrade resistance);

ω – a positive deflection at this point along the normal to the plate surface in deformed state [8].

The coefficient of subgrade resistance function can be described by two-dimensional approximation [2] using a certain number of the coefficient values of subgrade resistance for the considered section of the geological section, tconsidering the multilayer soils of the underlying coating and its thickness, providing the value of the equivalent total deformation modulus in this intersection.

Knowing a number of coefficient values of subgrade resistance, depending on the average, the vertical of the base thickness, the general module of soil deformation and using one or another analytical function: spline, trigonometric, index, power, or other, a specific function in this area of the calculation model is obtained.

For example, in the presence of a lens-like soil layer with known the limit values of the subgrade resistance coefficient and, also using an analytic spline according to the law of the sinus, it is got

$$c_N(x^2, x^3) = C_0 - (C_{\max} - C_0)\sin\frac{\pi x_N^2}{l^2}\,in\frac{\pi x_N^3}{l^3}\,, \qquad (2)$$

where x_N^2, x_N^3 – are current local coordinates of nodes in the calculated fragment in the global coordinate system;

l^2, l^3 – are the sizes of the calculated fragment in the global axis coordinate system $z^{2'}$ and $z^{3'}$.

In the considered scheme of MSFE, the discrete model has two limiting surfaces - the lower and upper, that is, the plate is considered as single layer - conditional (equivalent to bending and longitudinal stiffness). An equivalent analogue of the conditional plate is constructed from the equivalence condition for the specified stiffnesses for the uniform size of the plate:

$$\begin{cases} E_{(e)} \cdot \dfrac{t_e^3}{12} \cdot 100 = EI_{(0x)}^0 \\ E_{(e)} \cdot 100 \cdot t_{(e)} = EF_{(0x)}^0 \end{cases}, \quad (3)$$

where $EI_{(0x)}^0$, $EF_{(0x)}^0$ – are the actual rigidities of the construction of a multilayer plate, with the bending (with respect to the coordinate system in the center of the weight of the plate, considering the rigid basis) considers only the upper and lower layers, and the longitudinal rigidity $EF_{(0x)}^0$ considers two layers and a rigid foundation. The model of the equivalent analogue of the aerodrome plate construction is shown in Fig. 2

In order to ensure the adequacy of the plates interaction in the calculated fragment and in the real construction, it is introduced at the slab arris joints - the tie-ins (Fig. 3).

The equivalent thickness of the aerodrome coating plate is 81 sm (t_e = 81 sm).

An equivalent modulus of elasticity for the section of the aerodrome coverage is 2,57·10^5 kgp/sm^2 , and an equivalent Poisson ratio – 0,2324.

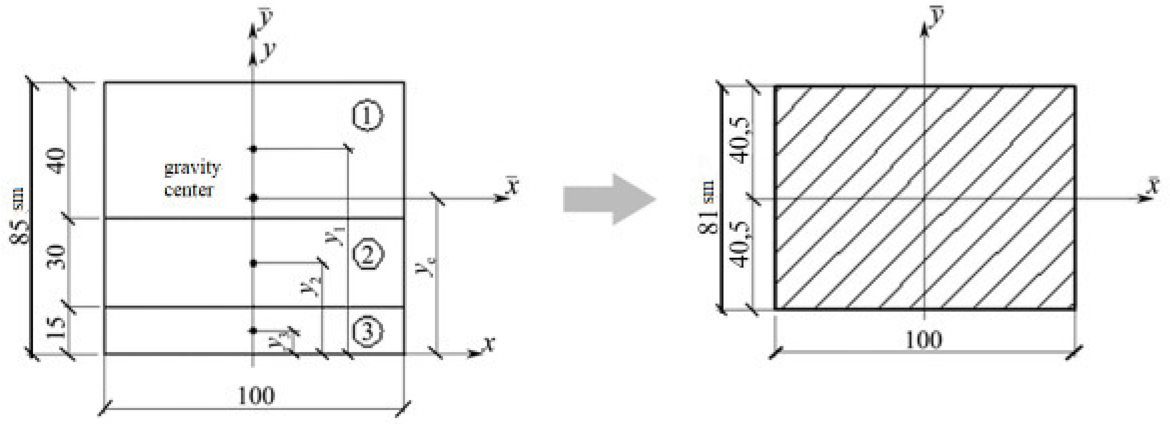

Figure 2 – The model is equivalent to the design of an airfield cover plate

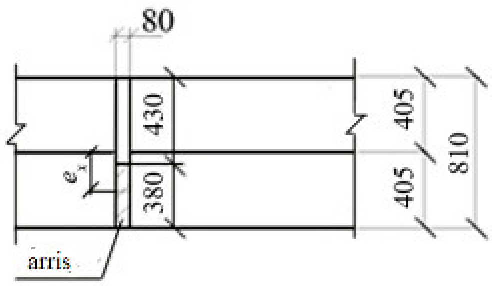

Figure 3 – Model of joints of aerodrome plates

Models parameters of butt inserts have the following meanings:

– the ratio of the rib thickness to the shell thickness:

$$\frac{h_p}{h_o} = \frac{38}{81} = 0.47;$$

– the eccentricity ratio to the shell thickness (in relation to the local coordinate system)

$$\frac{e_x}{h_o} = -\frac{21.5}{81} = -0.265;$$

– elasticity module of the insert material:

$E_{b(r)}$ = 2,6·10^3 kg/sm^2 ;

– poisson coefficient of insertion material $\nu_{b(r)}$ = 0.3;

– the ratio of the specific gravity of the insert material to the equivalent specific gravity of the airfield cover:

$$\frac{\gamma_{b(r)}}{\gamma_{cp}^e} = \frac{0.0016}{0.002527} = 0.633;$$

– insertion width: $b_r^{np} = 8$ cm.

As a rated aircraft, Boeing 767-300 has been adopted.

The parameters of the round wheel print are calculated by the formula:

$$F_d = \frac{F_n}{n_k} \cdot k_d \cdot \gamma_f, \quad (4)$$

where F_d – is estimated load on the wheel;
n_k – number of wheels of the main support;
F_n – load on the main support (F_n =724.9 kH);
k_d – dynamic factor (k_d =1.25);
γ_f – unloading factor (γ_f = 1.0).

Then F_d =226.53 kN.

The square footprint for the Aircraft B767-300 is 43 cm.

The calculation scheme of the coating is constructed so that the wheel load from the entire chassis of the aircraft was included, considering that the main bearing of the plane is placed on the middle plate of the calculated fragment.

Considering the symmetry of the Aircraft B767-300, a discrete model containing six plates of a runway covering 10×7.5 m has been constructed. The calculation scheme and the finite-element model of the calculated coating fragments for the Aircraft B767-300 are shown in Fig. 4

Equivalent discrete prints of wheel pneumatics with a pressure p_a = 1,21 MPa according to the finite element model presented, are located on one medium plate, that is, on four discrete areas of uniform surface loading, brought to the nodal.

The discrete model is constructed so that the main four-wheel support is located in the middle of the calculated fragment middle slab, with the aircfaft chassis parameters. Fragments of the topological model of wheel prints in the coating calculation scheme are described by the grid coordinates S_2, S_3 – the beginning and the end of the wheel load.

The dimensions of the grid area are: $M1\times M2\times M3$, that is 2×30×46, and the size of the calculated fragment – 15000×30000 mm. Total nodes in the FE-model, presented at fig. 4 - N_u = 2×30×46 which corresponds to the equilibrium equations system $k_p = N_u \cdot 3 = 2760\cdot 3 = 8280$, (without consideration the imposed connections) and the number of finite elements –

$$M_e^p = (M2-1)\cdot(M3-1) = (30-1)(46-1) = 1305.$$

According to the butt joint adopted model of the coated plates in the structure of the presented discrete model (see Figure 3), the plates borders introduced inserts (edges) which fragments are also described by network coordinates - only five fragments.

In accordance with the calculated scheme of plates discrete models, boundary kinematic communication conditions in the global coordinate system $OZ^{1'}Z^{2'}Z^{3'}$are:

– on the coordinate line $OZ^{3'}$ – the plane of symmetry is imposed by the joints on the movement $u_N^{2'}$ and angles of rotation $\nu_N^{2'}$;

– on the coordinate line $OZ^{2'}$ – The model of hinge tangential fastenings - is imposed on a link to move $u_N^{3'}$, $u_N^{2'}$;

– on the edges of the estimated fragments - if $Z_N^{2'} = 1500$ sm and $Z_N^{3'} = 3000$ sm the model of hinged tangential fixings is implemented – the connections are on $u_N^{3'}$, $u_N^{2'}$.

The numerical calculation results of the wheel load from an aircraft in B767-300 are shown on the isopoles and diagrams, which are presented in Fig. 5-13.

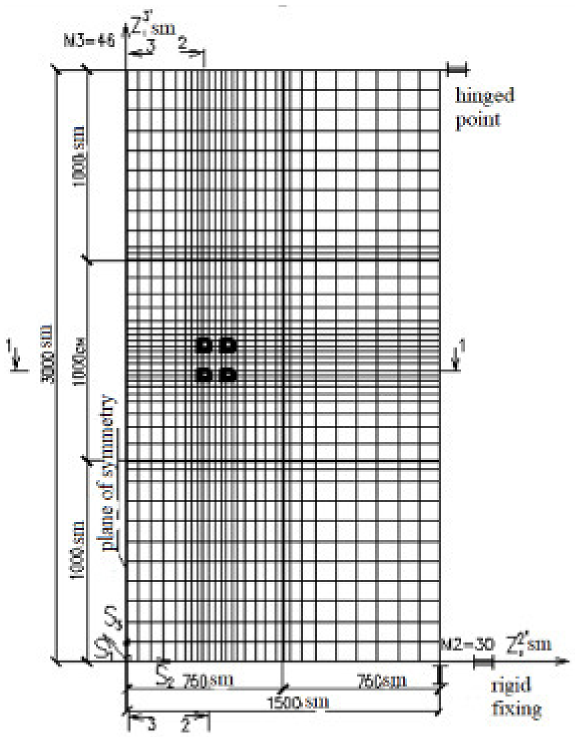

Figure 4 – Coverage scheme of the SZPS with the wheelbase chassis of the Aircraft B 767-300 chassis

Based on the results of calculations, data have been obtained to determine the maximum values of the main stresses in the conventional plate and stresses in the upper and lower layers of the real coating design.

$$\sigma_{1,\sup} = k_{1,\sup}\sigma_{\max}^{(e)}\ ;\ \sigma_{2,\inf} = k_{2,\inf}\sigma_{\max}^{(e)}\ .$$

The analysis of the obtained numerical results shows that the value of the lateral bending moment under the left lower wheel of the main support of the Aircraft B767-300 reaches the value $M_{552}^{33} = -99.724$ kHm/m, which corresponds to the stress in the lower fiber of the conditional plate $\sigma_{552}^{\max} = 950.884$ kHm/m^2.

To switch to a real 2-layer coating it should be used the formulas [4]:

$$\sigma_{1,\sup} = 1.4389\sigma_{\max}^{(e)}\ ;$$

$$\sigma_{2,\inf} = 0.4528\sigma_{\max}^{(e)}\ .$$

Considering the design resistance of the concrete strength for the upper and lower layers, respectively $R_{btn}^{\sup}$ and $R_{btn}^{\inf}$, it can be written:

$$\sigma_{1,\sup} = 1.4389\times 950.88 \le R_{btn}^{\sup}\cdot\gamma_c\cdot k_u\ ;$$

$$\sigma_{2,\inf} = 0.4528\times 950.88 \le R_{btn}^{\inf}\cdot\gamma_c\cdot k_u\ ,$$

where γ_c – is coefficient of work conditions, with [1] γ_c =0.75;

$k_u = 2 - 0.167\cdot l_g\cdot U_d$,

U_d – average annual application of wheel loads Aircrafts.

$\sigma_{1,\sup}$ = 1.395 MPa < 2.131 MPa;

$\sigma_{2,\inf}$ = 0.439 MPa < 0.711 MPa.

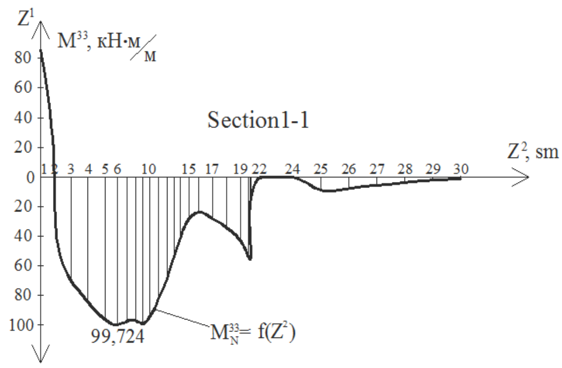

Figure 5 – Circle of bending moments M [33], kH*m/m from the action of the towers Aircraft B 767-300 in the section 1-1

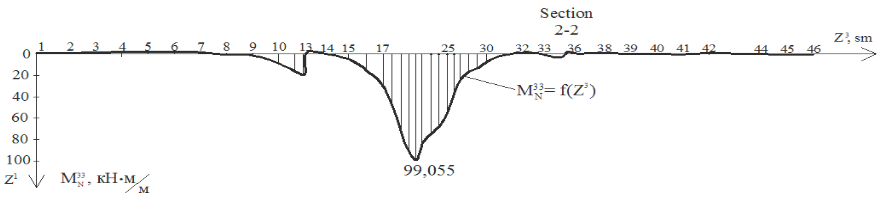

Figure 6 – Circle of bending moments M [33], kH*м/м from the action of the towers Aircraft B 767-300 in the section 2-2

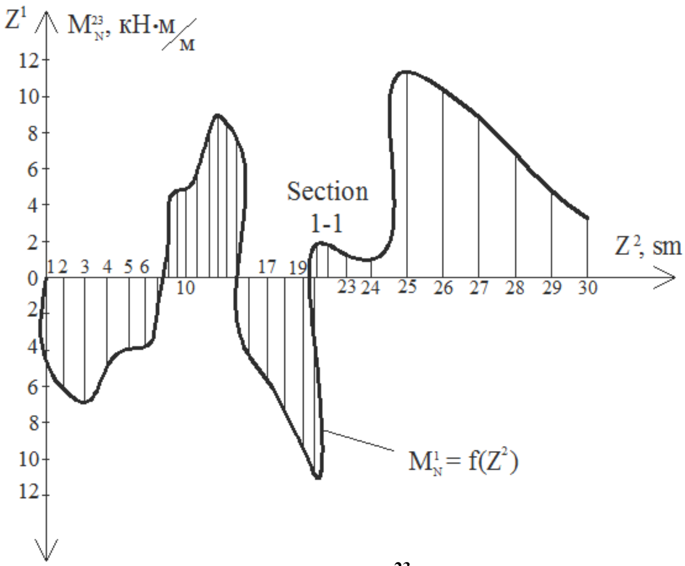

Figure 7 – Circuit running torque M [23], kH*м/м from the action of the towers Aircraft B 767-300 in the section 1-1

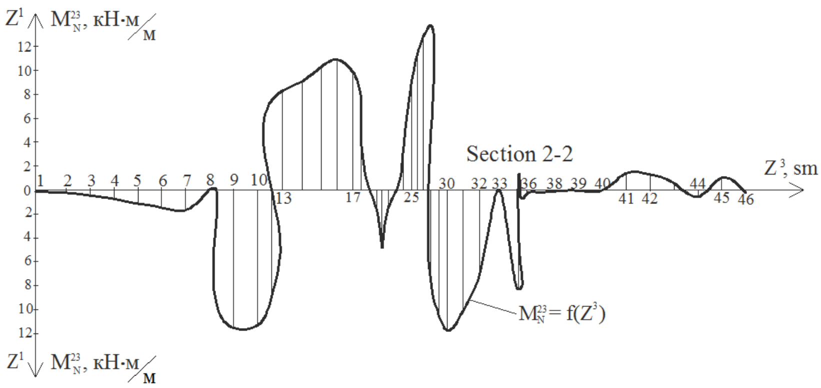

Figure 8 – Circuit running torque M [23], kH*м/м from the action of the towers Aircraft B 767-300 in the section 2-2

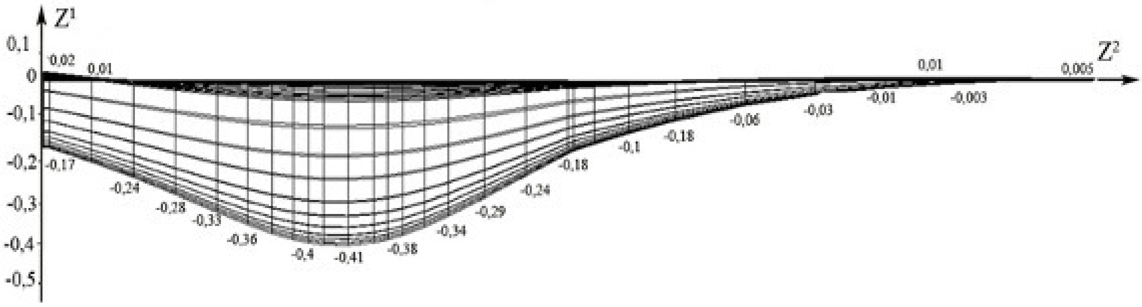

Figure 9 – The course of nodal displacements is a constant coefficient of subgrade resistance from the wheel loading of the bearings Aircraft B767-300 in the section 1-1

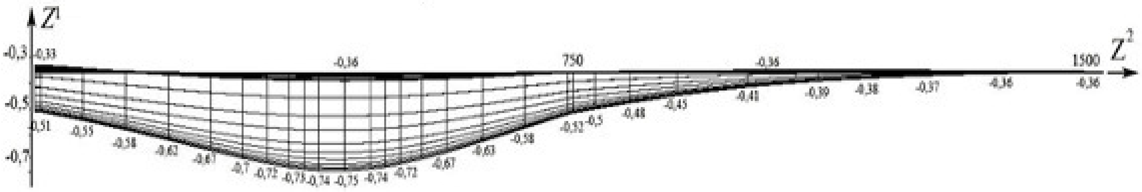

Figure 10 – Episode nodal displacement with variable coefficient of subgrade resistance from the wheel loading of the bearings Aircraft B767-300 in the section 1-1

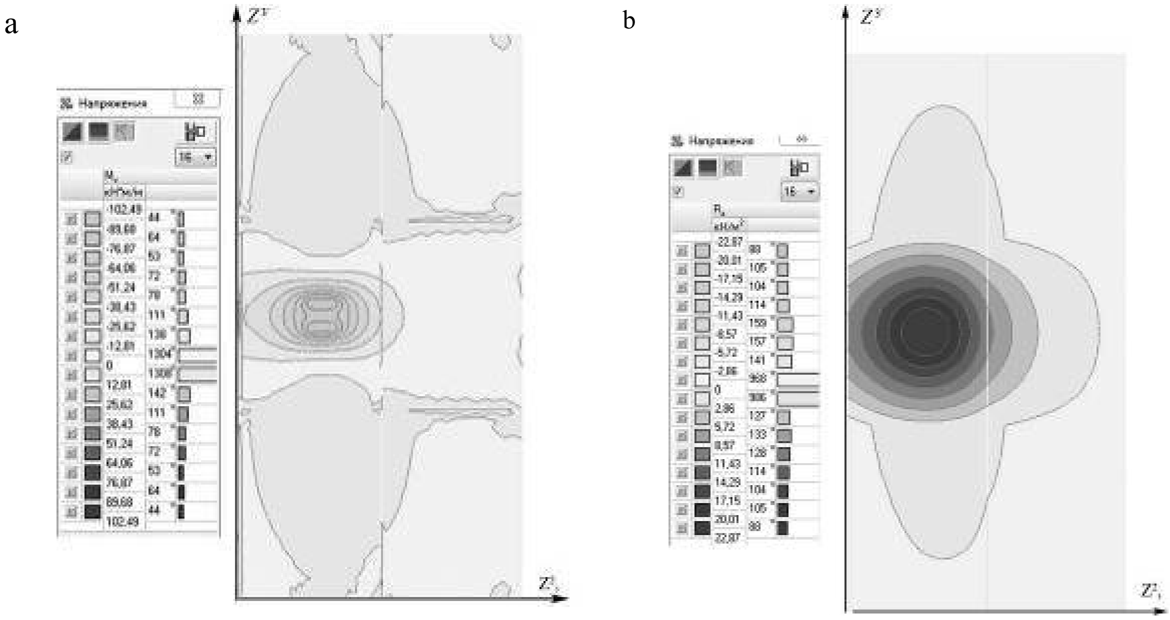

Figure 11 – Isolation of the calculation scheme of the coating from the action of the Aircraft B767-300 supports: *a* - bending moments M^{33}, kH*m/m; *б* - basic reactions R_{22}, kN/m

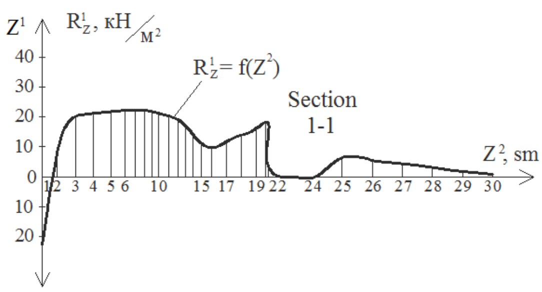

Figure 12 – Episode of nodal ground-level reactions in section 1-1

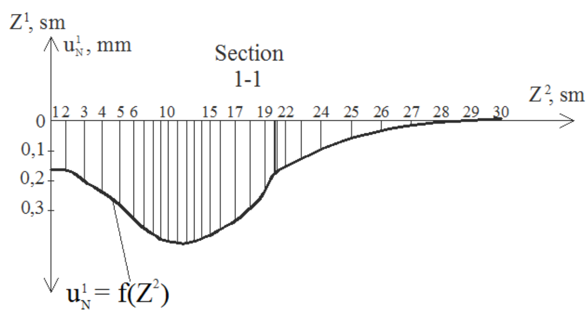

Figure 13 – The section of the nodal displacements from the wheel load of the Aircraft B767-300 in the section 1-1

Thus, the strength of the possible permissible stresses is ensured. The considered construction of the coating satisfies the conditions of the boundary state.

Calculation of coatings under the wheel influence of large-scale aircraft in the presence of weak soil layersinclusions should be performed only on the basis of numerical studies [4].

The results of numerical calculations of hard airdrome coverage are obtained in two variants:

1) with constant coefficient of subgrade resistance $C \equiv k_{se} = 55.8$ MH/m^3 ;

2) with variable coefficient of subgrade resistance within the calculated fragment by the formula (2) (in the presence of soil weak layer) with $C \equiv k_{se} = 55.8$ MH/m^3 and $C_{min} = 33$ MH/m^3.

Grades of the deflections have a smooth character, the maximum value of the deflection at constant coefficient of subgrade resistance is -0,41 mm fig. 9, and with a variable coefficient of subgrade resistance (c_0 = 33 MH/m^3, $c_{\max}$ = 56 MH/m^3) is -0,75 mm in the knot 201 FE-model fig. 10.

$$u_{201}^{1'} = -0.75 \text{ mm}.$$

Conclusions

A numerical calculation of rigid two-layer coating on rigid artificial basis, in interaction with inhomogeneous multilayer weak ground sub-space on the wheel influence of the chassis of the V767-300 subsystem, has been performed. According to the calculations of the real construction of the runway cover SIA "Odessa" the presence of soil weak layers on the basis of the coating, it can be concluded that the proposed methodology for numerical studies of hard coatings from the the wheel load action of super-heavy aircraft is sufficiently effective and reliable, meets the requirements for ensuring the reliable operation of modern airports airfields.

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Modeling using the LIRA 9.6 software package of contact interaction of the retaining wall with the base

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The article analyzes the research of scientists who studied the work of retaining walls under different soil conditions, and their contact interaction with the base. Examples of recent designs of retaining walls and their use are given. It is stated that the goal is to study the stress-strain state of a retaining wall with a structural surface, considering its contact interaction with the ground mass. The main stages and features of mathematical modeling of the retaining walls and soil foundation interaction in the conditions of a flat problem with the help of LIRA 9.6 software have been described. As a result, the nature of the retaining wall work has been analyzed under various conditions of interaction with the base and with different parameters of the retaining wall rigidity. The relationship between the retaining wall rigidity and the resulting equivalent stresses in the soil foundation under various conditions of interaction has been established. It has been concluded that on the contact surface of the retaining wall with a structural surface, the effect of stresses redistribution is achieved and the carrying capacity of the foundation soil is increased due to the formation of elastic cores and unloading arches.

Keywords: finite element method, retaining wall with a structural surface, modeling, stress-strain state.

Моделювання з допомогою програмного комплексу LIRA 9.6 контактної взаємодії підпірної стіни з основою

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В статті проаналізовано дослідження вчених, які вивчали роботу підпірних стін при різних ґрунтових умовах та їх контактної взаємодії з основою. Наведено приклади останніх розробок конструкцій підпірних стін та область їх використання. Мета таких досліджень полягає у вивченні напружено-деформаційного стану підпірної стіни зі структурною поверхнею з урахуванням її контактної взаємодії з ґрунтовим масивом. Описано основні етапи й особливості математичного моделювання взаємодії підпірної стіни й ґрунтової основи в умовах плоскої задачі з допомогою програмного комплексу LIRA 9.6. У результаті було проведено аналіз характеру роботи підпірної стіни при різних умовах взаємодії з основою та при різних параметрах жорсткості конструкції підпірної стіни. Виявлено залежність між жорсткістю конструкції підпірної стіни й еквівалентними напруженнями в ґрунтовій основі, які виникають при різноманітних умовах взаємодії. На підставі проведених досліджень встановлено, що на контактній поверхні підпірної стіни зі структурною поверхнею досягається ефект перерозподілу напружень та підвищується несуча здатність ґрунту основи за рахунок сумісної роботи конструкцій підпірної стіни й основи, що деформується.

Ключові слова: метод скінчених елементів, підпірна стіна зі структурною поверхнею, моделювання, напружено-деформаційних стан

Introduction

The pursuit of urban areas rational use leads to the development and search for new types of structures that are more economical and less labor-intensive. A number of requirements are imposed on the arrangement of retaining walls, most of which are based on a study of the geotechnical conditions of a territory that requires engineering protection. These structures in operating conditions are in a complex stress state, characterized by a large amount of capital costs, complexity and heterogeneity of interaction between the retaining wall design and the ground.

Special attention is paid to the methods of calculation and design of these structures in complex engineering and geological conditions. Modern methods of calculation enable to calculate the load on the wall and to simulate the stress-strain state of the soil with the help of software systems.

Methods and programs for calculating retaining walls were developed on the basis of recent advances in the field of structural mechanics, the theory of calculating reinforced concrete structures, soil mechanics, and the theory of elasticity, computational mathematics and practice. The calculation of retaining walls with design features, considering their joint work with the ground in areas with complex engineering and geological conditions, is the basis for further analytical and practical research.

Review of research sources and publications

Complex geotechnical conditions are geological environments that include specific soils, hazardous natural or man-made processes, geomorphological conditions, geological and hydrogeological factors of interaction with buildings and structures, belong to the II and III categories of engineering and geological conditions. Many scientists have devoted their work to the question of determining the lateral pressure of the soil on the retaining walls, considering their joint work with the soil mass.

So, V.F. Rauk [1, 2] investigated the nature and magnitude of the lateral pressure on the vertical face of the retaining wall, considering its deformations and displacement, using the model of a linearly deformable quarter-plane, but he did not consider the joint work of the vertical wall and foundation as a single system interacting with the ground. E.I. Chernysheva [3] investigated the question of the influence of the vertical wall flexibility on the value of the lateral pressure of the soil.

I.Ya .Luchkovsky [4], using the method of superposition, gives a solution to determine the lateral pressure of the soil on the retaining walls from narrow loads and concentrated force. He draws attention to the attenuation with the depth of the lateral pressure of the soil from the load. However, these authors in determining the lateral pressure did not consider the joint work of the vertical wall and the foundation as a single system that interacts with the ground.

I.A. Simvulidi [5] calculates a flexible retaining wall considering the interaction of all its elements with the ground, but introduces a linearly deformable half-plane as a model of the ground, both filling and base, which is not entirely correct. In addition, this method does not enable to consider the influence of the load on the stress-strain state (SSS) of the system.

A significant amount of research is devoted to the number of works on the dynamics of these structures, especially considering the elastoplastic properties of their materials, is significantly less [8, 9]. Usually, dynamic calculations are performed if the structure is subject to pulsed, vibratory or moving loads [10].

Definition of unsolved aspects of the problem

When building retaining walls in difficult geotechnical conditions, it is necessary to achieve: an increase in the stability and strength of the retaining walls; cost reduction of used building materials; reducing the volume of earthworks; reduction of non-uniformity of deformations; reduced construction time; improvement of the conditions for filling and compacting the backfill; increase of operational reliability, quality of performance of work and increase in service life of retaining walls. Thus, to assess the SSS retaining walls of the corner profile, it is necessary to consider the joint operation of the entire wall with the ground and the use of more reasonable soil models in the area of its vertical and horizontal elements.

Problem statement

The aim of the research is to study the stress-strain state of a retaining wall with a structural surface (RWSS), considering its contact interaction with the ground mass.

Basic material and results

The current regulatory documents recommend that when calculating retaining walls to perform calculations to determine the stability of the position of walls against shear, tipping, turning, to determine the local strength of the base and its bearing capacity, the strength of structural elements and joints should be ensured. It should be made calculations for base deformations. In difficult engineering and geological conditions, which are characterized by vertical and horizontal displacements of the base, which causes complex deformations in the construction of retaining walls. In such conditions it is impossible to implement the existing calculation methods [11, 12]. Earlier experiments on physical modeling [13–15] show areas of formation of elastic and plastic zones in the contact interaction of the RWSS and the deformable base.

It is known that even at low loads the soil goes into an elastoplastic state; therefore, this factor must be taken into account in the calculations. The theory of plastic flow with hardening is used, since deformation theories are not applicable under complex loading [16]. In this formulation, the problem considered here is studied for the first time. The calculation of retaining walls with a structural surface with regard to its joint work with the ground is not considered.

A promising area of research for the «retaining wall – ground» system is the use of mathematical modeling methods based on numerical analysis methods.

The most common today the finite element method (FEM) is, which forms the basis of modern software systems for calculating building structures, buildings and structures. FEM is most suitable for problems with a developed non-uniformity of strength characteristics. Compared with the classical variation methods, the FEM is more flexible in setting geometric parameters and boundary conditions, visual and universal for a wide range of tasks. At the same time, it is possible to choose different soil models for solving the set task. Regulatory documents currently in force in Ukraine [17–19] recommend performing calculations using software systems in which the FEM is implemented.

Digital and finite element models of the usual retaining wall and RWSS are presented in fig. 1 and 2. The retaining wall rests directly on the soil, represented by loam, and loam is also taken as the filling soil.

The calculation of the stability and strength of the retaining wall was performed by the FEM method using the LIRA 9.6 software package. Characteristics of the soil are given in table.1.

Table 1 – Characteristics of stiffness

Stiffness type	Name	Stiffness parameters
CE 284	Foundation soil, backfill soil	$E = 1800$ t/m^2 $\nu = 0{,}35$ $H = 100$ cm $C = 19{,}51$ kPa $R_t = 0{,}15$ t / m^2

a) b)

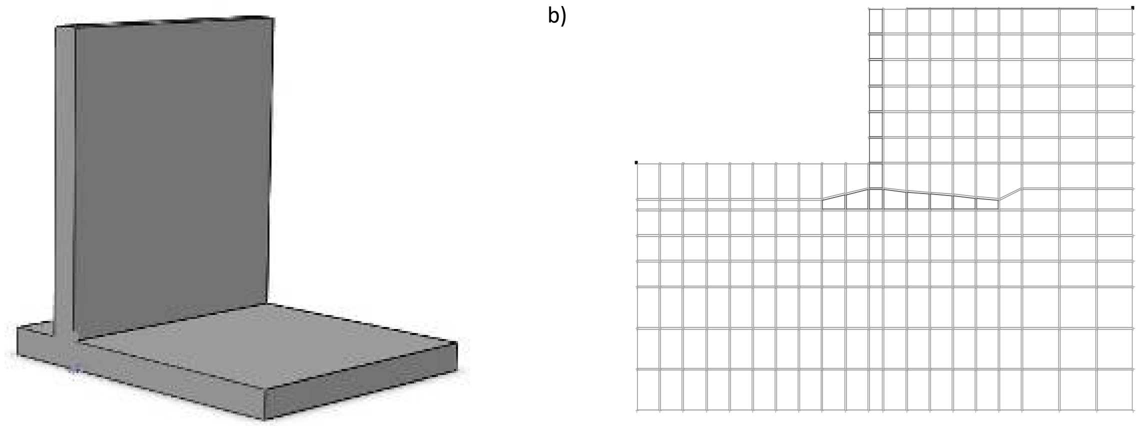


Figure 1 – Models of the usual retaining wall:
a – digital model; b – finite element model

a) b)

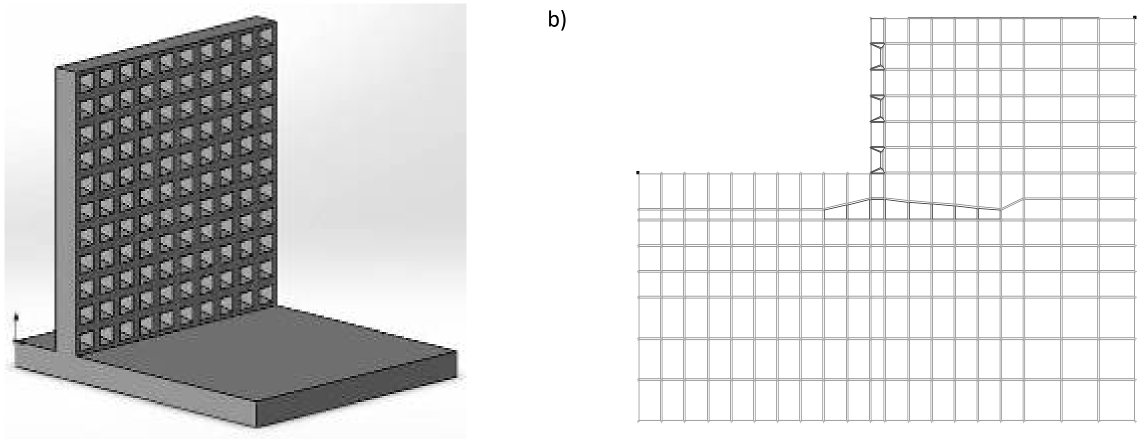
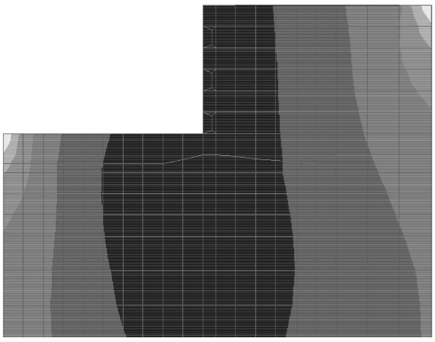
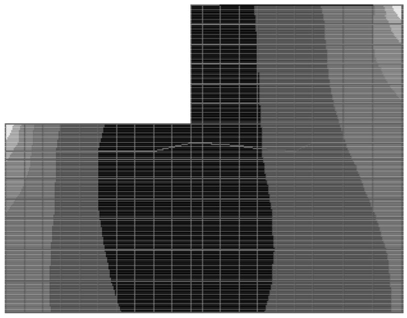
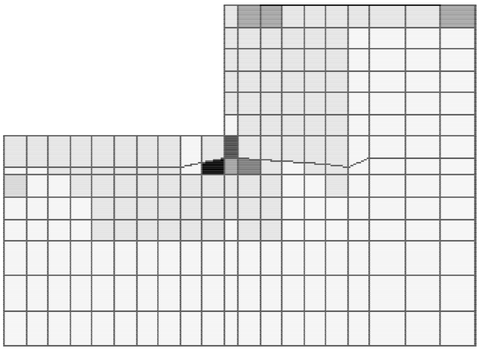
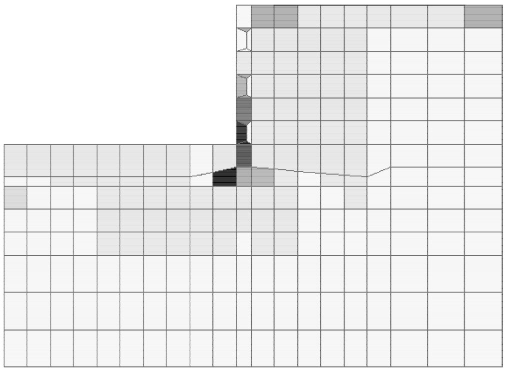
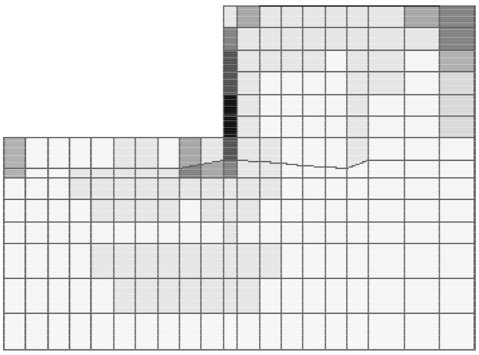
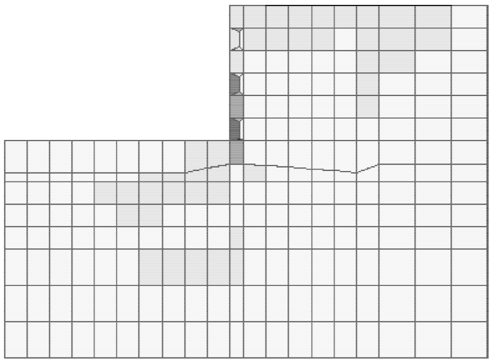
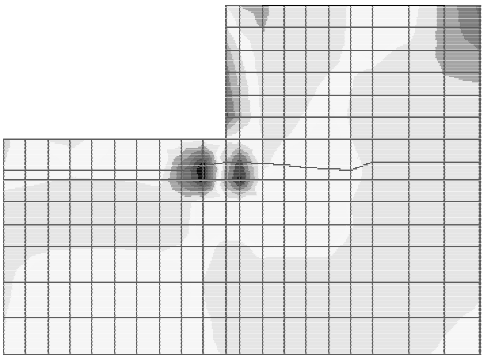
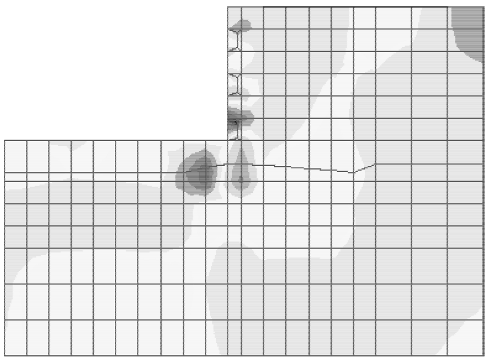


Figure 2 – RWSS models:
a – digital model; b – finite element model

Table 2 – Comparison of retaining walls

<table>
<tr><th>Plain corner retaining wall</th><th>Retaining wall with structural surface</th></tr>
<tr><td>1</td><td>2</td></tr>
<tr><td colspan="2">Isopole displacement, mm</td></tr>
<tr><td></td><td></td></tr>
<tr><td colspan="2">Voltage mosaic Nx, kPa</td></tr>
<tr><td></td><td></td></tr>
<tr><td colspan="2">Voltage mosaic Nz, kPa</td></tr>
<tr><td></td><td></td></tr>
<tr><td colspan="2">Isopole voltage Tzx, kPa</td></tr>
<tr><td></td><td></td></tr>
</table>

RWSS has vertical and horizontal elements on the surface, which are located on the contact side, supporting parts and voids in the form of truncated pyramids of the same size and directed by a smaller base into the vertical and base elements [20].

The monolithic angular retaining wall consists of a base plate, which has voids and supporting parts, which are located on the sole and on the back side of the vertical element. The soil is directed into voids, which have the shape of truncated pyramids. Under the sole of the base plate and on the back of the vertical element are two sheets of elastically flexible material. With the development of a deforming load in time, that is, with the vertical and horizontal displacements of the ground in relation to the monolithic wall of the corner type, after its installation, the soil gradually penetrates into the voids. Premature filling of voids is prevented by sheets of elastically compliant material.

The modeling of the operation of the RWSS and non-uniformly deformable base was carried out in the Lyra 9.6 software package under the conditions of a flat problem. It enabled to perform numerical modeling, calculation and design of such structures, considering the real soil conditions.

The modeling process was carried out in three stages: the creation of the calculated finite element model of the structure; creation of a calculated finite element model of a soil massif; contact surface modeling “buried construction - soil massif”.

To create a computational finite element model that considers the joint operation of structures with a ground massif, finite elements were used: КЕ 284 – a physically nonlinear universal rectangular for a planar problem (soil); CE 10 – universal spatial rod; CE 21 – rectangular CE for a planar problem (beam-wall); CE 22 – triangular EC for a plane problem (beam-wall); CE 30 is a quadrilateral EC for a plane problem (beam-wall) [21].

The finite element model (QE model) has deformation boundary conditions, namely, the movement of the upper nodes of the model extreme points, is limited. The model height is taken from considerations of the wall joint work and the surrounding massif, which is composed of loams.

CE-models are provided with deformation characteristics - elastic modulus E and Poisson's ratio ν, R0 is the specific weight of the material. After specifying the CE-model stiffness, the load values in the load cases should be specified.

Analyzing the horizontal movements of retaining walls, it can be concluded that there is an insignificant difference in the values for both structures, with the same soil conditions and loading.

At the same time, the obtained values of Nx = 0.78 kPa, Nz = 1.57 kPa, Txz = 4.70 kPa show a decrease in contact stresses, whereas for a typical retaining wall they were Nx = 1.96 kPa, Nz = 4.9 kPa, Txz = 11.76 kPa.

SSS of finite element models of retaining walls are presented in Table 2.

In this study, the LIRA 9.6 finite element software package was used to simulate the retaining wall, which is a finite element software method and has been at a continuous improvement stage for many years.

The backfill of the ground and ground of the base was modeled using the Coulomb – Mohr model, which is an elastic-plastic model capable of considering the expansion of the ground. Considering the fact that the retaining wall is much more rigid than the foundation and backfill primer, all elements of the retaining wall were adopted as an elastic material.

Conclusions

The calculation results have confirmed the experimental studies carried out earlier. Mathematical modeling enabled to demonstrate the voltage reduction on the contact surface. Phased filling of voids leads to a uniform distribution of deformations, which in the long term increases the structure life, thereby providing an economic effect. The peculiarity of the design of the RWSS, when the soil interacts with the structural surface, increases the base bearing capacity due to the construction joint work of the retaining wall and the deformable base.

As a result of this work, scientific results have been obtained:

– the features of the base and structure SSS in the flat base conditions – retaining wall system have been identified and analyzed using the model in the LIRA 9.6 software package;

– the influence of the structural surface use of the RWSS on the soil foundation SSS during their contact interaction has been investigated;

– it has been recorded that the main stresses in a number of sections in the RWSS models are less than the stresses in the ordinary wall models by 15-18%.

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Choice substantiation of a folded foundation model via laboratory experiment

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The examples of recent developments of foundation thin-walled structures and their applications are presented. It is noted that the purpose is to develop a method for conducting experimental studies of contact interaction between a folded shell foundation and a deformed base. The purpose of the laboratory tray research is to determine the optimal characteristics of the suggested foundation design. The article suggested a method for conducting a laboratory tray experiment to compare the operation of two models of folded shell foundations. The main stages of conducting the scientific experiment have been described and the basic principles of selecting materials, devices and equipment for carrying out the experiment have been considered. The conduct of the experiment including data recording for each foundation model has been described. As a result, subsidence data for each foundation model have been obtained. The findings have been analyzed; subsidence graphs for each foundation design have been built. It is concluded that better distribution of external load to the foundation system elements is achieved due to the application of a hinged joint of prismatic folds with supporting beams. It has been found that the chosen parameters of the second foundation model complied with the optimized results of the experiment planning and enabled to demonstrate the properties of the foundation system load redistribution better.

Keywords: folded shell foundation, method of conducting an experiment, soil

Обґрунтування вибору моделі складчастого фундаменту із застосуванням методики проведення лабораторного експерименту

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В статті проаналізовано дослідження вітчизняних та закордонних вчених, що займалися вивченням роботи фундаментів-оболонок при різних ґрунтових умовах, та їх контактної взаємодії з основою. Наведено приклади останніх розробок фундаментних тонкостінних конструкцій та сфери їх використання. Зазначено, що метою є розробка методики проведення експериментальних досліджень контактної взаємодії складчастого фундаменту-оболонки і деформованої основи. Метою лабораторних лоткових досліджень було визначення оптимальних характеристик запропонованої конструкції фундаменту. В статті було запропоновано методику проведення лабораторного лоткового експерименту для порівняння роботи двох моделей складчастих фундаментів-оболонок. Було описано основні етапи проведення наукового експерименту і розглянуто основні принципи при виборі матеріалів для виконання експерименту, приладів та устаткування. Було описано безпосередньо проведення експерименту із фіксацією даних по кожній із моделей фундаментів. В результаті було отримано дані осідань по кожній із моделей фундаментів, проаналізовано результати та побудовано графіки осідань для кожної фундаментної конструкції. Зроблено висновок, що завдяки застосуванню шарнірного з’єднання призматичних складок з опорними балками досягається більш рівномірне розподілення зовнішнього навантаження на елементи фундаментної системи. Встановлено, що обрані параметри другої моделі фундаменту у відповідності до результатів оптимізації планування експерименту дозволили більш повноцінно проявити властивості перерозподілу навантаження фундаментної системи.

Ключові слова: складчастий фундамент-оболонка, методика проведення експерименту, ґрунт

Introduction

Shallow foundations such as shell foundations are studied and used to improve capacity of a foundation structure under conditions of weak and water-soaked soils. The practice of designing and constructing buildings and structures using plate-shell foundations, pier shell foundations, folded shell foundations, and others, is rather common.

One of the main stages of research is to conduct an experiment in laboratory or on-site conditions. The results of such studies enable to investigate nature and character of a foundation structure operation. The development of a full-fledged methodology for conducting an experiment is the basis for further analytical and practical research.

Review of research sources and publications

Applications of shell foundations, diverse in form, operating conditions and materials, are found in many countries of the world. Modern investigations in the field of foundation engineering, in particular application of shell foundations, have common features aimed at constructing such foundations in different soil conditions and expanding the scope of their application. The development of fundamentally new constructive solutions is one of the main goals of research in this field.

Such domestic and foreign scholars of the industry as Pronozin Y.O., Vannyshkin SG, Tetior O.N., Mahmoud Samir El-kadya, Essam Farouk Badrawi, Kurian N.P. have studied the operation of shell foundations of various types when interacting with a soil in detail [1-9].

Poroshin O.S. in his study [2] solved the issue of introducing in building practice the flat thin-walled cylindrical binary shell foundations inverted to soil, providing tensile strength within a slab foundation. The purpose of such foundations introduction was to reduce the time and cost of construction, improve the reliability of structures erected on weak soils, solve a number of geotechnical problems which have complex engineering and geological conditions. As a result, a new foundation design was developed and its interaction with a base was studied.

Govorov D.V., under the supervision of Goncharov Yu.M., developed and investigated the operation of shell foundations in the form of spatial folded slabs applicable for permafrost soils [6]. The suggested foundation model was used for constructing a four-story panel administrative building in industrial area "Oganer" and a warm long-stay car park.

Mahmoud Samir El-kady and Essam Farouk Badrawi conducted experimental and numerical studies using five square foundations; one of them was a flat foundation as a reference sample and four foundations of a composite form of shell type [7]. The method for conducting laboratory experiments for studying shell foundations opration in order to determine their bearing capacity and stress-strain state of a base was suggested.

Definition of unsolved aspects of the problem

The complexity of a laboratory experiment is due to the lack of information on the correct algorithm of conducting such experiments for a folded foundation. Calculation and design of shell foundations require careful study of behavior and nature of the foundation model operation in laboratory conditions.

Problem statement

The aim of the research is to develop a method for conducting experimental studies of contact interaction between a folded shell foundation and a deformed base.

Basic material and results

One of the main engineering tasks is developing optimal constructive solutions considering specific operational conditions. Laboratory experiments are carried out using typical devices, special simulators, stands, equipment, etc. These experiments enable to conduct thorough and repetitive studying of some characteristics impact under alteration of others, and also provide relevant scientific information at minimum costs if the experiment is well-grounded scientifically.

Two types of foundations are considered: the first is a folded foundation in the form of separate thin-walled reinforced concrete folds that are joined together by a steel or reinforced concrete beam [3]; and the second - a folded foundation with an improved system of support beams and joints [10].

The basis for the experimental research is a laboratory tray experiment aimed at determining optimal characteristics of the suggested foundation design and identifying the qualitative patterns of interaction between a folded foundation and a base.

The set goal was achieved in two stages. At the first stage, the bearing capacity of foundation models depending on structural features of the foundations, their geometric parameters, and characteristics of the soil were investigated. At the second stage, it was necessary to compare contact interaction between the first and the second models of foundations.

The tray study was performed in a metal tray sized 600×650×680 mm. The metal frame was made of 80×80 mm angles and 50 mm wide steel stripes. The front edge was made of plexiglass with horizontal lines applied at 15 mm intervals. All other edges were made of 16 mm chipboard. The dimensions of the tray and the foundation models were specified in such a way as to exclude the compression by the tray walls (Fig. 1).

Sandy soil was taken as a base. The amount of fine sand necessary for the experiment was dried to a dry state. After that, the sand was sifted through a sieve with 0.5 mm axes diameter. The sand base model was arranged in 15 mm thick layers and tamped. This process was controlled with control horizontal lines on the front of plexiglas. Physical and mechanical characteristics of the soil were determined according to a conventional method [11] in laboratory conditions using a compression device of Litvinov system (Fig. 2). The soil surface in the tray was thoroughly planned.

Figure 1 – Tray in a metal frame to conduct the experiment

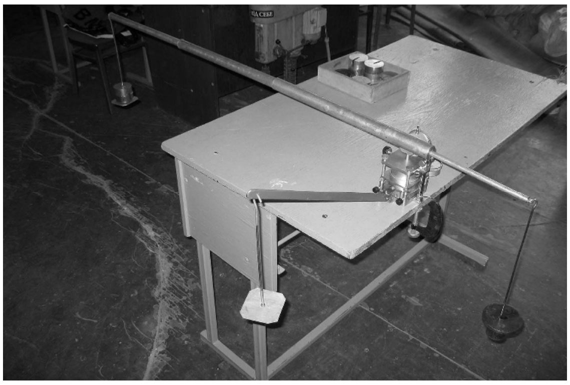


Figure 2 – Determination of physical-mechanical characteristics of soil using PLL-9 device

a) b)

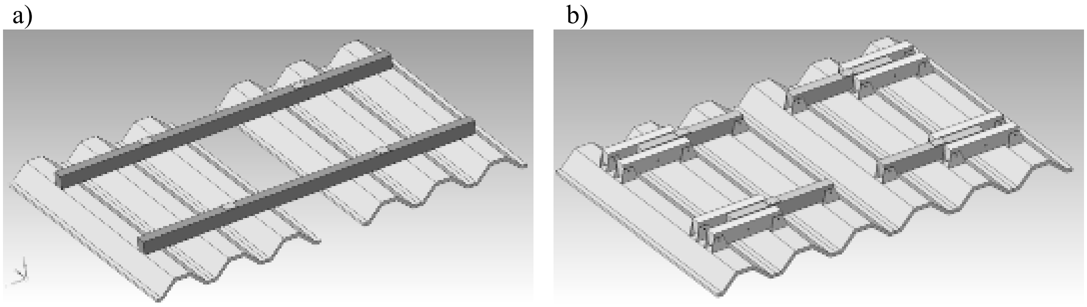


Figure 3 – 3D modeling of the folded foundation structures:
a) the first model; b) the second model

a) b)

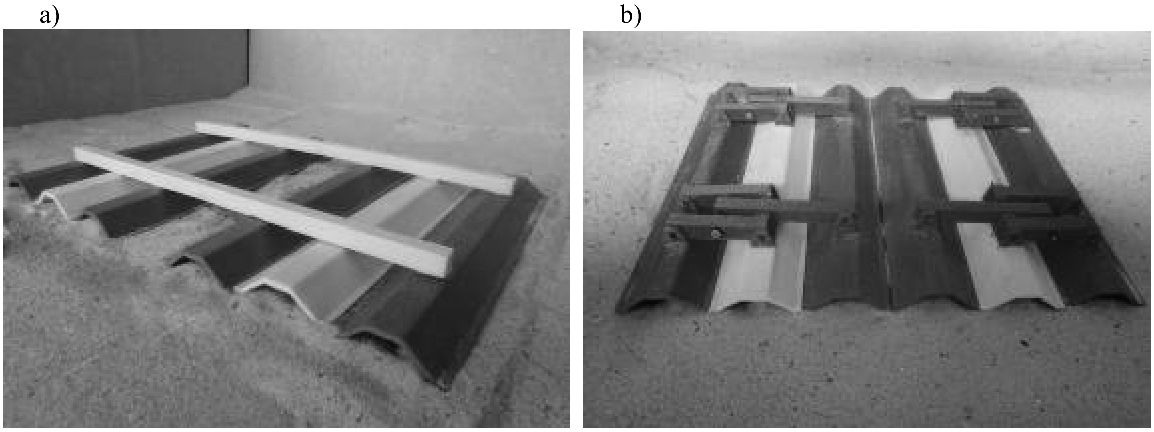


Figure 4 – Models of the folded foundation structures in assembled state:
a) the first model; b) the second model

a) b)

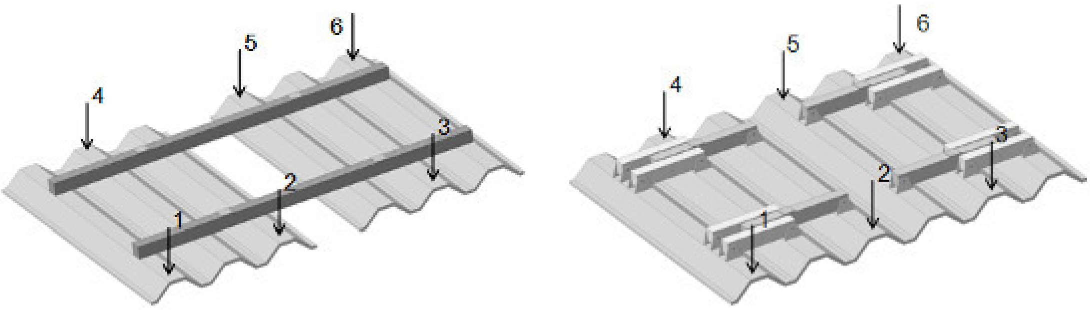


Figure 5 – Location of time-type indicators on elements of foundation models
a) the first model; b) the second model

Two series of tray experiments were conducted. The task of the first series was to study the impact of structural features of foundation models and ground characteristics of the base on the bearing capacity of the suggested foundation models.

The purpose of the second series of experiments was to compare subsidence and bearing capacity of the foundation models.

The foundation models were created using the technology of 3D modeling and 3D printing. The foundation elements were printed on a 3D printer XYZprinting da Vinci 1.0 Pro from plastics with typical characteristics (thread diameter – 1,75 mm; oval shape – +/- 0,03; density – 1250 kg/m^3; melting viscosity – 2000Pa·s; melting point – 230 C°; bending resistance – 20 times; printing technology – FDM). Computer modeling of foundations was carried out via Kompas 3D V16 software application (Figure 3).

All elements of the foundation were glued with the special glue; the parts which required preservation of hinges were joined with metal rods of the required diameter.

The first set of parts was made to simulate the first type of foundation. Here the supporting beam is to be rigidly joined with all six folds of the foundation. The second set was made to simulate the second type of foundation (Figure 4).

The quantitative values of foundations subsidence of were recorded using IT-10 time type indicators. Location of the indicators on the shelves of foundation folds is shown in figure 5. They were installed on a specially made metal frame which was rigidly secured with the experimental tray (Figure 6). The indicators fixed folds vertical movements. The data on the indicators were entered in a journal and based on the average indicators; the graphs of subsidence were drawn.

A special device was created for simulating a real load transfer of a power transmission line to the foundation. It consists of a metal plate which joins four metal rods through which the transfer of external load to the supporting beams and, accordingly, to the folds of the foundation (Figure 6) takes place.

The loads were applied in levels of 1 kPa, each pressure level was kept until conditional stabilization of soil deformation. The speed of foundation models subsidence (not more than 0.1 mm per 30 minutes) was taken as a criterion of deformation conditional stabilization [12]. Each subsequent level of pressure also lasted during the time of conditional stabilization.

As a result of a series of tray experiments, subsidence graphs for two foundation models were drawn. In order to facilitate further analysis, the subsidence graphs were separately drawn for the three front and three rear indicators (Figure 7, 8).

Additionally, the trend lines (linear type) were constructed on the subsidence graphs as a geometric representation of the mean values of the analyzed parameters obtained via any mathematical function. This is a graph of the approximating function. The R2 value shows the accuracy of the approximation. When this value approaches a unit, the approximation error is equal to zero [13].

Thus, it is possible to predict future behavior of the subsidence graphs. This type of the trend line (linear) was chosen as the alteration of the indicators that occurs almost linearly (increases at constant speed). According to the approximation values (within 0.95-0.99), chosen linear approximation provided high accuracy and a well-predicted result.

a) b)

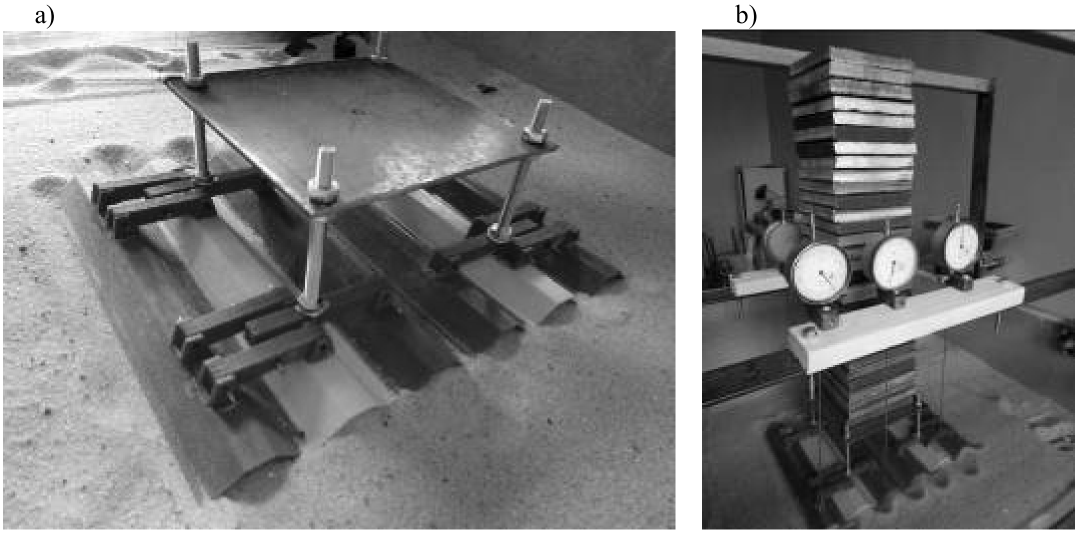


Figure 6 – Preparation of the laboratory equipment and conducting the experiment:
a – metal device for transferring external load to foundation structure;
b – loading the foundation structure and fixing the indicators
of subsidence using IT-10 time type indicators

Analyzing the subsidence graphs of the first foundation model (Figure 7), it is possible to see almost similar values of subsidence on indicators 2 and 3 (4 and 5 are similar) caused by external load. While the values of the indicators 3 and 6, following the same linear law, have considerably less values of subsidence. This indicates the uneven distribution of load in this system. The type of load transfer through a solid support beam does not ensure full and even operation of the entire soil under each fold at a time.

At the same time, in the graphs of subsidence on the second foundation model (Figure 8) it is evident that the curves of subsidence of the front indicators 1-3 are of the same nature and the value of the final subsidence is almost identical. The rear indicators 4-6 behave similarly. It shows that this foundation model is performed due to the peculiarities of co-operation of the structural elements of the folded foundation. More even distribution of external load to the elements of the foundation system is achieved through the use of the hinged joint prismatic folds with support beams.

a)

b

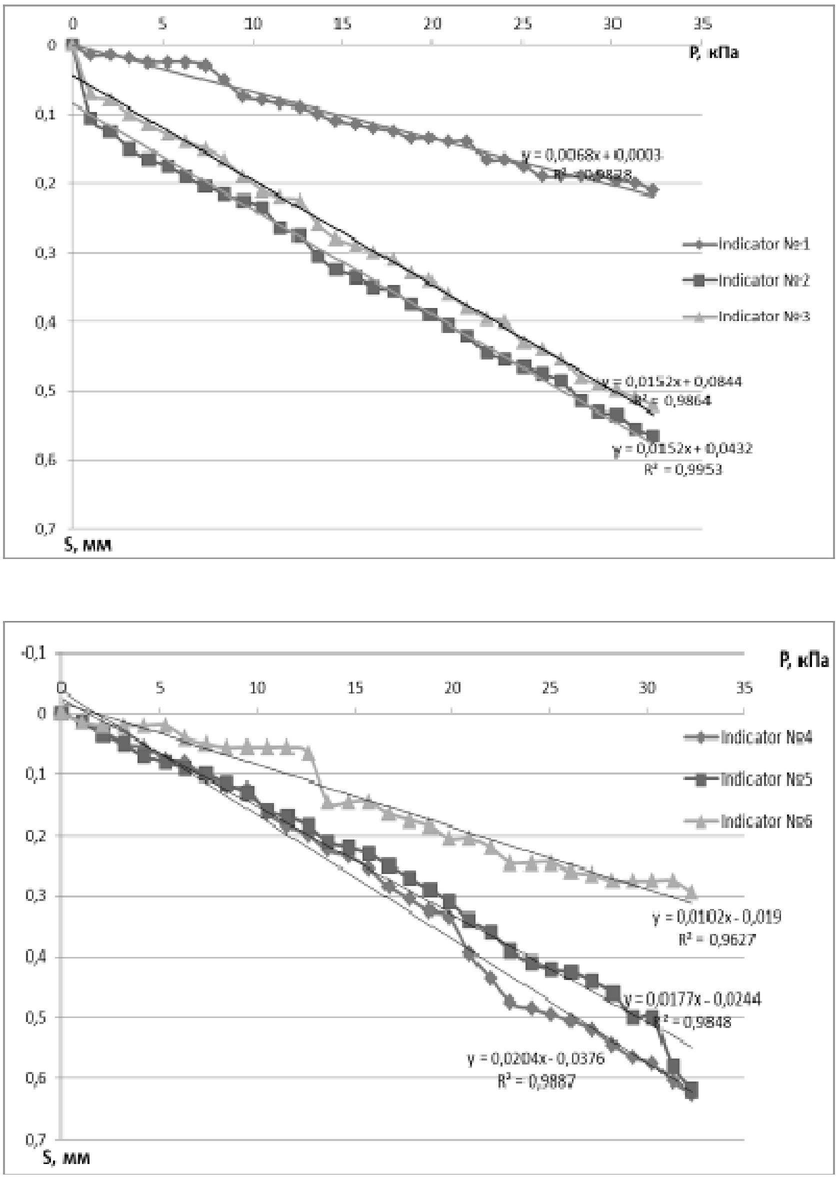
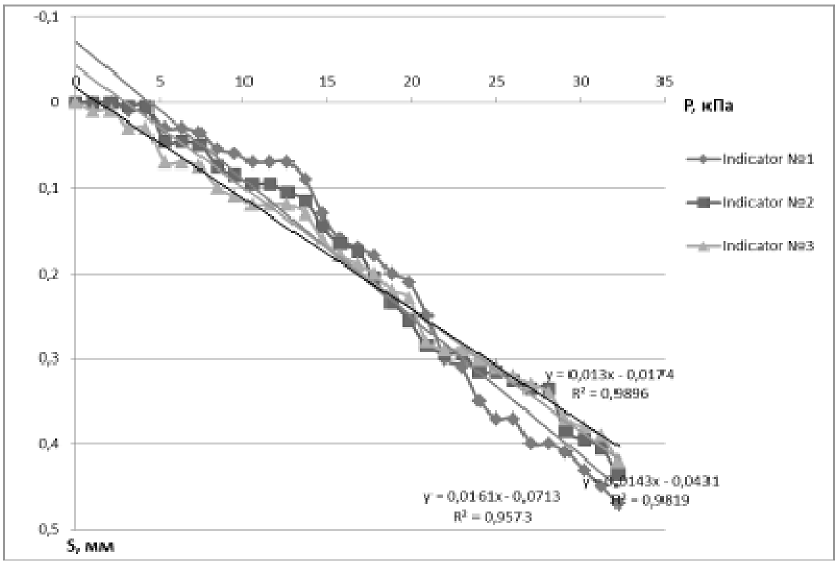


Figure 7 - Subsidence graphs based on indicators values of the first foundation model
a – for indicators № 1-3; b – for indicators № 4-6

a)



b

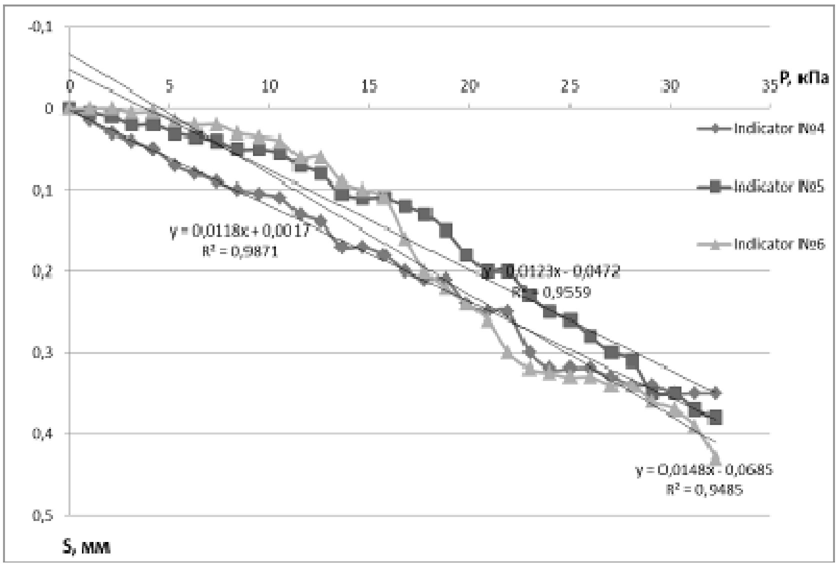


Figure 8 – Subsidence graphs based on indicators' values of the second foundation model

a – for indicators № 1-3; b – for indicators № 4-6

Conclusion

The methodology was developed and the laboratory experiment was conducted to determine subsidence of the foundation structures; two models of folded shell foundations were compared. The advantage of the second foundation model in terms of the conditions of interaction with a base was shown. As a result, it was found that the chosen parameters of the second foundation model complied with the optimized results of the experiment planning and enabled to demonstrate the foundation system load redistribution properties better.

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Research of the industrial facility underground structures settlement caused by its machinery dynamic loads

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The article is devoted to the issue of research of the soil basement vibrocreep negative impact, which takes place under dynamic loads. The results of on-site measurements of industrial facility underground structures forced dynamic vibrations parameters have been presented. The influence of the dynamic vibrations value on the development of underground cable channel precast concrete units uneven settlement has been analyzed. The interrelation between the value of the channel built-up construction settlement at the point of measurement and the amplitude of forced vibrations at this point has been determined.

Keywords: vibrocreep, soil base, foundation, uneven settlement, dynamic load, vibration amplitude.

Дослідження осідань підземних конструкцій промислового цеху від дії динамічних навантажень його обладнання

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Стаття присвячена питанню досліджень негативного явища віброповзучості ґрунтової основи, що має місце при динамічних навантаженнях. Дослідження проводилися на заводі з виробництва металопрокату з приводу триваючих нерівномірних осідань повнопрохідного двоповерхового кабельного тунелю розташованого в підземній частині виробничого цеху. Як було встановлено даний кабельний тунель піддається динамічному впливу при роботі двох пілігримових станів, на яких виробляються безшовні гарячекатані труби великого діаметра. Серйозність питання осідання кабельного тунеля для виробництва полягає в можливому ушкодженні прокладених у ньому комунікацій внаслідок нерівномірного й нестабілізованого осідання його збірних залізобетонних конструкцій. Установлене, що із двох розташованих у зоні проходження кабельного тунелю пілігримових станів один з них, позначений у статті під №2, спричиняє суттєвіший вплив на величину амплітуди коливань у всіх його точках у яких проводилися виміри. У статті викладені результати натурних вимірів параметрів вимушених динамічних коливань підземних конструкцій кабельного тунелю промислового цеху, які були зроблені на відмітці -5,200 від рівня чистої підлоги. Серед іншого було встановлено, що максимальна амплітуда коливань кабельного каналу перебуває в точці №4 найбільш наближеної до пілігримового стану №2. У районі зазначеної точки також перебуває й зона з максимальним осіданням конструкцій тунелю. Проаналізовано вплив величини динамічних коливань на розвиток нерівномірного осідання збірних залізобетонних секцій підземного кабельного тунелю. Встановлено взаємозв'язок величини осідання збірної конструкції каналу в точці виміру від величини амплітуди вимушених коливань у даній точці.

Ключові слова: віброповзучість, ґрунтова основа, фундамент, нерівномірне осідання, динамічне навантаження, амплітуда коливань.

Introduction

The increase in the productivity of metallurgical machines (new machines and the ones which are in operation for a long time), is caused by a significant intensification of their operation modes. At the same time, both static and dynamic loads within machines increase, which are fully or partially transferred to the foundations and "absorbed" by the base on which this machinery is installed. Due to acting loads, metallurgical machines are unparalleled among other heavy machines.

One of the main methods of producing seamless pipes of large and medium diameters with wall thickness from 7 to 100 mm is rolling on pilger mills. A wide range of pipes for almost any purpose is produced on pilger mills: oil pipes, casing pipes, boiler pipes made of special steels and alloys, bimetallic pipes. Seamless pipes of large diameter (up to 630 mm) with different wall thickness without using mill expanders can be made only on pilger mills.

The pilgrim method is one of the most economical and versatile methods for producing seamless pipes, since switching to a different size of pipes on pilger mills takes much less time than, for example, on a continuous multistand mill. Therefore, despite the demand for new rolling technologies, the production of pipes on pilger assemblies continues to be one of the most common in the world for producing hot-rolled pipes of a wide range of sizes and steel grades [1].

Technological loads of pilger mills are characterized by short duration (less than 0.8 s.) and a pronounced peak character, as well as significant amplitudes of vibrations transmitted to the base.

The values of the mechanical properties of soils of different composition, structure and state in general decrease depending on the mode of dynamic loads on them. Vibration, shocks and other oscillating movements typical for urban conditions, adversely affect the geotechnical properties of foundations of buildings and structures. Incoherent (sandy) soils are the most sensitive to such impacts, especially in the water-saturated state [2-4].

Review of research sources and publications

Geotechnics [5–7], by analogy with statics [8], distinguish three phases of base deformation depending on the intensity of the combined static and dynamic loads on it. The first phase takes place under small static and dynamic loads, the settlement of the base within it occurs due to a decrease in the soil porosityl. The second phase is characterized by the development of significant areas of plastic deformations in the massif (even under small dynamic loads, significant precipitation occurs with a tendency to their growth and lengthening of the stabilization time [2–4]). These settlements occur in sandy soils (including dense ones) and clayey soils. In the third phase, the settlements are destructive and have high speed – loss of stability or immersion of the foundation in the ground as in a viscous fluid [9 - 14].

Definition of unsolved aspects of the problem

When designing foundations for machinery with dynamic loads, the influence of vibration transmitted through the base of the foundation on the ground should be considered. It reduces their strength, increases compressibility and, as a result, causes cracks in structures when their strength limit is exceeded due to a combination of static and dynamic loads. In addition to the frequency and amplitude of forced vibrations of the machinery, the type and size of the foundations, as well as the natural vibration frequencies of structures and their elements affect the level of vibration.

Problem statement

It should be noted that if the structures of foundations for heavy machines (in this case, pilger mills) are often correctly designed for the perception and distribution of significant dynamic loads, then with other underground structures located in the zone of propagation of vibrations, the phenomena known as vibrocreep can occur. The purpose of this paper is to identify the influence of amplitudes and frequencies of the dynamic load on the value of the underground structure settlement, which is in the range of vibrations transmitted by the foundations of heavy metallurgical machines.

Basic material and results

Research of the parameters of dynamic vibrations in the active industrial facility of the metallurgical plant has been carried out in cable channels under the facility (mark -5,200) under pilger mills No.1 and No.2 in connection with complaints of cable channels geometry violation in the zone of the dynamic loads on the foundation grounds during operation of these pilger mills. The scheme of the vibration parameters measurement points is shown in Fig. 1.

To determine the vibration parameters, a set of equipment has been used, which consists of a vibrometer VIP-2 that includes a displacement sensor D21A and a digital oscilloscope Oscill connected to the vibrometer for spectral and visual analysis of the vibration nature and recording of the oscillogram of vibrations. The vibration parameters along the vertical axis, perpendicular to the earth surface have been recorded as the most significant component in the case of the base vibrocreep.

Next, the measured parameters of vibrations at the points indicated in Fig. 1 have been considered.

Point No.1

At point No.1 with the operating pilger mill No.1, the following vibration parameters have been recorded: the maximum vibration amplitude has been recorded at 30 μm at a frequency of a pulsed load of 1.2 Hz (see Fig. 2).

With inoperative pilger mills No.1 and 2, at point No.1, "background" vibration is observed at a level of ≈ 10 μm at a frequency of 15 Hz (between the pulse peaks in Fig. 2 and 3).

At point No.1 with the running piliger mill No.2, the following vibration parameters have been recorded: the maximum vibration amplitude has been recorded at 15 µm at a frequency of a pulsed load of 1.2 Hz (see Fig. 3).

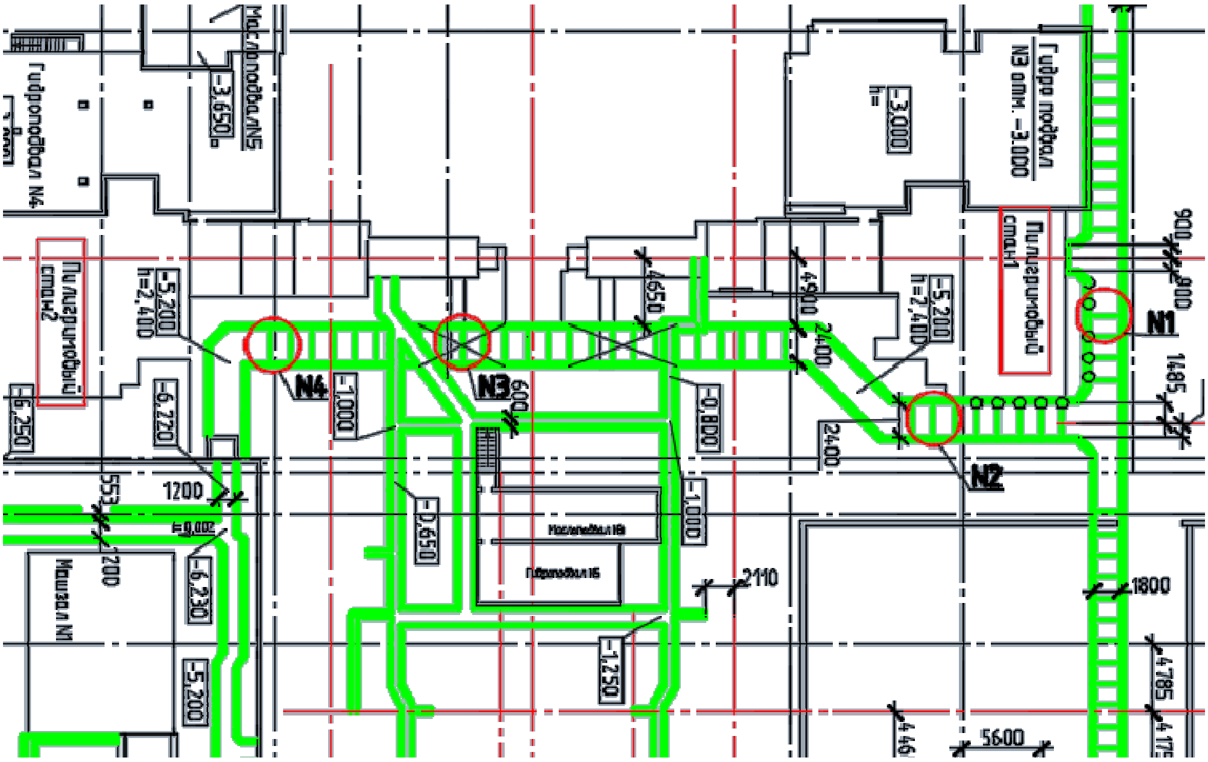

Figure 1 – The scheme of the points of measurement of vibration parameters on a fragment of the plan of cable channels

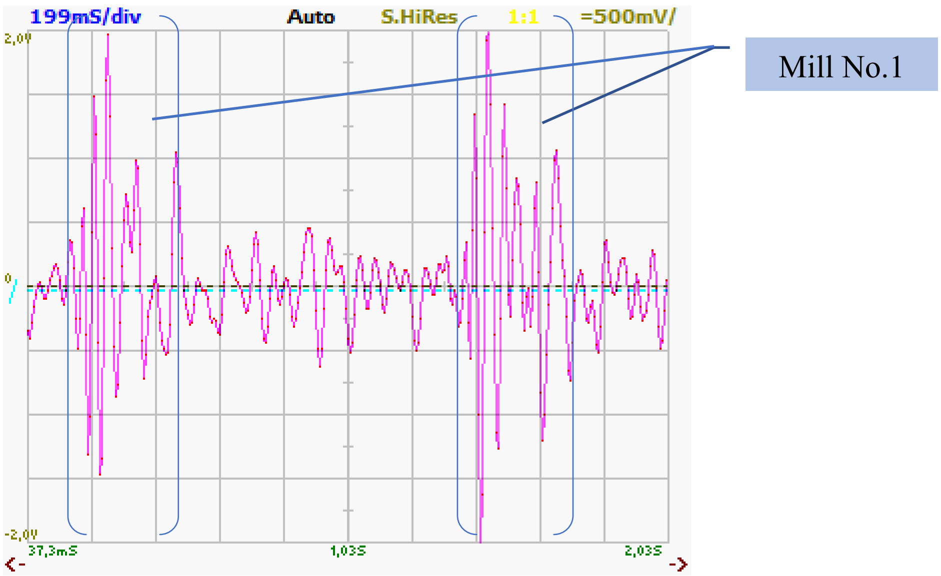

Figure 2 – Oscillogram of vibrations. Point No.1
Pilger mill No.1 is in operation. (1 div. Y axis = 3.75 µm, 1 div. X axis = 0.2 s)

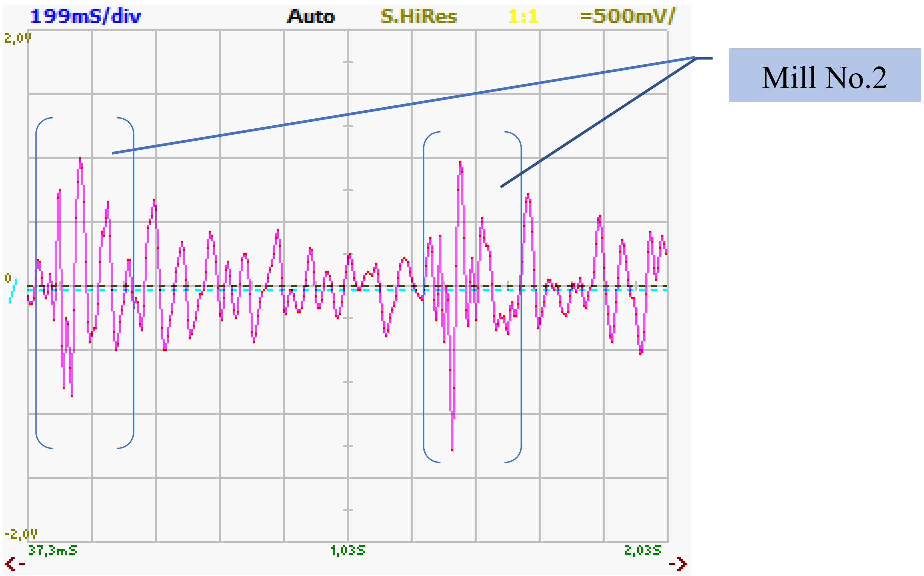

Figure 3 – Oscillogram of vibrations. Point No.1
Pilger mill No.2 is in operation. (1 div. Y axis = 3.75 µm, 1 div. X axis = 0.2 s)

Point No.2

At point No.2, the following vibration parameters have been recorded for operating pilger mills No.1 and No.2: the maximum vibration amplitude has been fixed at the level of 90 µm (mill No.1) and 50 µm (mill No.2) at a pulse frequency of 1.2 Hz (each of the mills separately) (see Fig. 4).

At point No.2 with the operating piliger mill No.1, the following vibration parameters have been recorded: the maximum vibration amplitude has been recorded at 100 µm at a frequency of a pulsed load of 1.2 Hz (see Fig. 5).

With non-operating pilgrim mills No.1 and 2 at point No.2, as well as at point No.1, "background" vibration is observed at a level of ≈ 10 µm at a frequency of 15 Hz (see the spectrogram in Fig. 6).

Point No.3

At point No.3 with the operating pilger mill No.2, the following vibration parameters have been recorded: the maximum vibration amplitude has been recorded at 100 µm at a frequency of a pulsed load of 1.2 Hz (see Fig. 7).

At point No.3, the following vibration parameters have been recorded for operating pilger mills No.1 and No.2: the maximum vibration amplitude has been recorded at 100 µm (mill No.1) and 25 µm (mill No.2) at a pulse frequency of 1.2 Hz (each of the mills separately) (see Fig. 8).

With inoperative pilger mills No.1 and 2 at point No.3, as well as in other currents, there is a "background" vibration at the level of ≈ 20 µm at a frequency of 15 Hz (see Fig. 9).

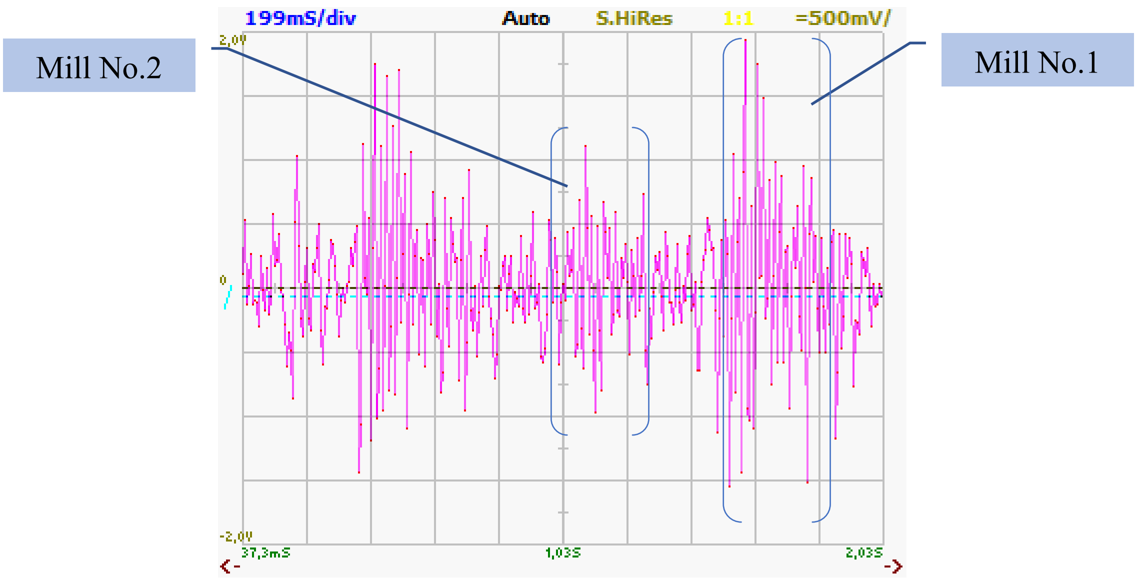

Figure 4 – Oscillogram of vibrations. Point No.2
Pilger mills No.1 и No.2. are in operation (1 div. Y axis = 12.5 µm, 1 div. X axis =0.2 s)

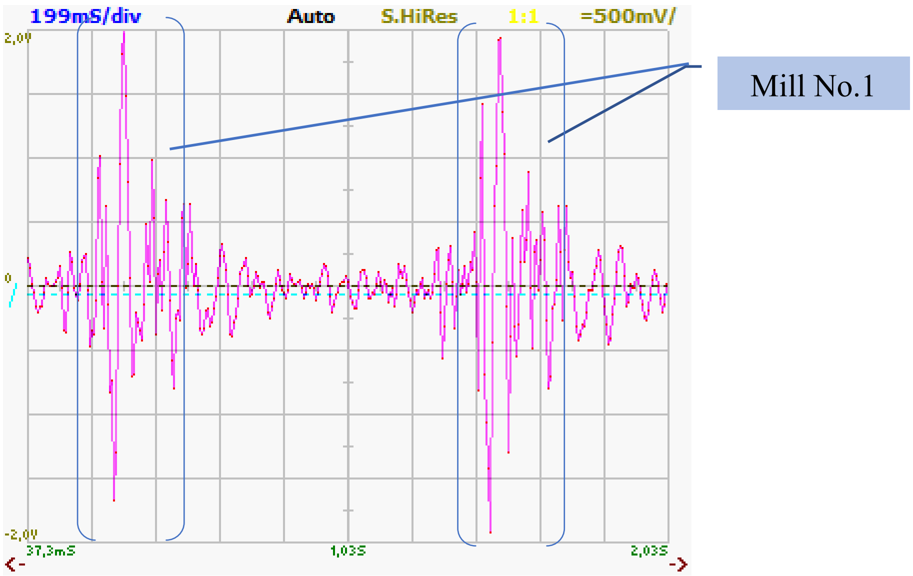

Figure 5 – Oscillogram of vibrations. Point No.2.
Pilger mill No.1 is in operation. (1 div. Y axis = 12.5 μm, 1 div. X axis = 0.2 s)

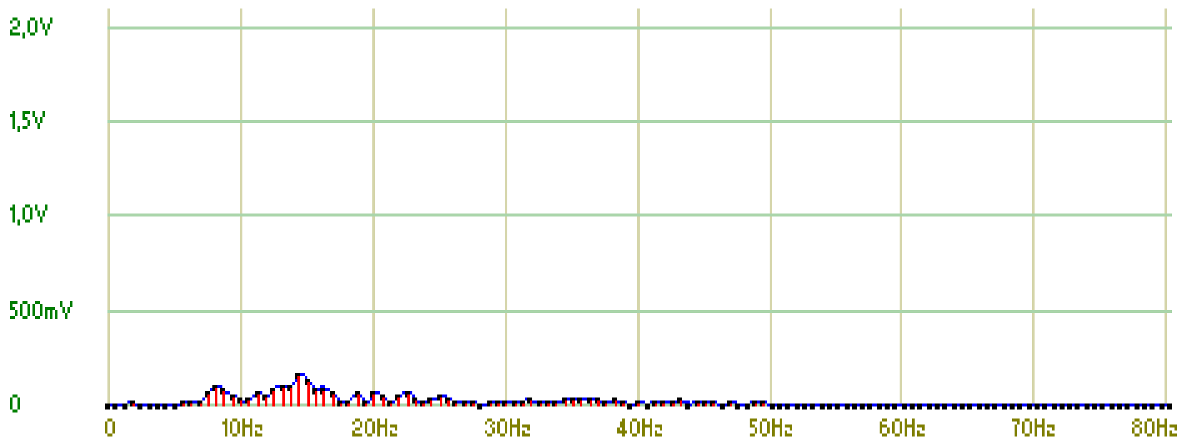

Figure 6 – Spectrogram of the "background" amplitude of vibrations with inoperative mills No.1 and No. 2. Point No.2.

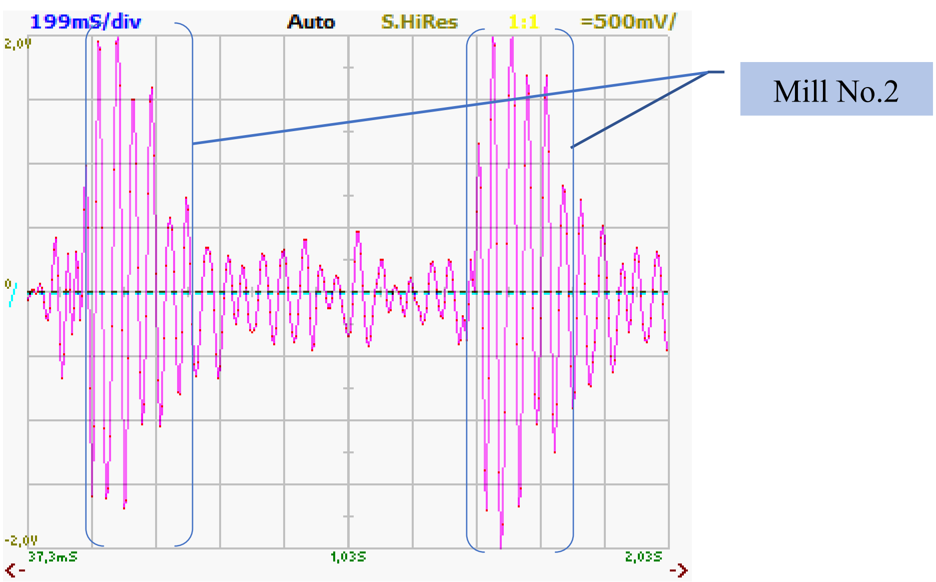

Figure 7 – Oscillogram of vibrations. Point No.3.
Pilger mill No.2 is in operation. (1 div. Yaxis =12.5 μm, 1 div. X= 0.2 s)

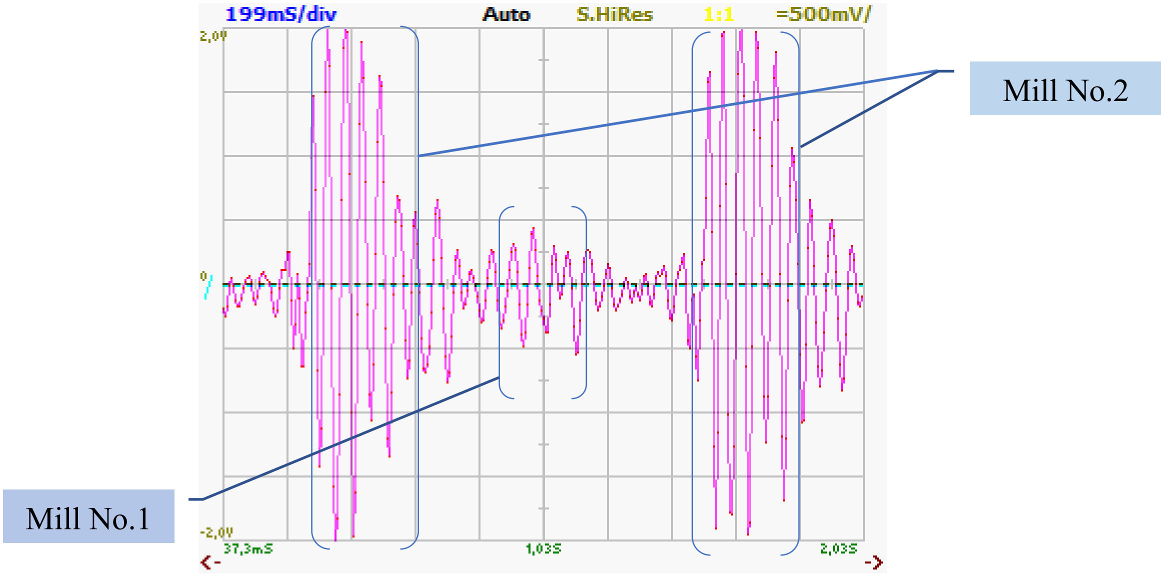

Figure 8 – Oscillogram of vibrations. Point No.3.
Pilger mill No.2 is in operation. (1 div. Y axis = 1.5 µm, 1 div. X axis = 0.2 s)

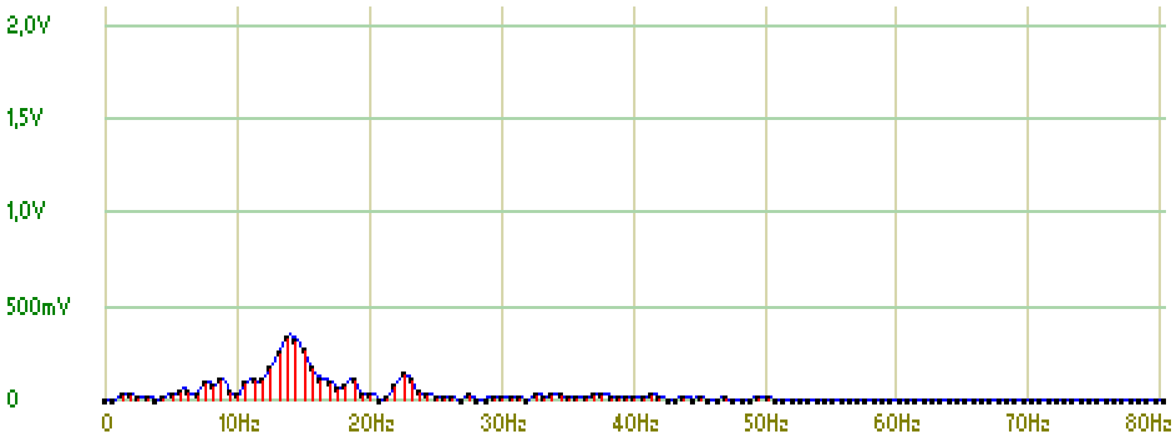

Figure 9 – Spectrogram of the "background" amplitude of vibrations when the mills No.1 and 2 are inoperative. Point No.3.

Point No.4

At point No.4 with the operating pilger mill No.2, the following vibration parameters have been recorded: the maximum vibration amplitude has been recorded at 250 µm at a frequency of a pulsed load of 1.2 Hz (see Fig. 10).

At point No.4, a "background" vibration at a level of ≈ 30 µm at a frequency of 15 Hz is observed, which is somewhat higher than at other points and is probably related to the approach to the source of these vibrations while moving from point No.1 to point No.4 (see Fig. 11).

Thus, in the process of measuring the parameters of vibrations in the pass-through cable tunnel under the industrial facility at the mark -5,200 under the pilger mills No.1 and No.2, the following has been established:

– The maximum amplitude of vibrations from the operation of pilger mills has been determined when pilger mill No.2 was operating at point 4 and is equal to 250 µm at a frequency of a pulsed load of 1.2 Hz.

– High values of vibrations from mill No.2 at points No.1 and No.2 (located under mill No.1); the vibrations from mill No.1 at points No.3 and No.4 (being under mill No.2) practically do not exceed background vibrations. And also, the fact that the maximum recorded value of the vibration amplitude of the cable tunnel when pilger mill No.2 is operating is 2.5 times larger than the amplitude of vibrations when pilger No.1 is operating suggests that the machine No.2 has more influence on the dynamic picture of the cable tunnel.

– With inoperative pilgrim mills No.1 and No.2, the amplitude of forced vibrations of the sections of the pass-through cable tunnel is also non-zero and increases from 10 to 30 µm as it moves from point No.1 to point No.4 (apparently in the direction to the source of vibrations), the frequency of this type of forced vibrations (called "background" in the article) is 15 Hz. Despite their relatively small amplitude, these vibrations can also contribute to the base deformation, compared to the amplitude of vibrations from impulses during the operation of pilger mills, however they have a greater number of effective impulses at a higher frequency.

Apparently, the structure of the foundations of pilger mill No.2 may have a zone of direct transmission of dynamic loads on the structures of the cable channel. This assumption is also supported by the fact that the greatest local violation of the geometry of the tunnel (displacement of sections of prefabricated cable tunnels in a vertical plane relative to each other about 20 cm) is also located in the area of point No.4.

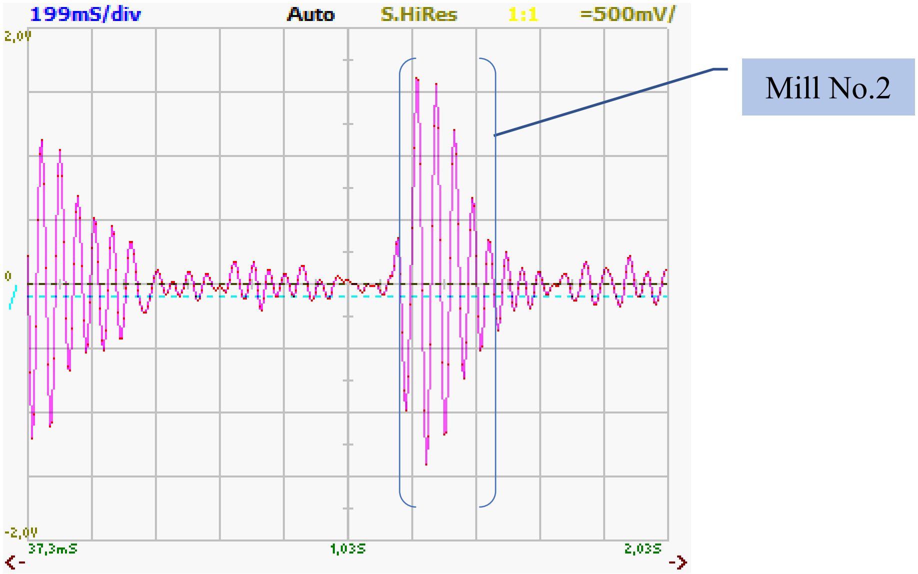

Figure 10 – Oscillogram of vibrations. Point No.4.
Pilger mill No.2. is in operation (1 div. Y axis = 37.5 µm, 1 div. X axis = 0.2 s)

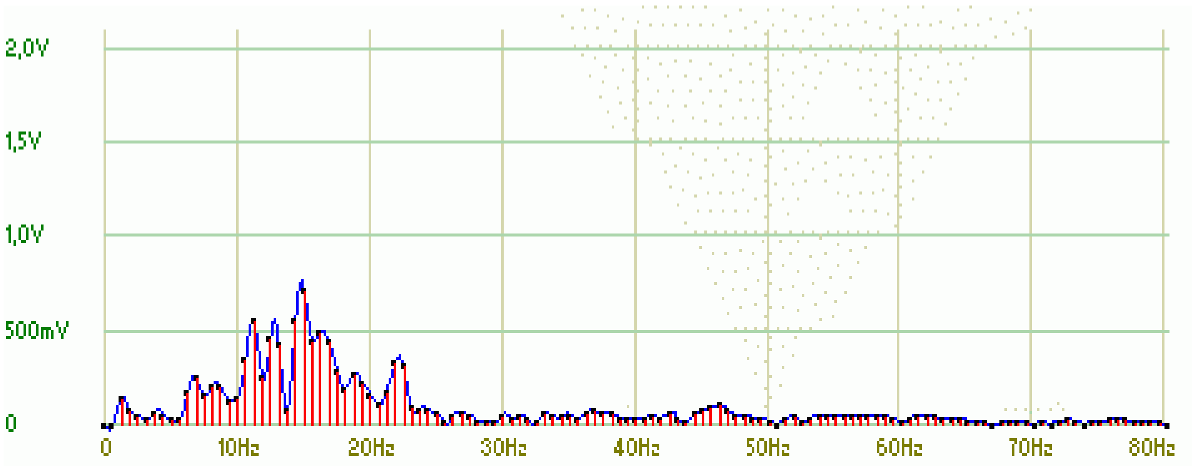

Figure 11 –Spectrogram of the “background” vibration amplitude when mills №1 and №2 are inoperative. Point No.4

Conclusions

The research of vibration parameters at various points along the length of the cable tunnel in the range of vibrations from pilger mills indicates a tendency for increase of vibrations from point No.1 to point No.4. On the plan of an industrial facility, point No.1 is the closest to pilger mill No.1, and point No. 4 is the closest to pilger mill No.2. Based on the nature of the violation of the tunnel geometry, its uneven settlement is observed from point No.1 to point No.4 with an extremum near point No.4. By the nature of the measured parameters of dynamic loads, pilger mill No.2 makes a greater contribution to the dynamic load on the cable tunnel structure with the maximum value near point No.4, which led to the maximum displacement of the cable channel sections near this point. The current situation directly indicates the need for a detailed account of dynamic loads not only when designing foundations for metallurgical machines, but also when constructing other underground structures located in the zone of vibrations propagation.

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Dynamic activity of military transportation investigation at the construction site

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In the paper it is confirmed that any theoretical data inferior to precisely measured data due to the impossibility of considering all the factors influencing on the oscillation process. The fluctuations magnitude of a non-residential building in Poltava and the dynamic influence on the building structure and the people who can be there periodically are experimentally investigated. To evaluate the vibration impact, it is necessary to compare measured data with the permissible level of vibration in public buildings. The recommendations for the further building exploitation are based on the building structures vibration acceleration measurements results.

Keywords: vibration, building, vibration acceleration, vibration velocity, measurement.

Дослідження динамічної дії міського транспорту на будівлі

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Експериментально досліджено величини коливань нежитлової будівлі, що знаходиться в м. Полтава й оцінено їх динамічний вплив на конструкції будівлі та людей, які там періодично перебувають. Для цього проведено вимірювання вертикального віброприскорення покриття будівлі, де очікувалися найбільші значення вимірюваної величини. Для точної оцінки впливу транспортної вібрації на будівлю проведено вимірювання у різні години доби. Коливання виміряно в тій точці конструкції, де їх амплітуда найбільша, і при такому режимі джерела вібрації, при якому відбуваються найбільш інтенсивні коливання цієї конструкції. Виникнення явища резонансу виключається, так як частота вимушених коливань покриття будівлі від впливу транспорту знаходиться у між резонансній області з мінімальним перевищенням 10%. Для оцінювання впливу вібрації на будівлю необхідно порівняно виміряні дані з допустимим рівнем вібрації в громадських будівлях. Для вибору точок і напряму вібрації використовувалася наступна схема вимірів: спочатку регістpуються коливання при будь-якому визначеному динамічному впливі, які забезпечують виявлення форми коливань конструкції і спектра частот коливань. Кріплення акселерометра на покриття будівлі, проводилося за допомогою магніту тобто забезпечується верхня межа робочого частотного діапазону. Для житлових і громадських будівель найбільш несприятливим зовнішнім джерелом вібрацій є рейкові транспортні магістралі: метрополітен, трамвайні лінії і залізниці. Надано рекомендації щодо подальшої експлуатації будівлі, виходячи із результатів вимірювань віброприскорення конструкцій будівлі.

Ключові слова: вібрація, будівля, віброприскорення, віброшвидкість, вимірювання.

Introduction

It is almost impossible to prevent mechanical vibrations, which cause vibration in practice, because they exist due to production processes in the enterprises, technology of construction works and other dynamic phenomena. Equipment with dynamic loads is a source of waves permeated in the soil and affecting on buildings and structures located near the buildings, facilities with equipment and apparatuses, sensitive to vibrations, service personnel and permissible values set by normative documents rarely exceed. Even mechanical oscillations with low amplitude often cause resonance phenomena of structural elements. By the intensity of fluctuations, urban transport is the most important for a person. The vibration occurring in buildings from traffic is regularly interruptible. Technogenic dynamic loads are distributed, as a rule, to the top of the soil mass to a depth of 10-15 m. approximately to such mark the foundations of most public and residential buildings deepen.

Review of research sources and publications

V. Chernyi [7] proposes to apply a heuristic approach based on determining the probability of damage to buildings and structures with dynamic impact. The application of this approach enables to clarify the possible mechanisms of damage to the basis of the object and for certain statistics to more clearly outline the reliability degree. But it needs a rather significant and adequately representative set of injuries and accidents statistics.

The research of S. Tishchenko [4] is also based on the theory where the following assumptions have been used: the environment is regarded as completely incompressible, ignoring the change in its volume; the assumption is assumed that the dynamic action happens instantly. It has been noted that the numerical value of the specific dynamic pulse is determined by the balance of energy transferred to the environment or construction.

But V. Boyko and others [2], on the basis of calculations and experimental data, believe that in the case of inclusion of a square section objects, the maximum stresses are at angular points. That is, the assessment of the ground fluctuations and controversy should be carried out at the corners of the building. But the disadvantage of this method is the requirement for the presence of complex measuring equipment. The problems of studying the dynamic load on the design of buildings and structures are devoted to the work of V. Shvets, V. Shapovala, M. Holmyansky, A. Perelmuter, V. Karpilovsky, S. Fialko, K. Yegupova, M. Swinkin [1; 3; 5; 6; 10-12]. Recently, the market for software engineering calculations offers a lot of domestic and foreign developments, allowing a fairly reliable performance of bearing structures calculations. Unfortunately, the area of calculations related to geotechnical engineering, which is based on the processes of interaction between foundations and soils, is much less developed. Each program has both its advantages and disadvantages in terms of solving a specific problem.

Definition of unsolved aspects of the problem

But any theoretical data yields precisely measured data, due to the impossibility of considering all the factors that influence the oscillation process. So Ye. Nesmashny, V.D. Sidorenko [1] have proposed a method for multichannel oscillation measurements to determine the level of seismic soil oscillations in the building basis. At the same time, the vector module of the maximum velocity V_{max} of oscillations was determined on the three its projections basis on the coordinate axes (two horizontal V_x and V_y and one vertical V_z) in this way: based on the maximum speed value, the level of seismic oscillations was determined according to the scale in balls.

Problem statement

Problem statement is the experimental study of the non-residential building located in Poltava fluctuations magnitude and the evaluation of the dynamic effect on the building structure and the people who are periodically there; to provide guidance on available dynamic impacts on the building.

Basic material and result

Before starting building structures vibrations measuring in order to determine the dynamic state, it is necessary to obtain information constructions design scheme describing: the type, the dimensions of spans and cross-sections, nodes design of the construction elements joints, the masses of the structure and the constructions attached thereto distribution, other constructive characteristics, which affect the stiffness and mass of structures, the characteristics of the adjacent to the construction structures of equipment with dynamic loads: vibration direction, which predominate, the state of deformation by the project (anti-vibration) stitches around the perimeter of existing equipment.

Figure 1 – Device for measuring vibration parameters «Vibrometer 107V»:
1 – accelerometer; 2 – probe; 3 – vibrometer

Vibration sources in residential and public buildings are engineering and sanitary-technical equipment and industrial installations and vehicles (subway of shallow foundation, heavy trucks, railway trains, trams), which create at work large dynamic loads that cause the propagation of vibration in soils and buildings construction structures. These vibrations are also often the cause of noise in the buildings.

The permissible vibration level of these structures is determined not only by the need to ensure the structures carrying capacity with the joint action of static and dynamic loads, but also the boundaries that exclude the possibility of harmful effects on people.

In the time of evaluating the strength and structures, fluctuations reliability can be considered harmless, if for buildings walls and columns the difference between the horizontal dynamic displacements of the lower and upper ends of the column in the boundaries of the floor does not exceed 1/50000 height of the floor. In this case, the oscillations are measured at that point of the design, where their amplitude is greatest and in such a vibration mode source, where the most intense oscillations of this construction occur [9].

That is, the admissible amplitude of structures oscillations

$$A_{adm} = 1/50000\ h_f = 1/50000 \times 3.3\ \text{m} = 0.066\ \text{mm}.$$

The spectral composition of the measured oscillations is by a large excess over the background value characterized in the octave bands frequency of 31.5-63 Hz. with the distance from the amplitude source amplitude of oscillation decreases.

The influence of transport vibration on the building of the «Family World» store was explored at the address Poltava city Lenin str. 10/19 where on the road cars, freight transport, buses, trolley buses move at the distance 4 meters from the building foundation. The storehouse «World of Family» is frame one and have 4 floors. To assess the impact measurements of vertical vibration acceleration of the building coverage were made, where the highest values of the measured value were expected.

Vibrometer 107B was used for oscillation parameters measurements. Vibrometer 107B is an autonomous, microprocessor measuring device was designed to measure oscillation parameters: vibration acceleration, vibration velocity and vibrational displacement (Fig. 1). At the same time, a spectral analysis of the vibration signal is carried out. For measuring, a piezoelectric accelerometer type DN-3 was used. No. 1155. The dynamic range of measuring the mean-square values of vibration acceleration, vibration velocity and vibrational displacement is limited above the maximum value of the input signal and from the level bottom of the signal amplifier own noise and depends on the conversion factor. The limits of the relative error in measuring the signal (without considering the accelerometer accuracy) are ±5%. frequency range of spectral analysis from 10 Hz to 10 000 Hz.

The instrument sensor is directly installed on the bearing elements surface. Contribution to the vibration of building structures of several vibration sources is determined by their alternating switching off and on. The free (own) oscillations frequencies establishment is necessary in the presence of resonant phenomena (with the frequencies coincidence of the own design oscillations with forced oscillations from sources of vibration). The equipment adjacent to the construction structures during free-range measurements was completely or partially disabled.

By the place of action distinguish vibration:

a) at workplaces of enterprises industrial premises;

b) at workplaces in warehouses, dining rooms, household, duty and other industrial premises, where there are no machines producing vibration;

c) at workplaces in the premises of the plant administration, design bureaus, laboratories, educational centers, computer centers, health centers, office premises, working rooms and other premises for employees of mental labor; the total vibration in residential areas and public buildings from external sources: urban rail transport (small deposits and open subway lines, trams, railways) and motor vehicles.

Table 1 – Characteristics of oscillations effect on people, depending on the speed and acceleration of harmonic oscillations with an amplitude of no more than 1 mm

Characteristic of fluctuations effect on people	Extreme fluctuations acceleration Wmax	Extreme fluctuations velocity Vmax (mm / s)
Untangible	10	0,16
Weak tangible	40	0,64
Well tangible	125	2
Very tangible	400	6,4
Harmful with prolonged exposure	1000	16
Definitely harmful	1000	16

Sanitary norms are obligatory for all ministries, departments, enterprises, associations, organizations, institutions, regardless of departmental affiliation and ownership forms; The requirements of these norms should be considered in the normative and technical documents: standards, building codes, technical specifications, instructions, methodological instructions, etc., which regulate the design and operational requirements for vibro hazardous machinery, equipment, equipment and tools, technological processes and regulations, overseas products. Therefore, it is necessary to evaluate the effect of vibration from transport.

The following measurement scheme was used to select the points and direction of vibration: firstly, oscillations are recorded at any given dynamic effect, which ensure the detection of the structure oscillations shape and oscillation frequencies spectrum. Accelerometer attaching to building cover, was carried out using a magnet providing the upper limit of the working frequency range.

By the fluctuations intensity urban transport is most important for a person. The vibration occurring in buildings from traffic is regularly interruptible. The spectral composition of the measured oscillations is characterized by a large excess over the background value in the octave frequency bands of 31.5-63 Hz. with the removal of the oscillation amplitude decreases.

For residential and public buildings, the most unfavorable external vibrations sources are railways: metro, tram lines and railroads. Studies have shown that fluctuations with distance to a different distance from the subway are extinguished, but this process is nonmonotonic, it depends on the components in the way of the vibration propagation: the rail - the wall of the tunnel - the soil - house foundation - building structures.

As a result of the measurements first stage implementation, the points and directions of oscillations registration, which are most characteristic for this dynamic process, are revealed.

By installing the device in these characteristic points, it is obtained the dependences of the measuring parameters (amplitude, frequency, etc.) on the modes of oscillations sources. The instrument sensor is installed directly on the elements bearing surface. The equipment adjacent to the construction structures during free-range measurements was completely or partially disabled.

For accurate assessment of transport vibration impact on the building, measurements were made at different time of the day. The greatest impact is expected at rush hours from 17:00 to 18:30. At this time, public transport transports the largest number of passengers. But for comparison, vibration acceleration was measured at 10.00 – 12.00 h. Measured data are reduced to table 1. to the wall of the 3rd floor fixed cable line of tension for the trolley bus. During the trolley bus movement, there are fluctuations of the 3rd floor wall, so it is necessary to check by means of measurements whether they enable vibration accelerations in the vertical and horizontal directions. Data is listed in Table 1.

To evaluate the impact of vibration, it is necessary to compare measured data with the permissible level of vibration in public buildings. The permissible level of vibration in public buildings is a factor that does not cause significant people embarrassment and significant changes in the functional state of systems and analyzers sensible to vibrational influences. According to [9], under the action of a constant local and general vibration, the normalized parameter is the vibration velocity mean-square value (V) and vibration acceleration (a) or their logarithmic levels in decibel.

The frequency range prevailing on the 3rd floor and covering the building according to the measured measurements from 17.7 to 45.95 Hz. according to Table 2, it is set the permissible values of vibration acceleration at the appropriate frequency, which are presented in Table 1.

According to [8, 9], in the requirements absence for limiting the amplitudes of oscillations associated with the placement of precision equipment or the systematic presence of service personnel, the permissible amplitudes of vertical fluctuations of the coating structures are determined by the carrying capacity and the maximum allowable dynamic deflection of the structure. The amplitudes of oscillation in mm, corresponding to the maximum allowable dynamic deflection of the structure, are determined depending on the frequency of forced oscillations by the formula.

Table 2 – The values of the building measured vertical vibrations

Frequency Hz	Average vibration acceleration m/s^2	Limit-permissible vibration acceleration m/s^2	Mark	Notes
Vibration coverage at 17.00-17.30				
49,95	0,0159	0,093	3	Vibration acceleration in the vertical direction
49,95	0,01980	0,093	3	
49,95	0,020	0,093	3	
49,95	0,01847	0,093	3	
49,95	0,01947	0,093	3	
49,95	0,01972	0,093	3	
Vibration on the 3rd floor at 17.00-17.30				
41,7	0,00671	0,038	2	Vibration acceleration in the vertical direction
41,7	0,00528	0,038	2	
45,95	0,00745	0,093	2	
45,95	0,00714	0,093	2	
45,95	0,00687	0,093	2	
44,01	0,03164	0,048	3	Vibration acceleration in the horizontal direction

Table 3 – Acceptable values of vibration in the administrative buildings and in public buildings according to [9]

Middle geometric frequency of bands, Hz	Vibration acceleration m/s^2
16	0,0019
32	0,012
63	0,42

For frequencies of oscillations from 10 to 100 Hz (in this range are the frequencies of forced fluctuations of coverage from traffic)

$$A_0 = \frac{1}{n_0}, \tag{1}$$

where n_0 – the frequency of forced oscillations, Hz.

Acceptable amplitudes (vibrational displacement) calculated by the above formula for the coating are presented in Table 1.

According to the normative documents, the amplitude structures forced oscillations at the same amplitude of the active force and at other equal conditions depends on the frequency ratio of forced oscillations to the structure of own oscillations frequency. At the frequencies coincide, the phenomenon of resonance occurs in a number of cases, the amplitudes of oscillations increase sharply. Therefore, it is necessary to investigate the possibility of resonance. The own frequency of the store building «The world of the family» by the street. Lenin 10/19 is calculated. It is determine the own oscillations frequency according to the empirical formula

$$\lambda = \frac{1}{T} = \frac{1}{0{,}0905\mu\sqrt{b}}, \text{ Hz} \tag{2}$$

$$\mu = \frac{H}{b} = \frac{14.2}{208.12} = 0.068 \tag{3}$$

where H – height of the building, m. Fluctuations have been measured on the roof of a 4 storey building, so it accepted the building height – 14,2 m; b – the size of the building in plan, m^2.

$$\lambda = \frac{1}{0.0905 \cdot 0.068 \cdot \sqrt{208.12}} = 11.24 \text{ Hz.} \tag{4}$$

So, the frequency of forced fluctuations of the building 3rd floor is 41,7 – 45,95 Hz, the excess of the forced oscillations frequency over it's own frequency is $\frac{41.7-11.02}{41.07} \cdot 100\% = 75\%$ The phenomenon of resonance does not occur.

According to the normative documents [9], the coating designs have a number of their own oscillations frequencies, which sequence is arranged in order of growth, called the spectrum. To each frequency spectrum corresponds its own form of oscillation. At calculating, there is no need to calculate the full spectrum of the own oscillations frequencies, but it can be limited to a reduced spectrum, by calculating several necessary frequencies of the spectrum. The number of the reduced spectrum of the coating structures own oscillations frequencies is determined depending on the forced oscillations source frequency so that the last from the calculated frequencies of the reduced spectrum are higher than the forced oscillations source frequency. Loads without mass (wind, inertial) in determining the frequencies and internal oscillations forms to the calculation are not accepted. To cover the frequency of their own vertical vibrations are determined by the formula:

$$n_s = 0.159\varphi_s^2 \sqrt{\frac{EI}{ml^4}}, \tag{5}$$

where n_s – internal oscillations frequency per s tone in Hz;

φ_s – frequency coefficient, on the type of construction depending;

E – module of longitudinal material elasticity, kg / sm^2;

I – the cross section inertia moment, cm^4; coating thickness ≈ 60 cm;

$$I = \frac{60 \cdot 860^3}{12} + \frac{60 \cdot 480^3}{12} = 37.32 \cdot 10^8$$

m – weight of coverage length unit, kg;

$m = 1 \cdot 0.6 \cdot 2400 = 1440$ kg.

l – estimated coverage length, m.

The own oscillations frequency of 7 tone:

$$n_7 = 0.159\varphi_s^2 \sqrt{\frac{EI}{ml^4}} =$$

$$= 0.159 \cdot (7 \cdot 3.14)^2 \sqrt{\frac{20 \cdot 10^4 \cdot 37.32 \cdot 10^8}{1440 \cdot 1340^4}} = 30.73$$

The own oscillations frequency of 8 tone:

$$n_8 = 0.159 \cdot (8 \cdot 3.14)^2 \sqrt{\frac{20 \cdot 10^4 \cdot 37.32 \cdot 10^8}{1440 \cdot 1340^4}} = 40.13$$

The own oscillations frequency of 9 tone:

$$n_9 = 0.159 \cdot (9 \cdot 3.14)^2 \sqrt{\frac{20 \cdot 10^4 \cdot 37.32 \cdot 10^8}{1440 \cdot 1340^4}} = 50.79$$

The calculation error of own oscillations frequencies due to materials properties changes, deviations from the calculation scheme from the actual modes of structure operation is considered by own oscillation frequencies ε error coefficient the by the formula

$$n_s^{/} = (1+\varepsilon)n_s \text{ Hz.} \tag{6}$$

The own oscillations frequency of 7 tone:

$n_7^{/} = (1+0.15)30.73 = 35.33$ Hz.

The own oscillations frequency of 8 tone:

$n_8^{/} = (1+0.15)40.13 = 46.14$ Hz.

The own oscillations frequency of 9 tone:

$n_9^{/} = (1+0.15)50.79 = 58.41$ Hz.

As the frequency of forced fluctuations building coverage from the transport impact is from 49.95 Hz and 41.07 Hz, which is in the interresonance region. The forced oscillations excess over it's own frequency of 8 tone: $\frac{49.95-46.14}{49.95} \cdot 100\% = 10\%$. The forced oscillations excess over it's own frequency of 9 tone: $\frac{41.07-35.33}{41.07} \cdot 100\% = 14\%$. The resonance phenomenon does not occur.

Conclusions

Comparing the measured mean-square values of vibration acceleration and on the 3rd floor of the store «World of the Family» at Lenin str. 10/19 with the maximum permissible values, it is concluded that the measured mean-square values of vibration acceleration do not exceed the maximum-permissible values. The resonance phenomenon emergence is excluded, as the transport impact building coverage frequency is between the resonant area with a minimum excess in 10%. Further building exploitation at such values of dynamic loads is possible.

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Scientific basis of design structures plaster solutions

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High-rise construction volume increase and new wall materials use require changing the approach to the design of plaster mortar compositions. The analysis has showed that it is possible to reduce the number of cracks in the plaster coating by increasing the water holding capacity of the mortar mixture. To optimize the prescription parameters of the mortar mixture, the five-factor experiment with fine aggregate and the filler with a low modulus of elasticity, disperse polymeric powders and cellulose ethers, a polymer fiber for microdispersed reinforcement has been used. The obtained data indicate that the proposed approach enables to obtain plaster mortars with physic mechanical characteristics that provide optimal working conditions “masonry - plaster coatings”.

Keywords: plaster solutions, new basic principles for designing their compositions

Наукові основи проектування складів штукатурних розчинів

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Збільшення висотності будівель, застосування нових стінових матеріалів, вимагає переосмислення підходу до проектування складів штукатурних розчинів. Вирішити поставлене завдання в рамках існуючих принципів неможливо. У статті проведений аналіз системи «штукатурне для кладки покриття» на стадіях твердіння і експлуатації, виявлені руйнівні чинники і механізм її руйнування. Відмічено, що основною причиною руйнування системи є деформації і напруги, що виникають в кладці, штукатурному покритті і контактній зоні між ними, із-за силових дій, температури і вологості, агресивних рідин і газів. Проведений аналіз показав, що зменшити кількість тріщин в штукатурному покритті, на стадії тверднення, понизити напругу в системі, можна шляхом підвищення водоутримуючої здатності розчинної суміші, зменшення усадки і модуля пружності штукатурного покриття, підвищення його еластичності. Для цього запропоновано використовувати дрібний заповнювач і наповнювач з низьким модулем пружності, редісперсні полімерні порошки і ефіри целюлози, полімерну фібру для мікродисперсного армування та ін. Для оптимізації рецептурних параметрів розчинної суміші використовували п'ятифакторний експеримент. Отримані дані свідчать про те, що запропонований підхід дозволяє отримувати штукатурні розчини з фізіко-механічними характеристиками що забезпечують оптимальні умови роботи «кладки- штукатурне покриття». Застосування запропонованих компонентів дозволяє підвищити водоутримуючу здатність розчинної суміші, зменшити усадку штукатурного покриття і кількість тріщин на стадії тверднення. Збільшити еластичність і знизити модуль пружності штукатурного покриття, напруги в ньому і швидкість розвитку тріщин на стадії експлуатації, що дозволить підвищити його довговічність і стінової конструкції.

Ключові слова: штукатурні розчини, нові базові принципи проектування їх складів

Introduction

The increase of high-rise construction share and the widespread use of new wall materials require changing the approach to the design of plaster mortar compositions. Is is due to the fact that the impacts and loads on the plaster coating located on the 24th floor of the building are significantly different from those that it experiences on the 1st to 3rd floors (Fig. 1).

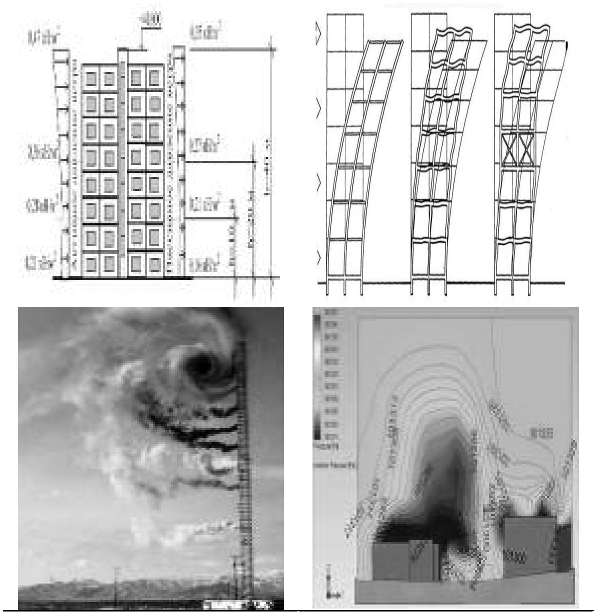


Figure 1 – Change impacts and loads on the plaster coating while increasing high-rise buildings

It is also necessary to consider the fact that when building high-rise buildings on the territory of Ukraine, the autoclaved aerated concrete with an average density of 150-600 kg / m3 is mainly used. Its properties (compressive strength, modulus of elasticity, temperature deformation) depend on the average density and significantly differ from the properties of traditional wall materials (ceramic and silicate bricks, blocks of lightweight concrete and rocks).

To achieve goal in the framework of existing ideas and principles is not possible. They are applicable for solutions used in low-rise construction. Normative requirements for plaster mortars for autoclaved aerated concrete walls are also not considered; they are contradictory and, in our opinion, are not substantiated. For example, compressive strength should be from 1.5 to 7.5 MPa (Russia), 2.5 MPa (Ukraine) and 10 MPa (Germany). The flexural strength should be from 1 to 1.25 MPa (Ukraine) and 2 MPa (Russia). The value of adhesion to aerated concrete laying should be from 0.15 to 3 MPa (Russia) and 0.5 MPA (Ukraine).

Problem statement

To achieve the required goals, the development of new scientific bases for the design of plaster mortar compositions is required. It is necessary to analyze the processes occurring in the plaster coating when it is applied and hardened, the knowledge of the destruction mechanism of the system "masonry - plaster coating", calculation and account of stresses.

The durability of the wall structure depends to a large extent on the number of defects in the plaster coating and the contact area between it and the masonry. The destruction of the system "masonry - plaster coating" is due to the accumulation and development of micro- and macro-cracks in it. To assess the service life of such a system, it is necessary to determine the internal and external factors, their impact degree, the calculation of the stress state, knowledge of the nucleation processes, the accumulation of damage and the macrocracks growth. The assigned physical and mechanical parameters and compositions of plaster solutions should ensure the "work" of the system at the maximum level of such stresses.

Basic material and results

Masonry made of autoclaved aerated concrete has a high capillary potential due to the considerable pore volume (520 mm^3/g) and their high specific surface area (22-34 m^2/g) [1]. During the application of the mortar mixture to the masonry, because of its low water-retaining capacity, the liquid is pumped out of it, with lyophilic pores and capillaries of the masonry material (Fig. 2a). The pore filling rate (v) is determined by the Poiseuille equation

$$v = -\frac{r^2 \Delta_p}{8\eta l} \ , \tag{1}$$

when l – length of the area of the absorbed liquid;
η – viscosity of the absorbent section;
Δ_p – the pressure drop across section l, equal to the capillary pressure of the meniscus

$$\Delta_p = \frac{-2s_{12}\cos q}{r} \ . \tag{2}$$

Therefore, the dissolution of cement and the formation of a supersaturated solution mortar with reduced water content. Because of this, incomplete hydration of cement occurs, nonequilibrium, metastable neoplasm of the following type is formed:

$$\begin{aligned} &1{,}83(3CaO{\cdot}AI_2O_3)+1{,}48(CaSO_4{\cdot}2H_2O)+\\ &+3{,}38Ca(OH)2+16{,}67H2O \rightarrow\\ &0{,}42(3CaO{\cdot}AI2O3{\cdot}6H2O)+\\ &+1{,}42\ (3CaO{\cdot}AI2O3{\cdot}CaSO4{\cdot}12H2O)\\ &+3{,}38Ca(OH)2+0{,}07CaSO4{\cdot}2H2O \end{aligned} \tag{3}$$

The loss of water leads to shrinkage of the plaster solution - 2.5 ... 5.8 mm/m [2]. And since the aerated concrete masonry "holds back" these deformations, it leads to stresses in it (δ), which are seven times higher than its tensile strength

$$\delta = \frac{\Delta\varepsilon^{*}\cdot E}{1-\mu} \ . \tag{4}$$

when E and μ – modulus of elasticity and Poisson's ratio of plaster coating;
$\Delta\varepsilon^*$ – difference in deformations of plaster and aerated concrete base [5].

Because of these stresses and the fact that a decrease in the degree of cement hydration has led to a decrease in the ultimate extensibility of the material (by 20 ... 50%) [4, 7, 8], cracks develop in the plaster coating (on the surface and in the volume of the material), as well as in the contact zone with the masonry (Fig. 2c, 2d, 2e).

During operation they "evolve" and merge into trunk. The causes of cracks are temperature and humidity deformation plaster and masonry [3,4] and the difference between them (Fig. 3c, 3e), the voltage caused by them (Fig. 3d), moisture, ice, corrosive materials.

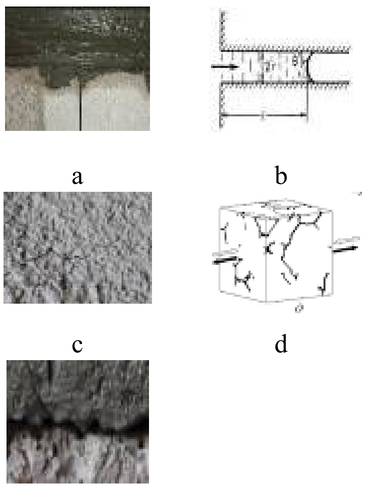


Figure 2 – Cracking in the plaster coating and the contact zone between them

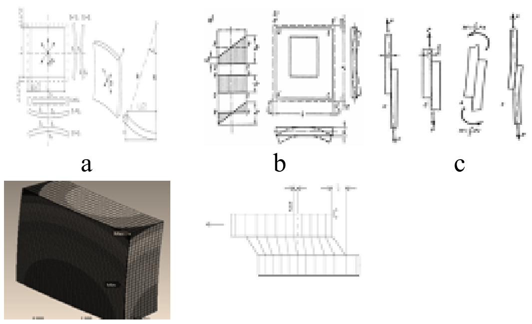


Figure 3 – Temperature and humidity deformation of the masonry (a, b) and stresses in it (d)

Cyclic temperature-humidity effects lead to deformation (ΔL_p, ΔL_c, ΔL_{tv}) and additional stresses in the plaster coating (δ), which are the development cause of the whole family of main cracks.

$$\Delta L_p = \alpha\, \Delta T^p\, L\ ; \qquad (5)$$

$$\Delta L_c = \alpha\, \Delta T^c\, L\ ; \qquad (6)$$

$$\Delta L_{tv} = \alpha\, \Delta T\, L\, F_m + \Delta L_q\, L\ .; \qquad (7)$$

where: ΔL_p, ΔL_c – deformation of tension and compression;

ΔL_{tv} – heat and humidity deformation;

α – coefficient of temperature elongation of the material;

ΔT^p, ΔT^c – is the temperature difference;

L – is the wall element length;

F_m – an indicator of material defects;

ΔL_q – humidity deformations [3].

$$\delta = k\sqrt{[a_t + b(L-c)]\cdot\delta^*}\ ; \qquad (9)$$

$$\delta^* = E\cdot a_t\cdot\Delta t\ ,$$

where E – modulus of elasticity;

α_t – coefficient of linear expansion of plaster coating;

Δt – calculated temperature drop.

Masonry and plaster coating have different in magnitude thermal deformations. The magnitude of these deformations is determined from the expression [5]:

$$\Delta L = L_0\cdot\alpha_t\cdot\Delta t\ ; \qquad (10)$$

$$\Delta t = t_2 - t_1\ , \qquad (11)$$

where ΔL – the wall structure elongation or contraction is relatively;

L_0 – the length of the wall structure at the time of construction;

α_t – coefficient of thermal expansion and autoclaved aerated plaster coating [1,5];

Δt – changing the temperature of the wall structure;

t_1 – environmental temperature at the time of construction of gas concrete masonry and applying stucco coatings;

t_2 – the maximum and minimum temperature, which affects the wall structure in summer and winter periods;

In winter, at -20°C, for a laying length of 8 m, made in summer at a temperature of + 30°C, with a coefficient of temperature expansion of aerated concrete laying of 8 · 10-6 grad^{-1} and a temperature change from +30 to -20°C, e=50°C, the total compression deformation is 3.2 mm. In summer, when heated to + 80°C [1], the expansion deformation is 3.2 mm.

In winter, the compression deformation of the plaster solution (1:4) is 0.55 mm/m, and the total deformation of the compression, the plaster coating of the wall 8 m long, is 4.4 mm. In summer of the total deformation expansion of the plaster coating is 4.4 mm. Deformations of expansion and contraction cause tension (σ) in the masonry and the plaster covering which can be determined by converting the equation:

$$\frac{\Delta L}{L_0} = \frac{\sigma}{E} \qquad (12)$$

where ΔL – the elongation or contraction of the wall structure;

L_0 – length of the wall structure at the time of erection;

σ – stresses in H/mm^2;

E – is the modulus of elasticity in H/mm^2 [5].

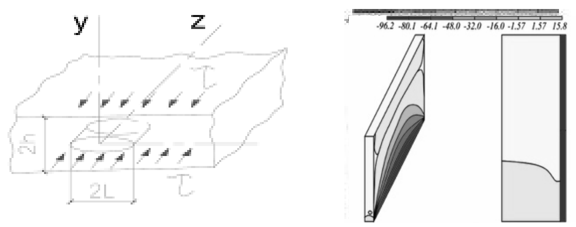
The difference between the deformations, the elasticity modules of the masonry and the plaster coating, is the reason for the shear strains in the contact zone "masonry - plaster coating" (Figures 3e, 4a) and stresses (τ) (Fig. 4b), which predetermine the development of a crack in the contact zone:

$$\tau = \frac{\Delta T_1 \alpha_1 - \Delta T_2 \alpha_2}{\frac{1}{E_1} + \frac{1}{E_2}}, \quad (13)$$

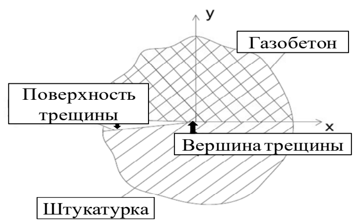
where τ – shear stress from temperature deformations, kgf/cm^2;
ΔT_1, ΔT_2 – the temperature difference at the time of installation and operation of the plaster coating and masonry, ºC;
α_1, α_2 – coefficient of thermal expansion of masonry and plaster coating;
E_1, E_2 – elasticity module of masonry and plaster coating, kgf/cm^2.

Atmospheric moisture, penetrating into the cracks of the plaster coating, and through them into the contact zone, creates a wedging pressure at the top of the crack. Thus, tensile stresses appear in this zone (Fig. 5a), which leads to further development of the main cracks plaster coating and its contact zone with the masonry.

At minus temperatures, in winter, the development of cracks is accelerated due to the transformation of water into ice (Fig. 5b, 5c), which ultimately leads to the destruction of the plaster coating and the wall structure (Fig. 5d, 5d).



a b



c

Figure 4 – Deformations (a), stresses in the contact zone (b) and development of the main crack (c)

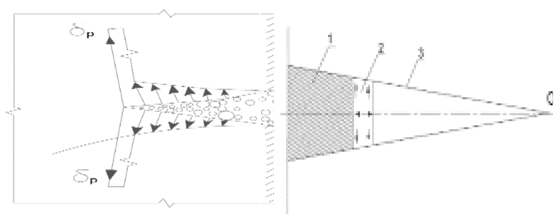
In order to prevent the destruction of the plaster coating, it is necessary to calculate the values of its physic mechanical characteristics (compressive and bending strength, elastic modulus, etc.), considering the stresses in it and the contact zone with the cladding that arise when the solution is hardened and the wall structure is deformed. In this case, the maximum permissible voltage, should be less than destructive. Selection of the composition of the solution must also be carried out taking into account the processes occurring during the application and hardening of the mortar to the masonry. It is necessary to increase the water-retaining capacity of the mixture, to reduce the shrinkage of the plaster coating during hardening, to reduce the number of cracks that occur during hardening and to prevent or slow down their development.

The aim of the research was to obtain plaster solutions with high crack resistance, while ensuring the requirements of normative documents for medium density, compressive strength and bending, and other parameters.

In the experiment, to reduce the shrinkage of the plaster coating and prevent the appearance of shrinkage cracks, a small filler and a filler with a low modulus of elasticity (vermiculite and from aerocrete battles (mix No. 1), limestone stone waste and pearlite waste (No. 2 mix) were used.

The resulting effect was enhanced by the addition of redispersible polymer powder Winnapas 5043 H and cellulose ethers Tylose MBZ 15009. Their presence enables to reduce the modulus of the plaster coating elasticity and the stress in it and the contact zone, and with the polymer fiber addition, the cracks development rate is reduced, which increases the durability of the coating and the wall structure.

To determine the properties of the plaster solution, samples of a 40×40×160 mm beam were made on a gas-concrete base. Curing regime air-dry, simulating the work of plaster coating in real conditions.



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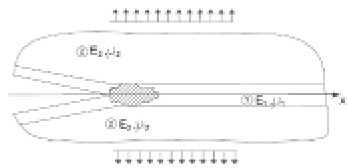

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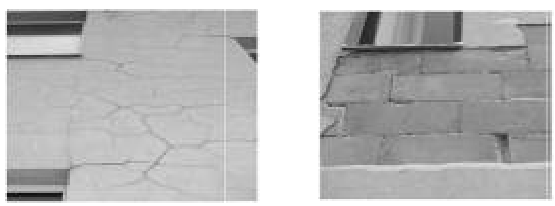

d e

Figure 5 – Development of the main crack in the contact zone due to the action of water (a), ice (b, c), and destruction of the plaster coating (d, e)

Table 1 – Factor variation range

Type of mixture	The range of variation	Binder consumption, kg/m^3	Filler and aggregate consumption, m^3/m^3	Fiber consumption, kg/m^3	Redispersible polymer powder consumption Winnapas 5043 H, %	Consumption Tylose MBZ 15009,%
		X1	X2	X3	X4	X5
Mix №1	1	500	1,05/1	1,2	5	0,5
	0	400	1,05/1	0,9	3	0,3
	-1	300	1,05/1	0,6	1	0,1
Mix №2	1	400	1,05/1	1,2	5	0,5
	0	300	1,05/1	0,9	3	0,3
	-1	200	1,05/1	0,6	1	0,1

Curing regime air-dry, simulating the work of plaster coating in real conditions. After 28 days of hardening were determined: compressive strength and bending strength, high density, etc. Fracture toughness was determined visually by the presence of cracks in the coating and calculating the fracture toughness ratio, as the ratio of the bending strength to compressive strength.

The result is plaster solutions with the following properties:

Composition No.1 (Fig. 6a-d):
– average density of 600-1500 kg/m^3,
– bending strength 12-18 kg/cm^2,
– compressive strength 18-36 kg/cm^2,
– crack resistance coefficient 0.56-0.74.

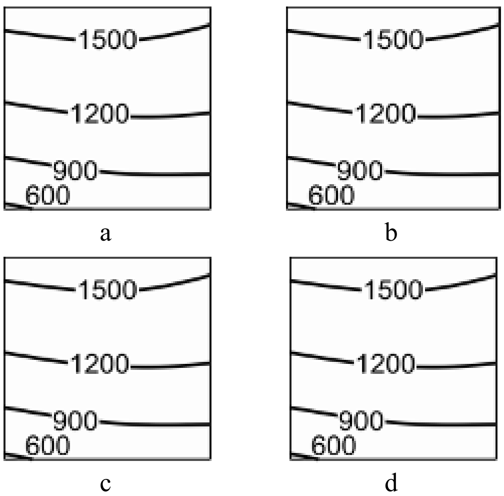

Figure 6 – Average density (a), flexural strength (b) and compression (c), crack resistance (d) of composition №1

Composition No.2 (Fig. 7a-d):
– average density of 700-1100 kg/m^3,
– bending strength 10-25 kg/cm^2,
– compressive strength 15-35 kg/cm^2,
– crack resistance coefficient 0.25-1

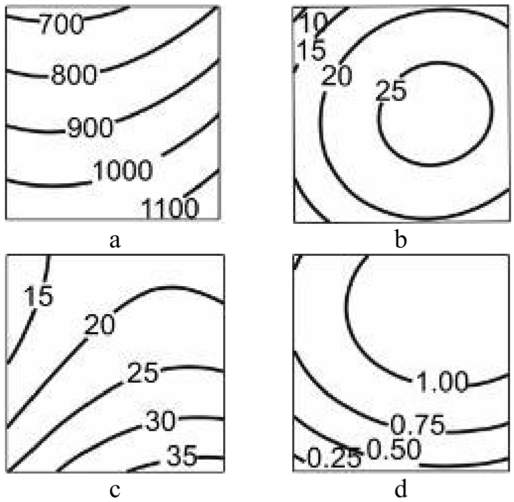

Figure 7 The average density of (a), flexural strength (b), the compressive strength (c), the crack resistance coefficient (d) the composition №2

The obtained plaster solutions meet the requirements of normative documents for medium density (600-1600 kg/m^3), flexural strength (10-25 kg/cm^2) and compression (25-50 kg / cm2). The plaster coating has a high crack resistance, the crack resistance coefficient is 0.25-1, while the cracking resistance is considered plaster with an index> 0.26.

Conclusions

The increase in the share of high-rise construction and the widespread use of new wall materials, requires the development of scientific foundations, the design of plaster mortar compositions. For this analysis processes occurring in the plasters in its application and hardening, the system is considered failure mechanism "bricklaying - plaster coating" are given formula for calculating stress. The principles and criteria are formulated under which the plaster coating and the wall structure durability is provided, the constituents and the material composition selected.

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Sealing materials effectiveness evaluation in the repair of bituminous-concrete surface with transversal cracks

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The method of sealing materials effectiveness evaluation in the repair of bituminous-concrete surface with transversal cracks is offered. Calculation schemes and mathematical models based on the application of the thermo-viscous-elasticity theory for the prediction of temperature stresses in the sealing material at the contact with the bituminous-concrete surface, taking into account the thermal-technical and thermorheological properties of the materials and the geometric parameters of the bituminous-concrete slabs and the transversal crack. The obtained analytical dependences allow to estimate the service life before the loss of sealant compound integrity or adhesive strength of their adhesion with bituminous-concrete surface taking into account annual and daily temperature fluctuations according to the main provisions of the kinetic theory of solid bodies strength.

Keywords: bituminous-concrete surface, temperature transversal cracks, sealing materials, thermo-stressed state

Оцінювання ефективності герметизуючих матеріалів при ремонті асфальтобетонного покриття з поперечними тріщинами

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Проведено аналіз негативних наслідків від утворенням температурних поперечних тріщин у асфальтобетонному покритті для конструкції дорожнього одягу. Показано, що поперечні тріщини призводять до потрапляння води з атмосферних опадів у конструкцію дорожнього одягу та ґрунт земляного полотна, викликаючи ослаблення в зоні тріщин основи і спричиняючи прискореному руйнуванню всієї конструкції дорожнього одягу. ці обставини потребують забезпечення ефективної герметизації покриття з поперечними тріщинами. Відмічено, що найбільш поширений метод ремонту асфальтобетонного покриття з поперечними температурними тріщинами полягає у застосовуванні герметизуючих матеріалів, якими заповнюють порожнини цих тріщин. Зазначено, що умови роботи герметизуючих матеріалів залежать як від геометричних параметрів асфальтобетонних плит між тріщинами, так і від розмірів тріщин, що заповнюються герметиком. Запропоновано метод оцінювання ефективності герметизуючих матеріалів при ремонті асфальтобетонного покриття конструкції дорожнього одягу з поперечними тріщинами. Розроблено розрахункові схеми роботи асфальтобетонного покриття та математичні моделі на основі застосування теорії термо-в’язко пружності для прогнозування температурних напружень в герметизуючому матеріалі на контакті з асфальтобетонним покриттям з урахуванням тепло-технічних та термо-реологічних властивостей матеріалів і геометричних параметрів асфальтобетонних плит та поперечної тріщини. Отримано аналітичні залежності дозволяють оцінювати термін служби до втрати суцільності герметиків або адгезійної міцності їх зчеплення з асфальтобетонним покриттям з урахуванням річних та добових коливань температури на базі використання основних положень кінетичної теорії міцності твердих тіл. Використання запропонованого методу потребує експериментального встановлення таких характеристик матеріалів як: коефіцієнта лінійного температурного розширення, коефіцієнта температуропровідності, функції повзучості, функції релаксації, функції температурно-часової аналогії, функції довготривалої міцності.

Ключові слова: асфальтобетонне покриття, температурна поперечна тріщина, герметизуючий матеріал, термонапружений стан

Introduction

Bituminous concrete is the most common surface material on both public and communal streets and roads with non-rigid pavement. This has several advantages: component availability, manufacturing and application adaptability, a wide variety of modifications, directional properties control, high weather resistance, strength and durability, etc. However, despite the large number of studies and practical measures developed to prevent and avoid transversal temperature cracks, these types of destruction still remain one of the most common defects in bituminous-concrete surface. These transversal cracks of thermal-shrinkage origin are formed with a certain step over the length as a result of both seasonal and daily temperature fluctuations. During operation, new transversal temperature cracks are formed and the distance between them decreases from tens to several meters. Their appearance has many negative consequences. In the zone of cracks, as a result of a significant decrease in the bearing capacity of the surface layers, there is an overstress of the base layers and soil of the earth bed. Water enters them through cracks and significantly reduces their strength (including through salinity at the ingress of ice-melting substances). This leads to the acceleration of the appearance of destruction various types under the influence of traffic loads and climatic factors as the bituminous-concrete surface itself and road pavement as a whole. Therefore, in the zone of cracks spalls, chipping, dimples, potholes, subsidence, cracking, wheel tracking are often formed. In this regard, the appearance of transverse temperature cracks leads to early failure of road pavement and reduces its working life.

Definition of unsolved aspects of the problem

In the current practice, to ensure the water impermeability of the bituminous-concrete surface and to reduce its negative effects on the strength and durability of the entire structure of road pavement, they realize sealing of temperature transversal cracks with the help of various sealants. Therefore, in this case, it is necessary to be able to evaluate the effectiveness of sealing materials in the repair of bituminous-concrete with transversal cracks. Their work effectiveness will depend on the adhesive strength at the contact "sealant – bituminous – concrete surface" and ensuring the sealing material integrity during the operation. The most influential factor in this situation is the sealing material deformation when the temperature decreases as it changes during seasonal and daily fluctuations.

Basic material and results

As it was previously shown in many studies [1], transversal temperature cracks on bituminous-concrete surface of non-rigid road pavement appear as a result of the influence of the following factors: tensile thermal stresses at temperature decreases as a result of casual pavement reduction in the longitudinal direction due to the friction force between the surface and the base. In this case, the design diagram (Fig. 1) presents a single-layer slab resting on the base which does not transmit its sensible deformations.

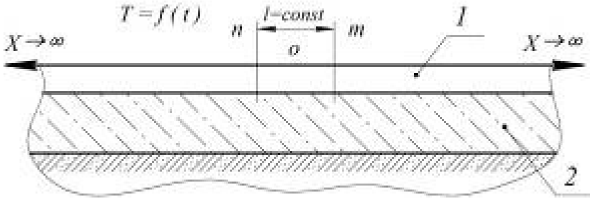

Figure 1 Design diagram of operation of bituminous-concrete surface at temperature change on the base, which does not cause additional horizontal normal stresses in the surface:
1 – surface; 2 – base.

In the first approximation in the elastic formulation of a one-dimensional problem for this design scheme, the thermal tensile stresses when it is impossible to freely reduce the longitudinal direction surface can be determined by the formula

$$\sigma_T(t) = \alpha\, E\, \Delta T, \qquad (1)$$

where α is the linear temperature expansion coefficient of bituminous concrete;
E – bituminous-concrete elasticity modulus;
ΔT – temperature gradient.

It is known that after a certain period of operation (1-5 years), transversal temperature cracks are formed on the bituminous – concrete surface due to the repeated influence of thermal stresses caused by the above-mentioned factors, as a result of temperature reductions during its fluctuations. After the formation of transversal temperature cracks, the operating conditions of the bituminous- concrete surface change. Depending on the distance of the transversal crack, the formed slabs of the pavement may be partially reduced, overcoming the friction resistance of the coating on the base, and the width of the cracks may increase with decreasing temperature. Therefore, the design diagram after the formation of transversal temperature cracks can be presented in the form shown in Figure 2.

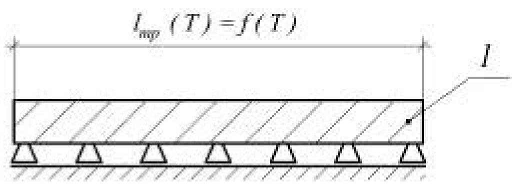

Figure 2 – Design diagram of bituminous-concrete surface operation after the formation of temperature transversal cracks

In order to seal the transversal temperature cracks in the bituminous- concrete surface, it is necessary to make a choice of material and technology, depending on the geometrical parameters of the surface and the crack and the properties of the bituminous concrete and sealing materials. To evaluate their effectiveness, it is necessary to consider the design diagram of the work of these materials in the surface with transversal cracks (Figure 3).

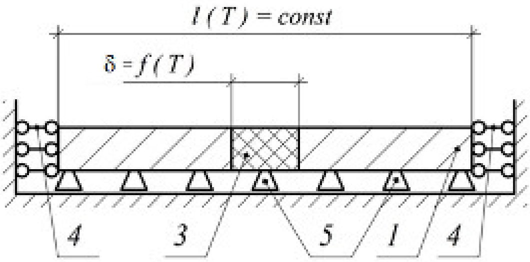

Figure 3 – Design diagram of bituminous concrete operation after sealing the transversal temperature crack:
1 – bituminous-concrete surface; 2 – base;
3 – material for repair of cracks;
4 – communication scheme with surface adjacent areas;
5 – the same with the underlying layers of the pavement structure, which ensures horizontal movement through friction

At the time of the transversal cracks sealing, their width is δ in bituminous concrete surface 1 between blocks of initial length l_{bl}. Then lowering the temperature will reduce the pavement units to the size l'_{bl}. With full adhesion of the material 3 with surface 1, it will stretch to width δ'. In this case, the thermal tensile stresses σ_T in the sealing material, as well as on the contact with the bituminous – concrete surface, will consist of two components: the intrinsic temperature stresses due to the impossibility to reduce the dimensions σ_{T1} and the stresses due to the reduction of adjacent surface units σ_{T2}:

$$\sigma_{T1} = \alpha_1 E_1 \Delta T ; \qquad \sigma_{T2} = E_1 \cdot \frac{\Delta\delta(T(t))}{\delta(T_0)} . \tag{2}$$

Then after the corresponding transformations we get

$$\sigma_T = E_1 \cdot \alpha_1 \cdot \Delta T \cdot \left(1 + \frac{\alpha_2}{\alpha_1} \cdot \frac{l_{bl}}{\delta}\right), \tag{3}$$

where α_1, α_2 are the coefficients of linear expansion, respectively, of materials 3 and 2.

The obtained dependence (3) allows to determine the level of thermal stress state of the mastic and adhesive contact, depending on the characteristics of mastic and bituminous concrete, as well as the initial size of the surface units and the width of the transversal cracks at the time of their filling with mastic.

Existing sealing materials and asphalt concrete bituminous – concrete surface are characterized by apparant thermo-viscous-elastic properties, which depend on both temperature and time, and the of stress change behavior, and such materials show kinetic fracture [1, 2].

Based on the fundamental provisions of the thermal elasticity theory, the temperature stresses that occur in the sealing material 3 (Figure 3) can be determined by the dependencies:

$$\sigma_T(t,T) = \int_0^t R\big(\xi(t) - \xi(\tau)\big) d\varepsilon_T(\tau) \ , \tag{4}$$

here

$$\xi(t) = \int_0^t \frac{dt}{a_T(T(t),Q)}$$

$$\xi(\tau) = \int_0^t \frac{d\tau}{a_T(T(t),Q)}$$

where $R\ (t)$ is the relaxation function of the sealing material;
ε_T – relative thermal strain;
t is the time at which the stress is determined;
τ is the instant of time preceding t;
ξ is the time given on the basis of the principle of temperature-time analogy (PTA) to the temperature Q at which the parameters of the relaxation function $R\ (t)$ are experimentally determined;
a_T - the PTA function.

The relaxation function of a sealing material can be written in the form of a modified power law [1, 3]:

$$R(t) = H + (B - H)(1 + t/r)^{-m} , \tag{5}$$

where m and r are constants determined from the experiment on relaxation;
H and B are long-term and instantaneous elasticity modules, respectively.

The PTA function can be described by an expression

$$a_T(T,Q) = e^{-p(T-Q)} , \tag{6}$$

where p is a constant, determined experimentally.

Based on the design diagram (Figure 3), the values for $\varepsilon_T(t)$ can be written as

$$\varepsilon_T = \alpha_1 \cdot (T_0 - T(t)) \cdot \left(1 + \frac{\alpha_2}{\alpha_2} \cdot \frac{l_{бл}}{\delta}\right), \tag{7}$$

where $T_0 - T(t) = \Delta T$.

Taking into account the harmonic change of temperature $T(t)$ according to [1], the annual and daily fluctuations of the average surface thickness, temperature can be written in the following form

$$T(t) = T_{cp} + \overline{A}_c \cos\frac{2\pi}{t_c} + \overline{A}_r \cos\frac{2\pi}{t_r} t \ , \tag{8}$$

where T_{cp} is the surface average annual thickness;
$\overline{A}_c$, $\overline{A}_r$ and t_c, t_r are respectively the amplitude and the fluctuation period of the surface temperature averaged thickness in the daily and annual cycles.

The amplitude of the fluctuations averaged over the thickness of the surface temperature ($\overline{A}_c$ or $\overline{A}_r$) can be considered approximately equal to the average thickness of the amplitude of the temperature fluctuations at different depths

$$\overline{A} \approx \frac{1}{h}\int_0^h A(z)dz = \frac{A(z=0)}{h\sqrt{\pi / t_n a}}(1-e^{-h\sqrt{\pi / t_n a}}), \quad (9)$$

where a is the coefficient of thermal conductivity of the surface material;

t_n is the period of temperature fluctuation.

On the basis of the above-mentioned expressions, it is possible to predict temperature stresses at any time t. However, since the strength characteristics also depend on the temperature and the time of the load, showing the kinetic nature of the cracks, it is necessary to determine the durability of the sealing materials and the condition of the limit state. Having the solution to determine the temperature stresses in the surface, the strength conditions and the expression for the durability function, it is possible to calculate the thermal crack strength index M_{mp} by the end of the service life $t_{сл}$

$$M_{mp} = \int_0^{t_{сл}} \frac{\sigma_T(t)^{b_\tau(t,T)}}{B_\tau(t,T)}dt. \quad (10)$$

Taking into account the expression (10), it is possible to check the fulfillment of the condition in which it is required that the crack strength M_{mp} does not exceed its admissible value, that is, to evaluate the condition for providing thermal crack strength C_{mp}

$$M_{mp} \leq C_{mp}. \quad (11)$$

In this case, based on the known results of the studies C_{mp} equals the limit value of the damage degree to the C_H, i.e. $C_{mp} = C_H = 1$ [4-6].

Conclusions

The results of the research allow us to draw the following conclusions.

1. Design diagrams and mathematical models based on the application of the thermo-viscous-elasticity theory for the prediction of thermal stresses in the sealing material at the contact with the bituminous – concrete surface, taking into account the properties of the materials and geometrical parameters of the bituminous – concrete surface slabs and transversal crack are worked out.

2. The obtained analytical dependences allow to estimate the service life before the loss of sealant integrity or the adhesive strength of their adhesion with the bituminous – concrete surface, taking into account the annual and daily temperature fluctuations on the basis of the kinetic theory main provisions of the strength of solids.

3. The method of efficiency of sealing materials evaluation in repair of bituminous – concrete surface with transversal cracks taking into account thickness and sizes of bituminous – concrete slabs between transversal temperature cracks, sizes of these cracks, thermotechnical and thermorheological properties of sealing materials and bituminous – concrete is offered.

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Research of the current Egyptian roads service facilities placement condition

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The state and provision level of road traffic participants to the objects of service on Egypt highways have been analysed. In general, there are a number of significant shortcomings regarding the systematic approach in justifying and standardizing the distances of the road services location, considering the requirements of road users. With the help of Google Earth and real survey, the current status of the service objects along the Egypt highways and in foreign countries of the world has been surveyed. The obtained data show that service facilities are unevenly located in Egypt, often do not meet normative requirements and are not characterized by complexity. Instead, in foreign countries there is even more distribution of complex service facilities along highways, all service facilities has transition-high-speed bands and the vast majority have separate congresses.

Keywords: highway, service objects, patrol station, functional need, spatial corridor

Дослідження існуючого стану розміщення об’єктів сервісу доріг Єгипту

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За даними Всесвітньої організації охорони здоров'я щорічно майже 1,24 мільйона людей гине в результаті дорожньо-транспортних подій на автомобільних дорогах, а 50 мільйонів – отримують серйозні травми. Майже 90% смертельних випадків трапляються на автомобільних дорогах слаборозвиненних країн, зокрема Єгипту. Враховуючи ці обставини, в роботі проведено аналіз стану та рівня забезпечення учасників дорожнього руху об'єктами сервісу на дорогах Єгипту. Виокремлено ряд істотних недоліків щодо системного підходу в обгрунтуванні та стандартизації відстаней розташування дорожніх послуг з урахуванням вимог користувачів доріг. За допомогою програми Google Earth та натурних вишукувань обстежено існуючий стан розміщення об'єктів сервісу вздовж автомобільних доріг Єгипту та інших країнах світу, зокрема Австралії. В результаті натурного обстеження виявлено наступні недоліки розташування об’єктів сервісу на дорогах Єгипту: погане облаштування, низька якість утримання, відсутність туалетів, використання місць відпочинку як складських територій, відсутність місць для стоянки вантажівок, заборонено в’їзд на багато зон для відпочинку, недостатній розмір місць відпочинку (недостатньо пропускної спроможності для необхідної кількості вантажних автомобілів). Фактичний стан об'єктів сервісу в Єгипті характеризується невідповідністю нормативним вимогам, відсутністю комплексого підходу до структури та нерівномірністю розташування (відстань між об’єктами змінюється від 5 до 25 км, середнє значення – 7 км). Натомість стан обєктів сервісу Австралії характеризується більш комплексною структурою та рівномірним розташуванням вздовж автомобільних доріг (середня відстань між об’єктами сервісу – 24 м). У результаті проведеного аналізу визначено рекомендації щодо поліпшення транспортно-експлуатаційного стану автомобільних доріг та обєктів сервісу Єгипту, що має позитивно вплинути на рівень аварійності на автомобільних дорогах.

Ключові слова: автомагістраль, об’єкт сервісу, автозаправна станція, функціональна потреба, просторовий коридор.

Introduction

Safety and security are of primary concern for any transport system. Fatalities and injuries caused by crashes are an increasing international health epidemic. Nearly 1.24 million people perish yearly in road crashes and an estimated 50 million are seriously injured. It is the highest cause of fatalities the young. Road crashes costs around 1-3% of the world GDP. Ninety percent of fatalities and injuries caused by road crashes happen in developing countries. Although the percentage of fatalities are estimated to drop in developed countries due to excellent health care conditions and modern technology advancements, it is unfortunately predicted to rise by 80% in the rest of the world [1]. Almost ninety-percent of the fatalities caused by traffic crashes happened in poor developing countries, even though it possesses half of the world number of vehicles [1]. Half of crash fatalities happen among road users such as pedestrians and bicyclists and heavy vehicles, they are more likely than drivers to be harmed in a road crash.

Injuries due to road crashes cause large economic losses to victims, their families and to national economy as well. These losses come from the cost of treatment and rehabilitation in addition to lost productivity for those killed or disabled, and for family members who need to take to care for the victims. The road network efficiency is vital to the transport system of Egypt as the country invests in developing major economic growth centers. Almost 93 percent of passengers in Egypt use the road network whereas rail and air passengers' percentage are 6.6% and 0.4% respectively [1].

The HSM provided transportation professionals with a quantitative evaluation and analysis of road safety. It offers tools and methods to estimate crash frequency and to economically evaluate proposed solutions to reduce crash frequency and severity. It was published by AASHTO in 2010 summing up years of research and experience in modelling crash data in the transportation field. The HSM used SPFs developed in the United States using road and crash data specific to the environment in the United States. Therefore, it was encouraged that the SPFs be calibrated or developed locally to suit the local characteristics of roads and crashes.

The challenge faced by researcher was the large amount of required data such as roadway geometric characteristics, recorded traffic volumes, and multiple years of recorded crashes. It could be very difficult in most developing countries, since the availability and quality of data in developing countries such as Egypt is questionable.

Egypt is a middle-income country, belonging to the Eastern Mediterranean Region (EMR). It has a population of almost 99,375,741 (2018) with the GDP per capita of USD 12,100. Egypt used to be the worst performing EMR country, with just over 45 fatalities per 100,000 populations (according to WHO modelled data.

Though the number of reported road traffic fatalities decreased to 16,800 in 2011 from about 24,400 in 2010, it is deemed necessary to take more action and additional measures to face these road safety challenges in order to build a safer road infrastructure [2].

Even though the road infrastructure in Egypt is designed using the Egyptian Code of Roads [3] which is mainly based on the AASHTO (2011) [4], there are still many safety issues that could be tackled through using a systematic approach. This procedure could be enhanced through initially quantifying the crashes on Egyptian roads, to have better understanding of the deficiencies and means of improvement Therefore, this research is constructing and it improves more rest stations along the roads to let vehicle drivers rest during long journeys.

Road economy is the main infrastructure element of any country's development. World experience proves that it was precisely from the development of highways that the economic crisis of many countries began to emerge. Along with the development of the road complex, road service facilities, which are an integral part of the roads, should be developed. Road service facilities are important for drivers. The rest area is an important element, because drivers constantly need rest. Driving a car requires energy and effort, and leads to physical and emotional fatigue. Therefore, it is important that along the roads there are complexes where you could have a rest.

Service may be essential even if there is a fairly low demand for it. For example, if only one in ten thousand motorists passing a rest area is so sleepy that he/she needs to stop, providing a safe place to take a nap for that person may save his/her life as well as other people's lives. On the other hand, a rest area may attract hundreds or thousands of motorists every day because it is a convenient place to buy snacks or obtain tourist information. But if the rest area were to be closed, existing fast food restaurants and convenience stores at the next exit may easily be able to provide the same services, if information is provided to the motorist. There may certainly be exceptions to this in areas that are rural. In other words, it is not only quantifying rest area usage that should form the basis of whether to keep a rest area or not and which services to provide there with. Also, the necessity of these services and available alternatives should be considered.

Filling stations are the most important service objects, because the car needs regular fueling.

Service stations are often "rescue islands" for drivers, because the car is a mechanism that has a function of wear. Replacement parts, repairs, car wash are also often needed by drivers and their car.

To focus on the high development of road service, the experience of foreign countries should be considered. In these countries there is an integrated approach to the development of road infrastructure.

Review of research sources and publications

Thomas Kweku Taylor, Chanda Sichinsambwe and Blessings Chansa from Kitwe Zambia research location of manufacturing and service activities in urban areas are guided by planning principles and standards, expressed in either structure plans or land use development plans. The paper is an exploratory study that applied a cross-sectional descriptive research design to find answers to the research questions and to validate the following hypotheses: Environmental Impact Assessment (EIA) Criteria is positively related to the location of filling stations in Kitwe; Entrepreneurs preferential location choices is positively related to location of filling stations; and Planning principles, standards and regulations positively influence locations of filling stations in the City of Kitwe. The t-test statistics was used to validate the hypotheses. The main finding was that, filling stations are located influenced by choices made by service station entrepreneurs [5].

David W. Fowler, Joseph F. Malina, Jr., Kirby W. Perry, Gary C. Vliet have researched design recommendations for rest areas. Based on the findings presented in reports, recommended design procedures are presented. Recommendations include spacing, site requirements, example architectural designs, materials, mechanical systems, and operations and maintenance. Recommendations for energy sources, water systems, and wastewater systems are made [6].

Zapolsky Yu. studied the buildings system formation and structures in the highways landscapes. His works devoted to determining the size of these objects [7]. Famous Russian researchers like Babkov V., Ornatsky N., Treskinsky S. devoted their works to the Russian highways improvement. Belarussian architect Sardarov A. studied the architecture of Belarus road environment [8].

Definition of unsolved aspects of the problem

Fatigue is a significant issue for drivers in Egypt and particularly for those travelling long distances in highway. It is estimated that fatigue is the main contributing factor in approximately 25% of road crashes involving serious injury.

Commercial vehicle operators in Egypt drive for long distances and often through periods when they should naturally be sleeping. Transport regulations require operators to have short rest breaks of 30 minutes duration (or two × 15 minutes) for every six hours of driving. Operators are also required to stop for a minimum sleep break of six hours under the regulations. Recent research has highlighted the benefits of taking short naps, especially in relation to the re-charging function of a power nap. The ability to utilize this and other rest time to the benefit of driver safety is therefore paramount. Roadside rest areas for commercial vehicle operators should provide for the needs of these drivers [9].

There appears to be a lack of consistency in the frequency of rest areas provided and the manner in which they are established. It is apparent that there is the need to promulgate nationally consistent guidelines for the provision of rest areas for heavy vehicle drivers. Further, it is suggested that an audit of rest area locations on long distance highways is warranted in the short term, especially where such audits have not been previously carried out. The results from these audits together with future freight movement forecasts assist in planning for future rest area demand.

The provision of rest opportunities through rest areas represents a management tool in addressing fatigue-related crashes. A major challenge to realize the potential benefits of rest areas is to increase patronage by those road users travelling long distances.

The increased use of rest areas depends largely on the quality of the rest provided area. The ability to maintain and provide a clean and attractive rest facility considers a major factor in attracting and retaining their use by road users.

Historically rest areas in Egypt have been subject to a great deal of criticism by road users and representative groups. Common issues identified through consultation with regard to rest areas in Egypt include (fig.1):

– Poor quality.
– Poorly maintained.
– Lack of toilets.
– Use of rest areas as sites for stacking aggregate.
– Lack of 'truck friendly' rest areas.
– Many rest areas are truck prohibited.
– Inappropriate size of rest areas – not enough capacity for the required number of trucks.

Figure 1 – ATM Service on highway in Egypt

Problem statement

In this research there are attempts to address these issues: a key component in the development has been the level of understanding gained through consultation of the use of rest areas by the different user groups. This research attempts to consider the key requirements of these different user groups, with the common aim of providing and encouraging greater use of rest areas for fatigue management. Additionally, recognition of the importance of a comprehensive well planned and managed network of rest opportunities has also been achieved. This aspect should ensure that the provision of rest areas and the facilities in them are strategically assessed against user demand.

Purpose of the research is improving the parameters of service objects locations along highways of Egypt.

Objectives of the research:

- Analysis of existing research on the problem service objects placed along highway.
- Survey of the current state of the service facilities placement along Egyptian roads and comparison with foreign experience.

Basic material and results

The objects of the service include: rest area, filling stations (gas stations) and gas filling stations, service stations, trading posts Fig.2.

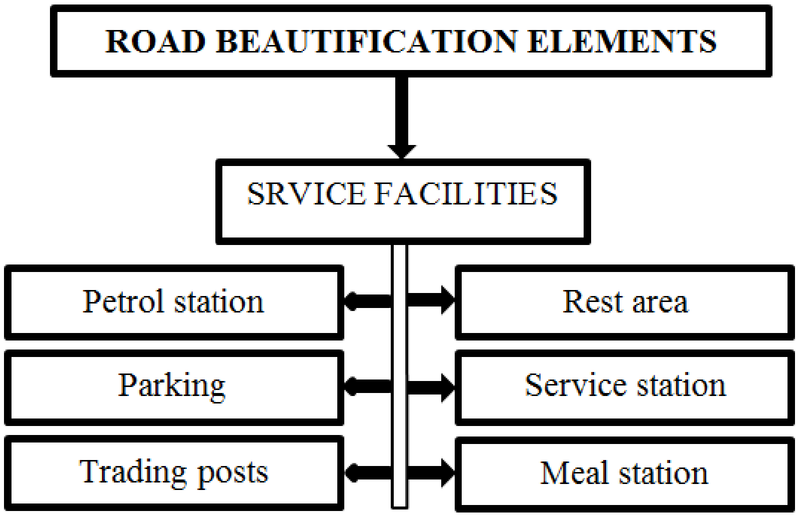

Figure 2 – Classification of highways service facilities

Description of the research methodology.

In the final form, a measurable object was conducted using Google Earth (Google Earth). This is a free, downloadable, Google-powered program that displays a virtual globe. The principle is based on aerial photographs and satellite images of most of the Earth. For some regions, these photos are of very high quality.

The order of the research:

1. To use the "Line" function to select "Line", measure-ment in meters. Thus, from a point to point distance from the edge of the travel section to the beginning of the service object, the width and length of the territory were measured.

In the final form the metered object shown on the fig. 3 – 4.

2. Also the distance to the next service object along the road in the same direction was measured. To conduct it, in the "Ruler" function, the tab "Path" in kilometers was chosen

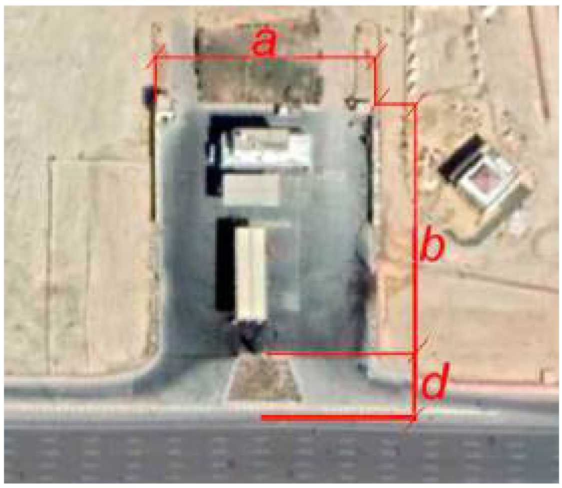

Figure 3 – An example of a measured petrol station in Google Earth

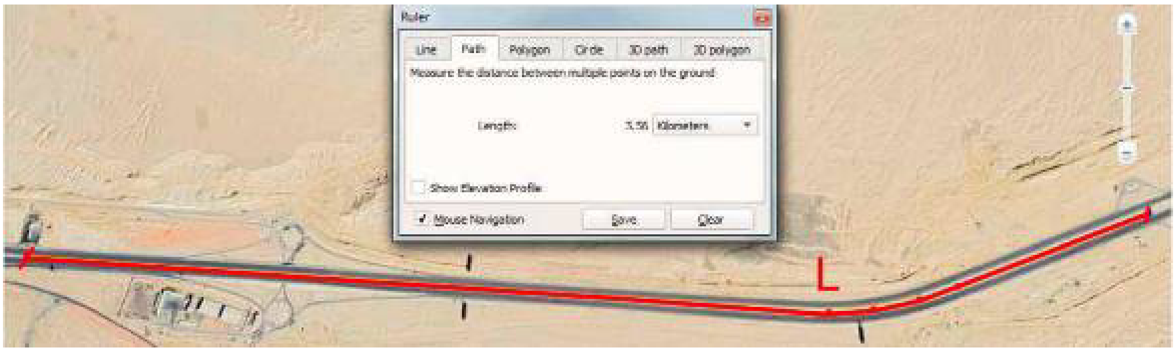

Figure 4 – An example of measuring the distance between a petrol station in the Google Earth program

Survey of the service objects placement in Egypt. The study of service objects in Egypt was carried out according to ministry of petroleum data. As of June 30, 2014, the number of registered gas stations in Egypt is 2902 petrol station [1]. In addition, unfortunately, in Egypt there are a very small number of service facilities that should include gas stations, parking lots, hotel, car-care centers, food items, which are very typical for foreign countries. In the regions of Egypt, gas stations are often located separately from service stations.

In Egypt, 52 stations were measured on the routes Cairo Alex-Cairo Desert Road – Wadi El natron Elalmin – Al Ain el Sokhna – Cairo (Fig.5).

According to the obtained data on the location of petrol stations in Egypt and the obtained data on the location of service facilities in Australia, the data from the edge of the roadway to the service object and the distance between the service objects along the route are compared.

Fig 6. It is shown the distance between station and from the edge of the roadway in Alex-Cairo Desert Road.

Fig 7. It is shown the distance between station and from the edge of the roadway in Wadi El natron Elalmin.

Fig 8. It is shown the distance between station and from the edge of the roadway in Al Ain el Sokhna – Cairo.

Table 1. It is shown the averaged data for 52 petrol sta-tions in Egypt.

Table 1 – Averaged data for 52 petrol stations in Egypt

Distance from the roadway edge (d, m)	Distance between stations (L, km)
24.71	7.1

The next step of research was the survey of the service objects placement along highways in foreign country.

Egypt

a b

Figure 5 – Planning solution of service station
a – Green Desert along Cairo Alex-Cairo Desert Road;
b – rest area along Wadi El natron Elalmin

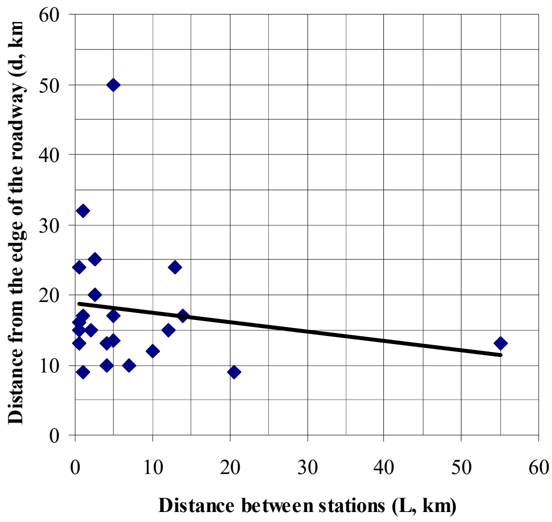

Figure 6 – Determine the distance between station and from the edge of the roadway

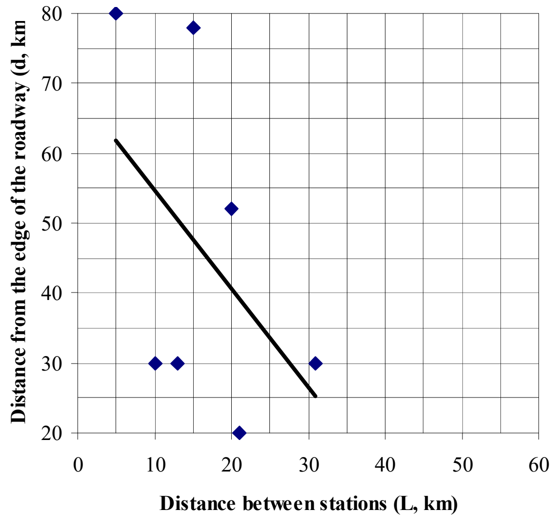

Figure 7 – It is determined the distance between stations and from the roadway edge

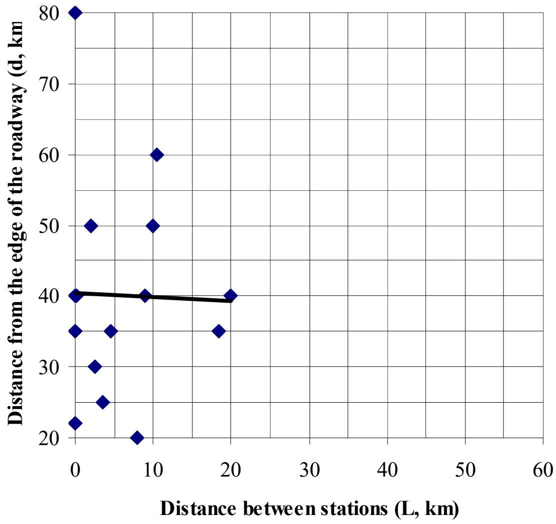

Figure 8 – It is determined the distance between station and from the edge of the roadway

Australia

Fig 9. It is shown the distance between station and from the edge of the roadway in Hume Highway.

Fig 10. It is shown the distance between station and from the edge of the roadway in Pacific Highway.

Table 2. It is shown the averaged data for 56 petrol stations in Australia

Table 2 – Averaged data for 56 petrol stations in Australia

Distance from the roadway edge (d, m)	Distance between stations (L, km)
38.56	23.91

Table 3. There are average values for the 5 roads studied to compare all the obtained parameters.

Table 3 – Average values for the 5 roads studied to compare all the obtained parameters

N	Name of Road	Distance		
		d/n	S/n	L/n
1.1	Alex-Cairo Desert Road	24	12302.	5.3
1.2	Wadi El natron Elalmin	41.5	6618	14.5
1.3	Al Ain el Sokhna – Cairo	41.5	6740.9	5.5
2.1	Hume Highway	42.9	3220.4	24.65
2.2	Pacific Highway	33.24	3634.4	23

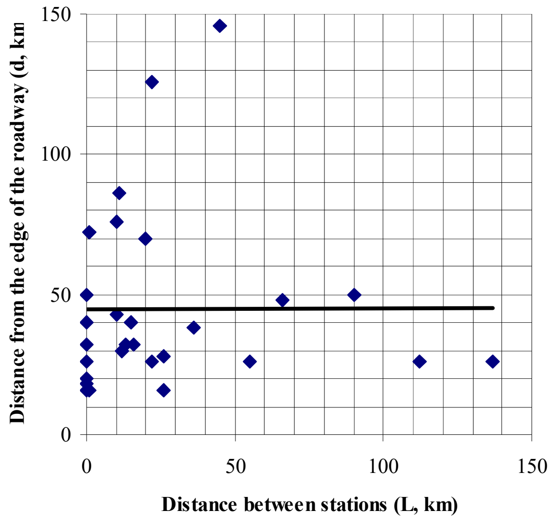

Figure 9 – It is determined the distance between stations and from the roadway edge

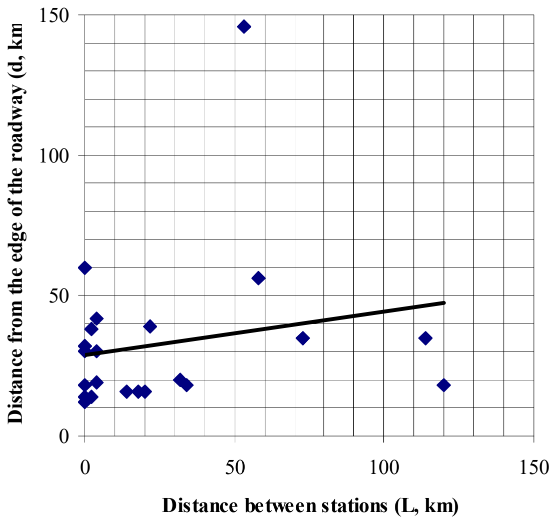

Figure 10 – It is determined the distance between stations and from the roadway edge

Figure 11 – Diagram for all roads for distance from the roadway edge (d, m)

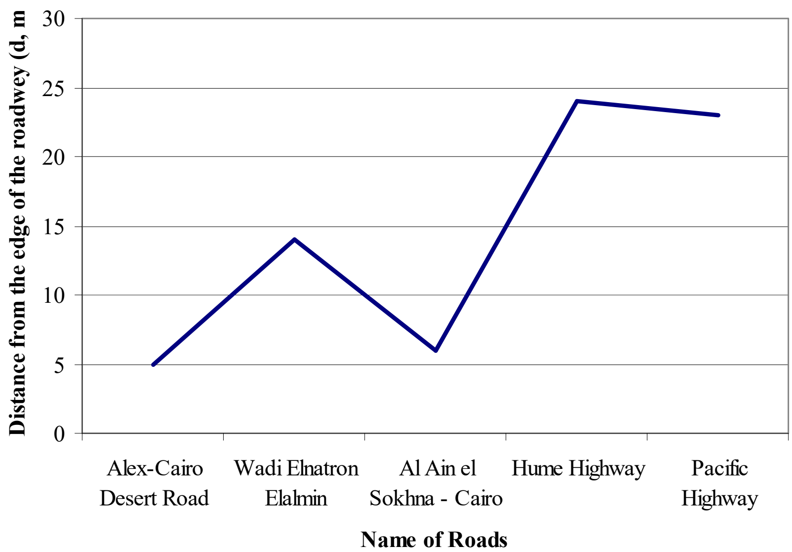

Figure 12 – Diagram for all roads for Distance between stations (L, Km)

Conclusions

The main tasks for improvement in Egypt include:

1) to develop new rest areas in locations where current spacing does not meet research requirements.

2) to improve safety of entry/egress through sealed shoulders, aprons, and acceleration/deceleration lanes if required.

3) to install appropriate facilities (including bins, lighting etc.) to create functionally attractive spaces for road users to utilize.

4) to install toilets in high demand rest are-as.

5) to resurface poor pavements (sealed & un-sealed as necessary) to create safe trafficable and hard stand areas.

6) to fence the rest areas to create clearly defined boundaries there.

7) to improve distribution of rest area information (maps, electronic data, on-route signage etc).

8) the distance from the edge in Egypt should be at least 15 m. In foreign countries, average values indicate that similar requirements are met. The greater the distance from the edge to the object service, the less probability of an accident.

9) the distance to the next service object in Egypt should be from 15 km to 40 km. These data are very volatile in Egypt. In foreign countries this indicator is more moderate.

10) the area of service objects in the investigated foreign countries is many times higher than Egyptian. They include: hotels, car parks, petrol stations. There were no additional services at the some petrol stations explored in Egypt.

11) transition speed bands are not present at all petrol stations in Egypt. Unlike Egypt in all investigated service facilities in foreign countries there are transitional high speed bands.

The placement of service objects in Egypt and abroad should be done with the help of Google Earth.

The difference between the placement of service objects in Egypt and foreign countries was revealed. The distance to the roadway edge in Egypt often does not meet regulatory requirements, as opposed to foreign countries, where there is a consistency of these indicators in countries. The distance between the objects of service in Egypt varies from 0, 25 km to 60 km (according to regulatory requirements it should be from 15 km to 30 km), in foreign countries this indicator is even more. The area of service objects in foreign countries is many times higher than in Egypt.

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Trends and approaches to reorganization of urban environment

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The article describes the standards of urban environment reorganization, which are oriented to the effective use of land resources and the load distribution for urban transport infrastructure, a high degree of functional diversity. The authors have analyzed the approaches and trends of the urban environment transformation, based on the concept of the compact cities formation. The authors determined specific features of the urban industrial environment transformation different types depending on the degree and direction of the objects modification. In the conditions of land plots shortage, redevelopment projects provide an opportunity to renew the urban environment, to solve transport problems, and to find the potential for building new objects.

Keywords: reorganization, redevelopment, reconstruction, revitalization, renovation

Тенденції та підходи до реорганізації міського середовища

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Розглянуто стандарти реорганізації міського середовища, орієнтовані на ефективне використання земельних ресурсів і розподіл навантаження на міську транспортну інфраструктуру, високий ступінь функціонального розмаїття. Проаналізовано підходи і тенденції перетворення міського середовища, виходячи з концепції формування компактних міст та стандартів комплексного розвитку територій. Промислові підприємства у великих містах припиняють свою діяльність через економічні причини або виводяться за межі міста через порушення екологічних норм. Частина таких підприємств приходить в занепад, при цьому здійснюючи негативний вплив на зовнішній вигляд міста. Через потребу в суттєвих капітальних вкладеннях власники або місцева влада не мають змоги здійснювати перетворення таких промислових об’єктів. Визначено специфічні особливості різних видів перетворення міських промислових зон, залежно від ступеня та напряму видозміни об’єктів, а саме: реконструкції, реновації, джентрифікації, браунфілду, ревіталізації, редевелопменту. Редевелопмент дозволяє використати ті очевидні переваги, які в сучасному місті надає концентрація і збільшення щільності міського середовища. В сучасних щільно забудованих містах редевелопмент може стати майже єдино можливим способом перетворення. Редевелопмент передбачає більш масштабні зміни об’єкту (зміна функції промислової території, історичних будівель і транспортної інфраструктури) та суттєві інвестиції. В результаті не запитувані і нераціонально використовувані території перетворюються на нові інвестиційно привабливі проекти. В умовах дефіциту земельних ділянок проекти редевелопменту дають можливість оновити міське середовище, вирішити транспортні проблеми, знайти потенціал для будівництва нових об’єктів.

Ключові слова: реорганізація, редевелопмент, реконструкція, ревіталізація, реновація

Introduction

One of the problems of the modern cities development is the deficit of free territorial resources. This leads to the need to increase territorial reserves by using territories previously used for other purposes. For example, the territory of industrial and warehouse enterprises can be used as such reserves [1, 2, 12, 13].

Since the middle of the 20th century, in connection with the development of automation and high technologies, sharp decline in production in large industrial centers around the world can be observed. This process reached its peak in the 70-80s of the last century.

In recent years, Ukraine, experiencing a period of deindustrialization of large cities, has faced a major challenge in the redevelopment of industrial zones and facilities. Industrial enterprises in large cities cease their activities for economic reasons or remove their activities from the city because of environmental standards violation. Some of these enterprises are slowly falling apart and this negatively affects the external appearance of the city. Because of the large amount of required capital investments, owners or municipal authorities cannot begin transformation of industrial facilities into active urban areas.

Review of research sources and publications

According to [3, 7, 14 – 17, 19], the consistency of the urban environment with the characteristics of a compact city is carried out by using the system of standards for integrated development of territories.

The standards are oriented to the effective use of land resources and the load distribution for urban transport and engineering infrastructure, a high degree of functional and social diversity, a wide choice of residential development types considering the needs of different groups of users:

– security: security in open spaces at any time of the day, preventing offenses and emergencies;

– environmental friendliness: air quality, quality of gardening, energy efficiency, waste utilization;

– comfort: mobility, balance of publicity and privacy, visual comfort, acoustic comfort, climatic comfort;

– identity and diversity: recognizability of the image, preservation of historical elements of the urban environment, consideration of established practices for the use of buildings and territories, the diversity of types of spaces, the diversity of functions in residential areas, cultural diversity.

Definition of unsolved aspects of the problem

The reorganization of industrial areas is one of the most urgent urban problems in the post-industrial society. For example, in Kiev they occupy about 7 thousand hectares [2]. Historically, industrial enterprises which were created on the wave of industrialization in the first half of the 20th century on the Kiev outskirts often found itself in the middle city in the postwar period, in the era of mass housing construction, when the capital housing stock increased almost 10 times compared to the pre-war period. Today, at the new stage of the economic development of the society, after extensive structural changes in the production complex and its stagnation, the buildings, structures and territories of industrial enterprises are partially empty or leased; they are used in many cases extensively and extremely irrationally. At the same time, the value of the land they occupy is extremely high. It is extremely irrational to leave these areas without proper attention while there is acute shortage of sites for the construction of residential and public buildings.

After all, with the system approach, using industrial enterprises' territories, it is possible to solve the problem of the residential and public buildings construction, and development of a green plantations system for the nearest decades. It is necessary to consider the opportunities for the large Ukrainian cities development, firstly, at the expense of internal territorial resources, rather than of expanding the boundaries of urban development. It significantly increases the cities compactness; make them more economical and comfortable for living.

Problem statement

That is why the **goal** of this article is to analyze the approaches and the urban environment transformation trends, based on the concept of the formation of compact cities.

Basic material and results

The process of transformation (reorganization) of the urban environment and its elements at the present time can be classified depending on the degree and direction of the objects modification.

With regard to the process of industrial zones transformation, the following types can be distinguished: reconstruction, renovation, gentrification, brownfield, revitalization, redevelopment. Despite the different approaches to environmental conservation, all these types of activities are related to reconstructive processes.

Reconstruction includes the reorganization of the construction object put into operation in accordance with the established procedure, namely: changing its geometric dimensions and / or functional purpose, that leads to a changing the basic technical and economic indicators (quantity of output, capacity, etc.), improving production, raising its technical and economic level and quality of manufactured products, improving operating conditions and quality of services [6, 10 – 11].

Reconstruction provides complete or partial elements preservation of bearing and enclosing structures and a whole stoppage of facility operation or of its parts (provided their autonomy) for work execution period. Reconstruction of buildings is a complicated and time-consuming process, especially if it is an architectural monument or historical buildings. In this case, the age of the structure can be several centuries and the task posed to the designer is complicated tenfold, since it is necessary not only to improve the operational characteristics of the building, to create conditions for its effective use, but also to restore its former shape, to preserve the spirit of the era.

In the reconstruction of industrial facilities, more productive high-mechanized and automated technological processes are introduced, more rational use of production areas is achieved, and the efficiency of using capital investments is increased.

The reconstruction of existing enterprises includes the reorganization of existing workshops and facilities of the main, ancillary and maintenance purposes, as a rule, without the expansion of primary purpose existing buildings. While reconstructing existing enterprises, it is possible to expand separate buildings and structures in those cases when a new high-performance and more advanced equipment can't be located in existing buildings; to build new ones and expand existing workshops and other facilities of the complex in order to eliminate imbalances; to build new buildings and structures of the same purpose instead of ones that are being liquidated on the enterprises territory, which further exploitation, according to technical and economic conditions, was not considered to be necessary.

The reconstruction significantly differs from the new construction and has its own peculiarities in the design, the development of the construction technological process, the specifics of the construction and installation works, which is connected with the variety of structural and space planning decisions, tightness of the construction site, the need for step-by-step execution of work in various areas, a combination of the enterprise production activities with the performance of civil and erection works, disassembling dilapidated structures or their parts in some cases, etc.

There are such major factors that affect the nature of building reconstruction as the characteristics of the city itself, the place of development in the city planning structure, the quality of the microdistrict, quarter, buildings.

The considerable amount of work during the reconstruction relates to the disassembly and destruction of the buildings structures. They are very labor intensive and largely determine the terms of reconstruction [11].

Renovation is a process of restoring the structure and type of damaged historical town-planning structures, facades and interiors of morally and materially obsolete buildings [8].

Gentrification is the reconstruction and renovation of buildings in previously unpublished urban neighborhoods, either under a planned urban rehabilitation program or as a result of decisions taken by professionals and managers. As a result of gentrification, the average level of incomes of the district population increases due to the replacement of low-income residents by the more affluent [18].

Another type of reorganization of urban industrial zones is brownfield, which is the creation of industrial parks on pre-existing production sites (former factories, workshops or port docks). As a rule, in such places there have already been buildings that are being reconstructed, some are being completed, usually there is a suitable infrastructure, communications and ready storage facilities [4].

Revitalization is a term used in scientific and practical activities to describe the processes of reproduction, revitalization and restoration of urban space; it means literally a «revitalization» of a territory or an object that no longer functions [5].

In urbanistics, the term «revitalization» means the restoration of the urban environment, which makes it becomes more habitable.

The principle of revitalization consists in revealing and showing new possibilities of old forms, considering their functions.

In addition, in revitalization, an integrated approach is most often used to preserve the identity, authenticity and historical resources of the urban environment.

The revitalization of existing industrial complexes that are located within the city limits is very popular in the modern world, which is connected with the contradictions between the needs of society and the existing structure of the urban environment.

In such cases, revitalization is seen as a reconstruction of the industrial architecture with a change of its functions. For example, re-equipment of industrial buildings for living space is known as loft. Today, revitalization includes such a semantic tinge as an attentive and careful attitude to the object, preserving the spirit of place and material memory of the past. The degree of change in the urban environment in the process of revitalization depends on the degree of value of historical and cultural objects.

The task of revitalization is the socialization of space, the development of infrastructure elements that regulate tourism and scientific activity, the development of industry, care for the environment and, as a result, the attraction of investments.

Revitalization is a special case of redevelopment.

Redevelopment is one of the most effective ways of re-profiling of unclaimed real estate objects or irrationally used territories [9, 12].

Is is especially acute in large cities, where inefficiently used territories and individual objects are clearly seen.

Zones of cities, formed under the influence of economic and political conditions of other times, provide ample opportunities for redevelopment of: buildings used with underload and suitable for reconstruction, as well as land for the implementation of point redevelopment. Intensive construction of recent years has led to the almost complete filling of free land in the territory of large cities in Ukraine.

The logical extension of the urban real estate transformation is residual development of several land patches and parallel implementation of reconstruction projects of existing buildings or redevelopment of the existing territories of large industrial complexes:

– redevelopment of enterprises' buildings that do not use high technologies, in order to redirect them to use for administrative purposes and research work related to high technology;

– reconstruction of old large enterprises for further use as warehouse and modernized office and technological buildings;

– demolition of heavy industry facilities and subsequent use of land for the construction of residential community or commercial real estate;

– re-profiling of inefficiently used territories with garage-construction cooperatives, small industrial enterprises, snow blowers.

For the redevelopment of degraded industrial areas, it is necessary to do:

– system analysis of the real estate market: general trends in the real estate market, the state of the real estate market segments, the forecast for the development of the real estate market;

– full and comprehensive assessment of the property in next areas: functional property of the real estate object (industrial, communal, warehouse, degraded territories, carpools, etc.), the attractiveness of the territory (economic, urban, landscape, ecological, etc.), degree of negative impact on the environment, analysis of cultural, historical, architectural value of the territory and buildings (monuments, private buildings, unique modern architecture), form of ownership, profitability, degree of reliability and durability of buildings and structures;

– formation of the strategy, concept and project team (the concept includes: new functional use, architectural requirements, consumer requirements, engineering and investment aspects, legal parameters; the formation of a professional team of executors implies: tendering among various specialists, designers, consultants, contractors; tender documents development);

– investment analysis (main assumptions and standards for financial and economic calculations, organizational plan, project cost estimates in accordance with the main components, income calculation, investment development schedule, sources, financing volumes and terms, cash flow statement, forecast balance sheet, profit and loss account, development of various project budget scenarios, including assessment and forecast, a business plan development).

Sometimes redevelopment is identified with reconstruction, but this is not entirely correct, because the first one doesn’t always include the modification of the object's constructions. We can talk about three main types of redevelopment: complete, partial and superficial.

The complete redevelopment assumes absolutely new development of objects and territory: starting from the change of the special purpose of the site and the coordination of the new project and ending with the laying of modern engineering networks and the organization of new transport interchanges. In this case, industrial enterprises transform into trading or shopping and entertainment centers, specialized or grocery supermarkets or business centers, and logistics complexes. It is necessary to conduct marketing research, develop a concept, look for a suitable architectural idea, do financial analysis. We use only the land plot in this variant. A complete redevelopment of industrial facilities is the leader in terms of costs, as the amount of required investment is equivalent to the amount of costs for the implementation of a new project plus the costs of changing the specific purpose of the site and cleaning up the territory from existing buildings and structures.

Partial redevelopment means a new life of industrial areas and partly of facilities with the modernization or fragmentary renovation of the existing engineering and transport infrastructure. In this case, it is not always necessary to change the status of the site, often the future project is implemented within the existing target purpose or with small adjustments. Typically, this is how office-warehouse or logistics complexes with administrative buildings are formed. Partial redevelopment takes a second place in terms of costs.

As for superficial redevelopment, it usually does not involve serious modernization and constructive changes of existing facilities, so the objects usually are either enterprise’s administrative buildings or separate workshops. This type is often used for building grocery or construction supermarkets, as well as small office complexes of class C. In this case, the transport infrastructure does not change radically. Surface redevelopment is most beneficial in terms of time and financial investments.

A short list of the main steps for investor who has decided to redevelop industrial areas include next:

– to develop a project for transferring production capacities outside the city limits and prepare for the corresponding costs associated with the organization of this process;

– to prepare a project of existing buildings demolition and demolish them in fact;

– to carry out works on cleaning the territory;

– to update communications, more than half of which probably are in a deplorable state;

– to change the purpose of the site;

– to get the right of ownership of the land plot;

– to develop a new plan for the development of the territory;

– to develop real estate projects;

– to conduct directly construction work (to carry out capital costs for construction or equity participation in it);

– to implement the finished object (to lease or sell).

But, despite the complexity of the process, redevelopment remains a very attractive area of activity. Today redevelopment of industrial areas, in particular, industrial buildings is a good way for the investor to minimize costs. As a rule, spacious, multi-span production buildings, especially one-store ones, enables to make a flexible plan, to solve with minimal means the problem of obtaining a large number of today' commercial real estate (retail, entertainment and logistics objects). Many enterprises, that are currently undergoing restructuring, are located near the highways and metro stations, which often becomes almost the determining factor in the successful implementation of the future project. In addition, most of these facilities are provided with engineering infrastructure, in particular power supply, which is advantageously different from undeveloped territories. Today, in large cities, the problem of electric power distribution has become much more acute, and each enterprise, as a rule, has its own powerful switchgear.

It is worth noting that the «starting position» of an industrial facility is not always attractive enough, and this greatly complicates the work with it. For example, very serious problems can arise in the re-profiling of industrial buildings and territories located in the sanitary protection zones of adjacent industrial facilities where the level of environmental pollution is unacceptable for public buildings. The situation with the plant territories is also ambiguous. In some cases, they are quite sufficient for the organization of guest parking lots, corresponding to new needs, and this provides a high attractiveness of the facility for visitors. In others, exact opposite, the territory is extremely small, and then, if decisive measures to organize a multi-level parking are not taken, the facility comes to the «culprit» of automobile pandemonium on the roadside. At the same time, during the construction of new building, when objects in industrial territory are not of value and are subject to demolition, it is important to maintain a balance between the load on it and the areas necessary for servicing transport. It is of great importance to link the transport load to the adjacent city highways, which arises with the construction of a new facility, with the intensity of traffic along them.

Despite all the bureaucratic and economic difficulties of redevelopment, there are enough examples of successful transformation of industrial facilities into commercial buildings in Ukraine. Many cities can boast of good examples, but Kiev and Kharkiv still have leadership in this respect. In the capital such shopping and entertainment centers as «Caravan», «Bolshevik», «Promenade», office facilities «Forum Business City», «Forum Park Plaza» and others were built on the site of industrial facilities. In Kharkiv, the process of redevelopment began with the acquisition of industrial property (production and storage buildings, administrative buildings of large enterprises with an area of more than 1000 m^2) by new owners and their reconstruction with redevelopment for supermarkets («Rost», «Target»), shopping centers («Sun City»), cultural and entertainment centers («Colosseum») and business centers («Telesens»). Today the largest project in Kharkiv from the point of view of industrial redevelopment is the plant «Serp i Molot», the territory of which occupies more than 50 hectares of land in the city center [2, 9].

Both redevelopment and revitalization of spaces are effective methods of functional and stylistic changes in the concept of objects and property management. The difference between concepts lays in the scale of the object and the amount of capital investment. Redevelopment involves more large-scale changes in the facility (changing the function of industrial territory, historic buildings and transport infrastructure) and significant investments. Revitalization of the territory is realized by the redevelopment of the industrial territory in the shopping and entertainment area, the revitalization of buildings – by the restoration of architectural monuments, and the revitalization of urban space – by the reconstruction of transport infrastructure.

Redevelopment can be confused with renovation, revitalization, reconstruction and gentrification. The common thing in all these processes is that they are based on the change, improvement and activation of space. But the ways and programs are different for each one. So, revitalization is not always connected with demolition and re-building.

Redevelopment is a secondary land conversion – the creation of a new, more demanded product of real estate. The economy and society are not still, technologies and social models become obsolete. The objects and territories targeted to them become ineffective or unclaimed. It is a natural process. Therefore, as a rule, redevelopment means re-profiling which is a partial or complete replacement of the function, also by active construction intervention. Redevelopment enables to use those obvious advantages that are given by the concentration and increase in the density of the urban environment in the modern city. Actually, redevelopment can be generally the only possible way to do something in the developed, densely built-up cities.

Redevelopment is a reloading of real estate. The new investment-attractive projects from unclaimed and irrationally used territories are created.

Conclusions

In the conditions of land plots shortage, redevelopment projects provide an opportunity to renew the urban environment, to change not only the architectural appearance, but also the social level. If the urban space is re-organized, transport problems can be solved, the potential for building new roads, bridges, parks and embankments can be founded. Territories that were previously closed, on the contrary, become new places of attraction for citizens.

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Energy performance of buildings in European Union countries and Ukraine

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Complex comparative analysis of building energy performance rates in EU countries and Ukraine has been carried out. The relation between building insulation rates and European countries climate condition has been investigated. It is illustrated that there is a significant gap between building energy efficiency characteristics in Ukraine and in most of the EU countries. Economically justified rates of building envelope heat exchange resistance which can lead Ukraine to common European level based on optimized calculations are suggested. The necessity for further increase in building envelope heat exchange resistance rates in order to raise building energy efficiency and put Ukrainian building regulations in harmony with EU countries corresponding norms is proved.

Keywords: energy performance of buildings, the structure of building energy consumption, building envelope heat exchange resistance, room temperature in winter time.

Енергоефективність будівель у країнах Євросоюзу та Україні

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Розглянуто стан та методи оцінювання енергетичної ефективності будівель у країнах Євросоюзу та Україні. З’ясовано, що вони суттєво відрізняються, це зумовлено кліматичними, економічними й історичними особливостями регіонів. Виявлено, що у деяких країнах ЄС регулярно переглядаються і підвищуються вимоги до енергетичних характеристик будівель. З’ясовано, що одним із головних показників енергоефективності будівель є теплотехнічні характеристики огороджувальних конструкцій і передусім їх опір теплопередачі. У всіх країнах установлено власні вимоги до теплозахисних характеристик огороджувальних конструкцій, які періодично змінюються, як правило, у бік збільшення. Таку динаміку зростання нормативних показників теплозахисту огороджень будівель в Україні простежено за останні півстоліття. Засвідчено, що за цей період вони зросли у 3,5 – 4 рази. Виконано комплексний порівняльний аналіз показників енергоефективності будівель у країнах Євросоюзу та в Україні за декількома показниками теплозахисту: опором теплопередачі огороджувальних конструкцій, відносними втратами тепла через огородження й температурним режимом житлових приміщень. Проаналізовано зв’язок показників теплової ізоляції будівель з кліматичними характеристиками для країн Європи. Показано, що за характеристиками енергоефективності будівель Україна суттєво відстає від більшості країн ЄС. На основі оптимізаційних розрахунків з урахуванням сучасної вартості теплової енергії та теплоізоляційних матеріалів запропоновано економічно обґрунтовані значення опору теплопередачі огороджувальних конструкцій, які за цим показником виведуть Україну на загальноєвропейський рівень. Підтверджено необхідність подальшого збільшення опору теплопередачі огороджень з метою підвищення енергоефективності будівель та гармонізації українських будівельних норм з нормативними вимогами країн ЄС.

Ключові слова: енергоефективність будівель, структура енергоспоживання будівель, опір теплопередачі, огороджувальні конструкції.

Introduction

The issues of energy efficiency and creating of corresponding micro climate within the building is the component of government energy saving policy. In 2017 Ukraine adopted the Law of Energy Performance of Buildings [1], which determined legal, social and economic as well as organizational operational principles in provision of construction energy efficiency and was aimed at reduction of energy consumption in buildings, mainly: ensuring building energy performance according to existing technical standards, national standards, norms and regulations; stimulation of energy consumption reduction in buildings; assurance of the reduction of greenhouse gases emission; provision of building thermal modernization and stimulation of renewable energy sources usage, development and implementation of the national plan concerning the expanding the number of buildings with energy consumption level close to zero (“passive houses”). Despite of the fact that for the latest decades building thermal protection level has grown significantly, heating energy costs still remain unreasonably high, especially in building without thermal modernization.

That is why it is reasonable to study and introduce European practices to ensure energy performance of buildings.

Review of research sources and publications

The issue of energy efficiency and energy saving has been studied in many papers of scientists from Ukraine and other countries: H.H. Farenyuk [2 – 5], Yu. A. Tabunshchykov [6], V. L. Kurbatov [7], Yu. A. Matrosov [8] and others. The researchers consider the methodological grounds for establishing norms and standards alongside with provision of reasonable values for energy performance of buildings and enclosing structures thermal reliability, the increasing of basic heating-performance rates of heat-insulating envelope major elements etc. But these studies, as well as some other, do not deal with the issue of optimal (economically feasible) level of thermal protection - enclosing structures heat transfer resistance - based on the criteria of cost given and often neglect common European experience.

Problem statement

Recommendations for economically feasible values of walls and surfaces heat exchange resistance for non-industrial buildings in Ukraine are given based on calculations and comparative analyses of buildings energy performance rates in EU countries and Ukraine, especially the building envelope thermal protection levels.

Basic material and results

The methodology for evaluating and regulating energy performance of buildings differs in EU countries and in Ukraine. This is determined by their climatic, economic and historical peculiarities. It also has a negative impact on the development of civil engineering in general and creates certain difficulties at common European market due to incompatibility of different building energy efficiency rates.

Some differences in calculating methods, in requirements to building thermal characteristics and methodology of their control can be observed, as well as the proposed measures of building regulations harmonization in the energy efficiency sphere. The group of EU experts conducted a research into the issues mentioned above in 13 countries: Belgium, Great Britain, Denmark, Italy, the Netherlands, Germany, Norway, Romania, Slovenia, Hungary, Finland, France and Sweden.

In all these countries active measures are taken on both levels of energy efficiency provision standards and technology and engineering. In some countries regulations are being revised every 2–3-years and the demands towards building energy efficiency are increased according to long-term programmes.

The analysis shows that countries in Central Europe follow EU and European Institutions regulations in a stricter way in comparison with the countries on the outskirts. The countries with cooler climate traditionally pay much more attention to general energy efficiency issues on the contrast to ones with more favourable weather conditions, where the emphasis are on the matters of reducing energy waste for cooling the interior in summer. Among the leaders in consistent energy efficiency policy implementation one can name Germany, France, Great Britain as well as Denmark and some new EU members like Slovenia [9].

In all countries energy costs for heating, hot water supply and ventilated air heating (including infiltration) are regulated. In many countries air cooling and conditioning expenses are also included. Besides, the electric energy consumption used by heating, ventilation and conditioning systems is, as a rule, controlled. The structure of building energy consumption according to construction standards in Finland can be taken as an example (Fig. 1).

Most of the countries use natural energy losses as a criterion to express energy performance of buildings which is as a rule given as K Wh/m² per year (in Italy K Wh/m³ per year). Only Great Britain and Romania use the amount of CO_2 emission as a criterion for energy performance of buildings. Natural energy use level coefficient differs significantly for various countries, thus in most cases it is equal to 1 for all the fuels and 2.5 for electric power [9].

One of the major indicators for energy performance of a building is its enclosing structures (building envelopes) thermal and technical characteristics, mainly heat exchange resistance. In all countries have their in-house requirements to enclosing structures characteristics which are being changed regularly with a tendency towards increasing (Fig. 1 according to data from [9]). Such dynamics of raising standards for building envelope heat exchange resistance values for non-industrial buildings in Ukraine for the last 50 years was investigated in [10]. The study shows that within this period these values increased for 3.5–4 times. In some countries (Italy, Spain, France) the values of enclosing structures characteristics can vary according to the region depending on climatic conditions, determined by location, including proximity to the sea and the altitude. Using the same principal and with accordance to existing regulations [11], the territory of Ukraine is divided into two temperature zones with minimal required values for residential and civil building heat exchange resistance R = 3.3 m²·k/W in the first zone and R = 2.8 m²·k/W for the second one. The first zone includes central, western and northern regions of Ukraine, the second one - its southern parts.

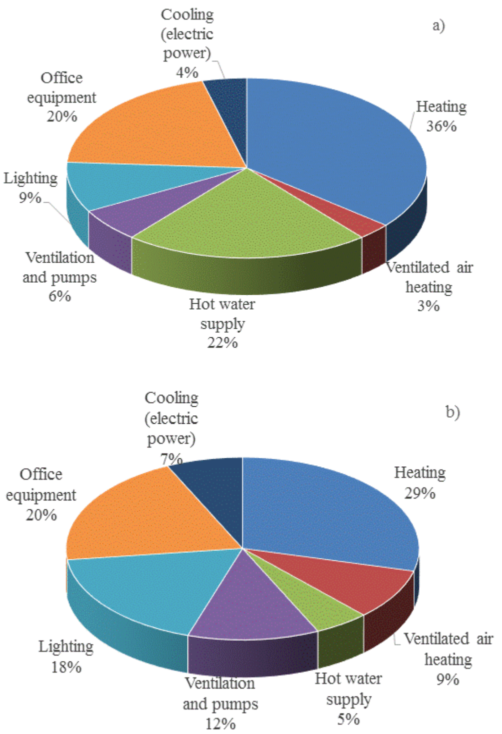

Figure 1 – The structure of building energy consumption in Finland:
a – one apartment building with total power consumption (heat and electric power) 78 K Wh/(m²·year);
b – office building – 123 K Wh/(m²·year)

Sometimes heat exchange resistance values in European countries can differ for residential and civil building as well as for building of different shapes. In Italy, Denmark, Slovenia and Germany (for residential buildings) heat exchange through enclosing structures is limited by usage of mid value of thermal characteristics and in Hungary - by demand for heating power. Finland and Norway use less severe requirement to thermal protection in wooden structures aiming at preserving traditional wooden house construction. In Sweden significantly higher building envelope heat exchange resistance values are set for buildings with electric heating.

Thermal protection values for translucent enclosing structures (windows) are also calculated and standardised in all the countries, including Ukraine [9, 11]. However, in Finland, besides this characteristic, limitations are set for areas covered by translucent enclosing structures.

In many European countries there are measures to reduce heat input with solar radiation, but only several as well set the limits for the numerical values of heat inputs with solar radiation through translucent enclosing structures (so-called g-window factor). In Ukrainian State Construction Regulations [11], instead, there are requirements for some regions to make calculation for heat resistance in summer time for structures with relatively low thermal inertia, which are based on setting limitation for temperature fluctuation amplitude at external enclosing structure surface.

Table 1 – Thermal resistance enclosing structures necessary values in Europe

Europe	Norms introduction year	Average January temperature τ_e	Walls		Roofs	
			R_0	$\Delta\tau/R_0$	R_0	$\Delta\tau/R_0$
Belgium	2008	2	2.00	9.00	3.33	5.41
UK	2010	3	5.55	3.06	6.67	2.55
Denmark	2006	0	5.00	4.00	5.56	3.60
Italy	2010	5	3.03	4.95	3.45	4.35
Nether-lands	2011	2	3.45	5.22	3.45	5.22
Germany	2009	-1	3.57	5.88	5.00	4.20
Norway	2007	-7	5.56	4.86	7.69	3.51
Romania	2006	-2	1.41	15.60	3.03	7.26
Hungary	2006	-1	2.22	9.46	4.00	5.25
Ukraine	2013	-5	3.30	7.58	4.95	5.05
Finland	2010	-8	5.88	4.76	11.11	2.52
France	2005	3	2.78	6.12	5.00	3.40
Sweden	2008	-6	5.56	4.68	7.69	3.38

As it is given in Table 1, the parameters given can change in different countries within a wide range. For instance, average temperature in January varies from –8˚C in Finland to +5˚C in Italy. Necessary heat exchange resistance values for walls are set within the range of 1.41m²·k/W in Romania up to 5.88 m²·k/W in Finland, and ones for the roofs are from 3.03 m²·k/W in Romania to 11.11m²·k/W in Finland.

In Fig. 2 there are dependencies of standard walls and roofs heat transfer resistance from average temperature in January in different countries. As one can see from this figure, there is not any proportional or other kind of dependency between those parameters, though logical tendency for increasing of required walls and roofs heat transfer resistance values at lower average January temperature is observed.

In both graphs points that reflect requirements of the UK norms are to bigger side and values for Romania are to lesser side. The dot with large luminous marker, which reflects Ukrainian standards [11], is also on lower bound of both dependencies, indicating certain understatement of requirements [11] compared with other European countries.

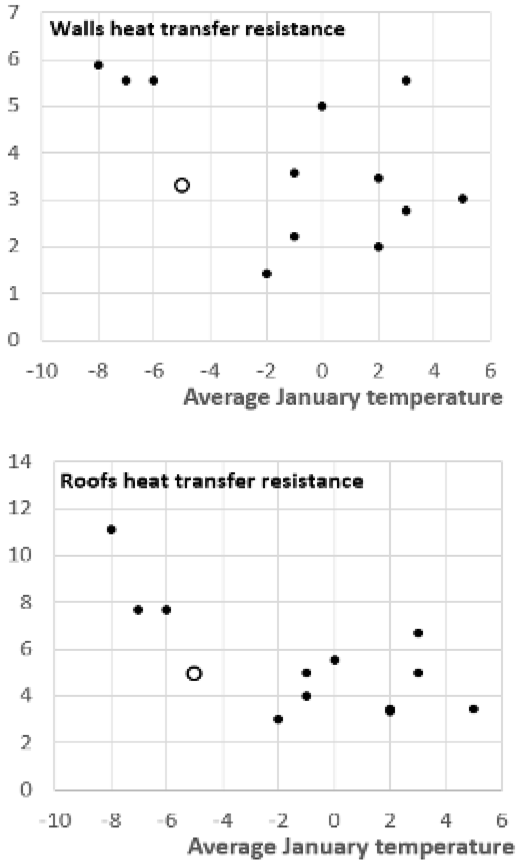

Figure 2 – Heat transfer resistance dependence from average January temperature

Table 1 also shows temperature difference ratio between internal and external air $\Delta\tau$ = 20ºC – τ_c to walls and roofs heat transfer resistance required R_0. This value is proportional to heat losses through enclosing structure and can serve as a material for analysing compliance of walls and roofs thermal characteristics of their operation climatic conditions. According to Table 1, values $\Delta\tau/R_0$, diagrams, shown in Figure 3, are plotted, indicating buildings thermal insulation relative efficiency in European countries.

Figure 3 diagrams indicate some requirements imbalance for walls and roofs in different countries. For example, France occupies 9th place for wall constructions effectiveness and 4th place for roofs effectiveness. The smallest heat losses are provided by Northern Europe designing norms, and the greatest heat losses are allowed by standards in Romania. Ukrainian norms for buildings thermal insulation [11] are in the 9...10th position among the 13 countries considered, with relative efficiency. This is confirmed by the results of dependencies analysis in Figure 1 and indicates increasing requirements expediency for required to enclosing structures heat transfer resistance in Ukraine.

In the research [12] by minimizing total reduced costs for enclosing structures building and heat energy cost for heating buildings, appropriate dependences of walls and roofs heat transfer resistance to thermal energy cost C_T: $R_C = 0.16\sqrt{C_T}$, $R_П = 0.21\sqrt{C_T}$ are obtained. Calculation according to these formulas indicates that according to existing norms [11] minimum required heat transfer resistance values of walls R_{0C} = 3.3 м^2·K/W and roofs $R_{0П}$ = 4.95 м^2·K/W correspond to thermal energy cost 425...556 UAH/Gcal.

At current heat cost about 1400 UAH/Gcal, walls heat transfer resistance in accordance with above formulas should be set equal to R_{0C} = 6.0 m^2·K/W, and for the roofs $R_{0П}$ = 7.9 m^2·K/W. Figure 2 shows that these values are in line with general trend for Europe. In this case, heat losses relative index through walls is $\Delta\tau/R_{0C}$ = 4.2, and through roofs $v\Delta\tau/R_{0П}$ = 3.2. It is evident from Table 1 and Figure 3 that such indicators lead buildings Ukrainian norms for thermal isolation to third place among 13 considered European countries.

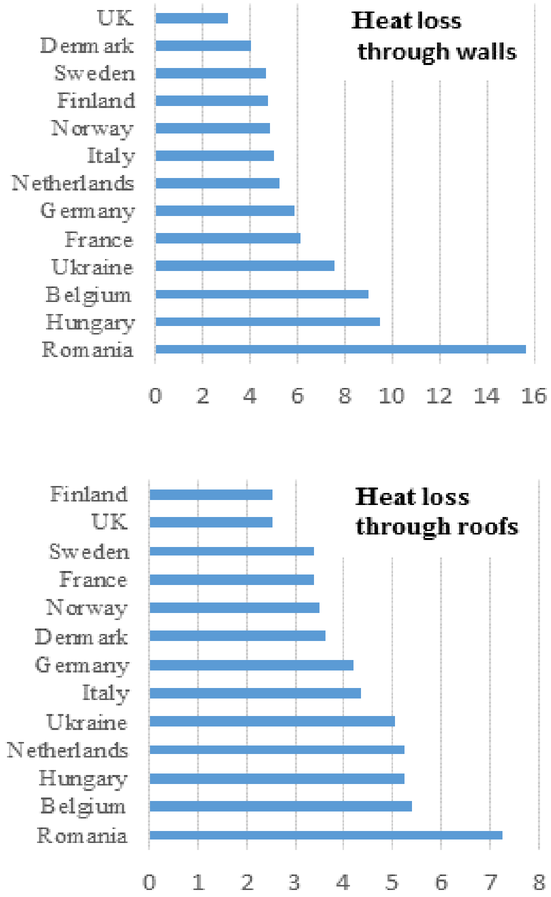

Figure 3 – Heat loss through enclosing structures relative indicators

German company Tado, which produces intelligent thermostats that are connected directly to the Internet, published interesting statistical data of average over-night (lower) temperatures that the users maintain in their apartment or house bedrooms. Data was obtained from tens of thousands of thermostats installed in different European countries (Table 2) [13].

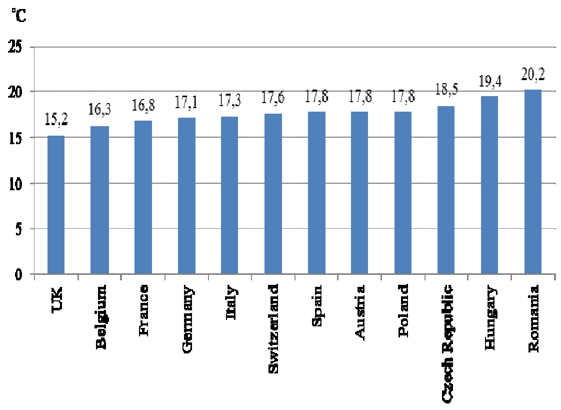

Figure 4 – Average overnight temperatures in bedrooms in European countries (winter time)

Calculated temperature in rooms in Ukraine is 20˚C as it is set by the design specification [14]. If to assume that actual temperature in winter is close to that value, then Ukraine is next to Romania according to this energy saving rate.

As it appears, Britons tend to save more and keep the average temperature at 15.2˚C at night. The highest temperature is in Romania – 20.2˚C. It is notable that data from Table 2 corresponds well to the heat losses through enclosing structure values given in Fig. 3. It means that such countries as Great Britain as well as some others take the systemic approach to heat energy saving: Adopt high levels of building envelope heat exchange resistance and, at the same time, maintain relatively low temperatures inside in winter. The conclusion that can be made based on the given data is the more prosperous the country is, the more practical its residents are, and vice versa, if the citizens take more reasonable approach to energy saving, it can help the country to thrive.

Conclusions

The analysis made has indicated that in terms of building energy saving and energy efficiency rates Ukraine takes one of the last positions in Europe. The actual requirements to building for thermal isolation do not correspond to common European standards and call for increasing of minimal heat transfer resistance value for enclosing structures. It should be applied to both newly constructed buildings and thermal modernization of already existing ones. The proposed economically justified values for walls and roofs heat resistance based on calculation with account for current power costs can help Ukraine take its thermal protection standards to European level.

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Calculation of phace change heat accumulator in complex of energy efficient ventilation system

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The classification of the main seasonal heat energy storage batteries is given, and the modern circuit diagram of the phase shift battery as part of the system with a heat pump and a solar collector is considered. The disadvantage of water heaters is their large volume in most cases. By utilizing the accumulated latent heat of substances, a significant reduction in capital costs is achieved. The possibility of creating energy-saving ventilation systems with the use of a seasonal heat accumulator working on phase transformations of heat-accumulating material is considered. The linear stationary mathematical model of the battery thermal balance is made and on the basis of the calculation results graphs and diagrams are constructed enablingto analyze the work of the heat accumulator during the year. The classification of the main seasonal accumulators of thermal energy and ventilation systems with a ground heat exchanger use is considered. In the article the theoretical and computational research of the seasonal heat accumulator creation and application possibility working on phase transitions of heat-accumulating substances (water) in the ventilation system of a residential individual house is given.

Keywords: heat accumulator, energy efficient ventilation system, phase transition.

Розрахунок сезонного теплоакумулятора у складі енергоефективної системи вентиляції

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Наведено класифікацію основних сезонних акумуляторів теплової енергії та розглянуто сучасну схему роботи акумулятора фазового переходу у складі системи з тепловим насосом і сонячним колектором. Недоліком водяних акумуляторів тепла є те, що в більшості випадків вони займають великий об'єм. Шляхом застосування акумулюючих приховану теплоту речовин досягається значне зниження капітальних витрат. Розглянуто можливість створення енергозберігаючих систем вентиляції з використанням сезонного акумулятора теплоти, що працює на фазових перетвореннях теплоакумулюючого матеріалу. Розглянуто класифікацію основних сезонних акумуляторів теплової енергії та систем вентиляції із застосуванням ґрунтового теплообмінника. Складено лінійну стаціонарну математичну модель теплового балансу акумулятора й на основі результатів розрахунку побудовано графіки та діаграми, що дозволяють проаналізувати роботу теплоакумулятора протягом року. Прийняті спрощення у процесі моделювання полягають у тому, що рішення виконано для точкової задачі. Розв'язок здійснено з використанням логічних та математичних функцій на основі рівнянь, складеної математичної моделі. Наведено теоретичне і розрахункове дослідження можливості створення й застосування сезонного акумулятора теплоти, що працює на фазових переходах теплоакумулюючих речовин (води) у системі вентиляції житлового індивідуального будинку. Основні отримані результати полягають у тому, що у теплий період року акумулятор виконує роль вирівнювача амплітуди коливань температури, тим самим створюється можливість зменшити потребу в холоді для роботи системи кондиціонування повітря. Також доведено перспективну можливість зменшення маси теплоакумулюючого матеріалу за рахунок поперемінної зміни процесів заряджання-розряджання в акумуляторі у результаті добової зміни зовнішньої температури повітря у холодний період року, а саме її добового підвищення.

Ключові слова: теплоакумулятор, енергоефективна система вентиляції, фазовий перехід.

Introduction

Conventional water accumulators in most cases, as well as heat accumulators of rocks and earth, occupy a large volume. By using the accumulated latent heat of substances, a significant reduction in the cost of materials, as well as the required area for heat-storage facilities is achieved. It applies to low-temperature and high-temperature heat accumulators. It is due to fact that the heat-accumulating material is replaced without significant changes in the design of already existing heat accumulators. It can be done by placing in the reservoirs of accumulators heat-accumulating bodies of various forms (balls, thin cylinders, flat plates) [1].

In the article there is given the theoretical and computational research of the possibility of seasonal heat accumulator creation and application, working on phase transitions of heat-accumulating substances (for example water), as a constructive element of the ventilation system of a residential individual house.

Review of research sources and publications

The classification of the main known seasonal accumulators of thermal energy for today [1, 9, 10] is given. There are work circuits of year-round soil heat exchangers in the building ventilation structure [6, 7]. The principle scheme for the use of a heat pump with a phase change accumulator is known [2]. The use of phase change materials for the needs of life support systems for buildings and structures is considered (Fig. 1). These are: 1 -- The accumulation of hidden heat for heating the interior space of the premises; 2 -- Plaster and complex systems of facades with high specific accumulation capacity; 3 -- Translucent thermal isolation and daylight design; 4 -- Shaving phase transition materials in composite building systems; 5 -- Application of phase transition material in gypsum materials and paints; 6 -- Phase transitional materials in solar-air systems to stabilize their temperature mode of operation.

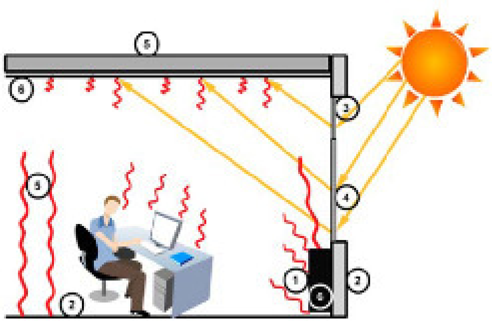

Figure 1 – Application of phase change materials in buildings and structures

Definition of unsolved aspects of the problem

Due to the cost constant growth of traditional energy resources, the massive installation of plastic windows is acutely a problem in the development of energy-efficient and low-cost ventilation systems of the building, especially in the winter period of the year. A well-designed and assembled ventilation system provides the necessary hygienic conditions in premises of the buildings and does not require much cash expenditures. The article aims are to explore the prospect of reducing the mass of heat-accumulating material by using the process of periodic recharging of the phase transition battery with warm air, which is possible due to the daily change in the temperature of the external air, namely its increase.

Problem statement

The main tasks of the work are: to carry out the calculation of the heat storage accumulator of the phase transition installed in the individual cottage during the season with the estimated time step of 3 hours using the linear mathematical model of the stationary heat transfer process; to determine the need for heat accumulating material (HAMs) and the temperature mode of the accumulator, assuming that the air temperature at the outlet of the battery is equal to the temperature of water or ice at the end of the heat exchange process for a certain period of time, that is, stationary mode is established; to consider the basic designs of the technical knowledge of seasonal thermal accumulators available at the present level; to investigate the possible temperature regime that occurs in the phase of accumulator change and to construct schedules for changing the absorbed and devoted energy during its operation over the year (one season).

Basic material and results

Recently, in the modern oversea market of engineering solutions, an energy-efficient scheme (Fig. 2) of the seasonal heat accumulator with a phase change with a heat pump use, an air solar collector and passive use of the Earth thermal energy, provided that the accumulation capacity is laid below the soil freezing zone, has appeared.

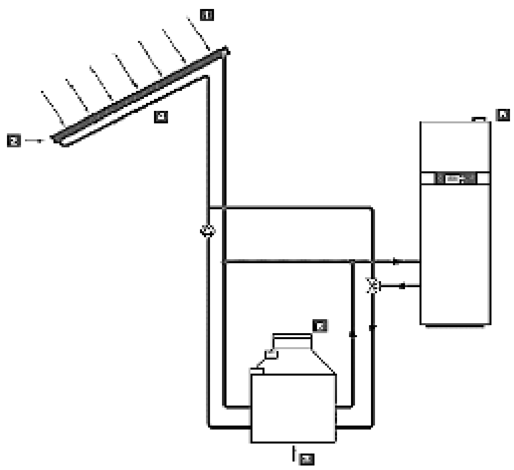

Figure 2 – Principle diagram of a heat pump with a solar collector and a phase change heat accumulator:
1 – solar heat; 2 – heat of air; 3 – heat of the earth; 4 – solar-air absorber; 5 – heat pump; 6 – thermal storage tank of the phase transition

The above method of using the heat-exchange circuit of a heat pump with a heat storage tank of the phase transition is completely ineffective because the tube icing dramatically reduces the heat output from the water, even though the higher thermal conductivity of the ice compared with the thermal conductivity of the water. Here it is necessary to consider the convection motion of water, because due to this movement the main heat exchange occurs in the liquid, but not at the expense of thermal conductivity.

If it is applied a heat exchange system of this type, then:

1) it is had to use an unreasonably long tube;

2) it is impossible to freeze all water in the heat accumulator due to the fact that the expansion of the ice leads to the depressurization of the container or the rupture of the heat exchange tube, which requires the application of special measures to prevent this negative phenomenon;

3) changing such a heat accumulator, it can be imagined that in a block of ice, the tube, which circulates the heat carrier, is depressurized, that leads to a complete stop of the heating system at the time of the thawing of the battery;

4) due to ice freezing of ice in this system, a low coefficient of efficiency is obtained;

5) the high final cost of the entire system is got due to the above-mentioned shortcomings.

Get warm in winter from the solar collector and heat the bottom of the heat accumulator due to the heat of the earth - are relevant and promising solutions, but the accumulator should be buried below the depth of the freezing zone of the soil, which in turn requires substantial investment. There can be also used low-potential waste heat from drainage and exhaust ventilation systems. Using these and other heat sources significantly reduces the amount of seasonal heat accumulator.

A seasonal or long-lasting heat accumulator requires a significant amount of accumulation material. This material should be inexpensive, have as high a heat capacity as possible and be non-toxic in terms of environmental impact. Another problem is the corresponding reservoir, which, in turn, should have good thermal insulation properties and a fairly rigid construction. Famous large-scale seasonal heat accumulators can be divided into the following types [3, 4, 8]:

- water pools and reservoirs (natural or artificial);
- ground accumulators (with vertical or horizontal heat exchangers);
- aquifers (filled with water by porous geological formations).

Widespread distribution in ventilation systems for preheating and cooling of the inflow air acquire soil heat exchange contours [7], Fig. 3. However, during the operation, the problem of choosing the material of the battery pipes is acute, as they are subject to complex "freeze-thaw" processes on the outer surface and icing in the winter and the formation of condensation in the summer on the inner surface. The need to control the purity of the tidal air - the formation of condensate is an enabling environment for the development of various pathogenic micro-organisms. Considering capital installation and construction costs, the difficulty of installing a coil of pipes laid down below the soil freezing zone.

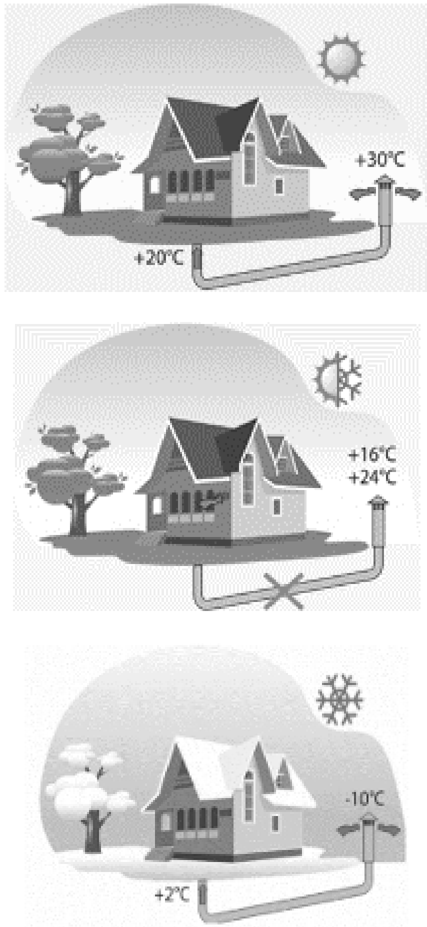

Figure 3 – Different operating modes of the ground heat exchanger GHE (from up to down: summer, shoulder period and winter)

In order to investigate the prospective possibility of using a thermal battery for phase transition in domestic ventilation systems and assessing its energy efficiency during one season (year), it is considered its application for the needs of providing fresh air to the premises of an individual dwelling house (cottage) with a total living area of 100 m2.

Total air flow rates for an individual dwelling house are

$$L_{tot}= L_r+ L_c+ L_t, \qquad (1)$$

where $L_r = F{\cdot}K$ and L_c, L_t – the value of air exchange, respectively, for living rooms, kitchens and detached lavatory, taken in accordance with the norms [10], m^3;
F – area of living rooms, m^2;
K – multiplicity of air exchange, 1/our.

The system time is 720 hours per month. The estimated internal air temperature in the living rooms is + 18 ° C. The temperature of the inflow air can be reduced by 3 ° C, considering that the air is distributed through a grid located under the ceiling of the room. The total air exchange for a home is:

$$L_t = 100 + 90 + 50 = 240 \text{ m}^3/\text{our}.$$

Output values of the outside air initial temperature are taken according to [5] over a period of 3 hours for the town of Poltava for 2015 (correspondingly, and increase in the amount of air for a 3-hour increment). The initial mass of heat-accumulating material (water) was taken at 4.5 tons and in the process of computer calculation has been clarified. The initial temperature of the working material for December 31.12.2015 is assumed to be 5.1 °C. Simplifications are accepted, namely, the solution is made for the point problem, that is, the battery is at a certain time, either water or ice. The solution was implemented using MS Excell software using logical and mathematical functions based on the equations given and obtained below in the presented work.

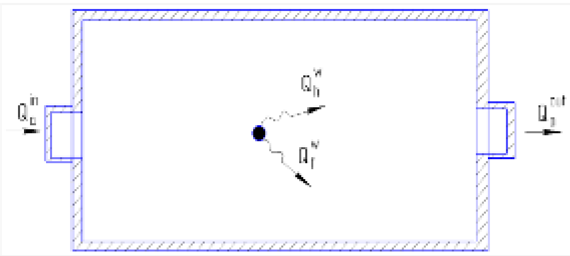

Figure 4 – Graphical representation of heat flows in a heat accumulator: Q_a^{in}, Q_a^{out} - respectively, the thermal flows of the inlet and outlet air; Q_h^w,Q_f^w is the explicit and hidden heat of the phase-accumulating material, respectively

The model diagram of heat flows in the thermal accumulator is plotted (Fig. 4) and from the heat amount thermal balance equation of air, water and ice the formula for the final temperature of water, °C is determined:

$$\begin{cases} IFM_1 = 0 \rightarrow c_a \times G_a \times (t_a^k - t_a^n) \times \tau = c_w \times M_w \times (t_w^n - t_w^k) \\ c_a \times G_a \times (t_a^k - t_a^n) \times \tau = K \times F \times \left(\dfrac{t_w^n + t_w^k}{2} - \dfrac{t_a^n + t_a^k}{2} \right) \\ IFM_1 < M_w \rightarrow c_a \times G_a \times (t_a^k - t_a^n) \times \tau = r \times \Delta m_1; \\ t_w^n = t_w^k = t_f^w = 0 \\ c_a \times G_a \times (t_a^k - t_a^n) \times \tau = K \times F \times \left(0 - \dfrac{t_a^n + t_a^k}{2} \right) \\ IFM_1 = M_w \rightarrow c_a \times G_a \times (t_a^k - t_a^n) \times \tau = c_1 \times M_1 \times (t_1^n - t_1^k) \\ c_a \times G_a \times (t_a^k - t_a^n) \times \tau = K \times F \times \left(\dfrac{t_1^n + t_1^k}{2} - \dfrac{t_a^n + t_a^k}{2} \right) \end{cases} \quad (1)$$

where c_w, c_i – heat capacity of water and ice, kJ/kg·°C;

t_w^n, t_w^k – respectively, the initial and final water temperature, °C;

t_a^n, t_a^k – respectively, the initial and final air temperature, °C;

$r = 336$ – hidden heat of melting and crystallization, kJ/kg,

M_w, M_I – respectively, the mass of water or ice in the heat accumulator, kg;

Δm_1 – amount of formed ice as a result of leakage of heat exchange processes, kg,

c_a – heat capacity of air, kJ/kg·°C;

G_a – mass flow of supply air, kg/s;

t_1^n, t_1^k – respectively, the initial and final ice temperature, °C,

t_f^w – temperature of the phase transition of the heat-accumulating material (water), °C;

F – the area of the heat-exchange surface of the capsules with the material, m^2;

K – heat transfer coefficient, W/m^2°C,

τ – time settlement interval, s.

Since stationary heat transfer process, it is got:

$$\begin{aligned} &IFM_1 = 0 \rightarrow t_w^k = t_w^n - \frac{c_a \times G_a \times (t_a^k - t_a^n) \times \tau}{c_w \times m_w} \\ &IFM_1 < M_w \rightarrow t_w^k = t_w^n = t_f^w = 0 \\ &IFM_1 = M_w \rightarrow t_1^k = t_1^n - \frac{c_a \times G_a \times (t_a^k - t_a^n) \times \tau}{c_1 \times m_1} \end{aligned} \quad (3)$$

The summary mass of ice (M_I) formed as a result of heat exchange between cold air and water from i=1 to n – all estimated time intervals, kg:

$$\begin{cases} \Delta m_1 = \dfrac{c_a \times G_a \times (t_a^k - t_a^n) \times \tau}{r} \\ M_1 = \sum\limits_{i=0}^{n} \Delta m_1 \end{cases} \quad (4)$$

The results of modeling the thermal mode of the thermal battery are shown in the graph (Fig. 5).

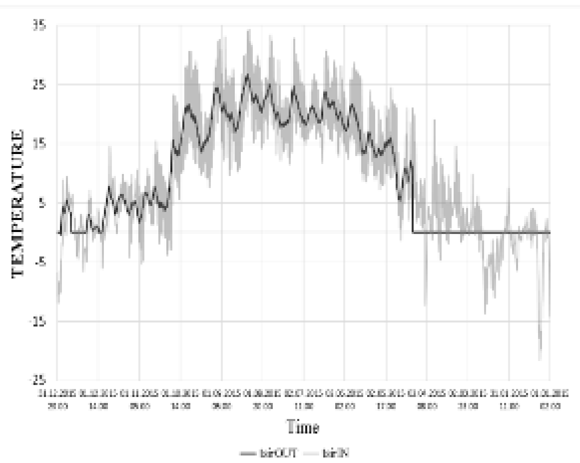

Figure 5 – Chart of the external temperature and air temperature at the outlet of the heat accumulator every 3 hours

From the graph it is clear that the temperature of the air after processing in the phase transition accumulator in the winter period of the year is maintained at not less than 0 ° C, which enables to increase the supply air to the room in the recuperator to the desired temperature and prevent the icing of the above heat exchanger. In the warm period of the year, the accumulator carry out a pivotal role in the stabilizer of the amplitude of temperature fluctuations, which reduces the cost of cooling energy for the needs of the air conditioning system.

The graph of changes in the amount of water and ice present at a certain time in a phase-change accumulator is shown below (Fig. 6).

Analyzing the chart above, it can be stated that with a more detailed calculation, the need for heat accumulating material drops is due to alternating changes in charging and discharging of the accumulator during the year. As a result, the weight of water in the battery decreases.

Diagram showing the change in the thermal mode of the battery for the study season is shown in Figure 7.

In the graph above the zero is shown the process of charging, that is, the amount value of heat absorbing HAM (water) in the process of charging; below zero is the process of discharging, namely, the amount of heat that emits water or cooling, or as a result of a change in aggregate state, turning into ice. In this case, the outside air is either cooled or heated, respectively. Also, the alternation of the heat exchange processes occurring in the battery is clearly visible.

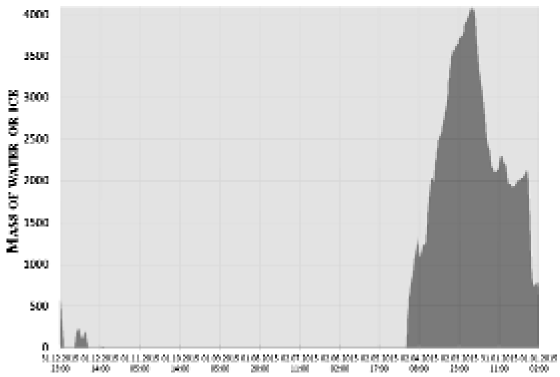

Figure 6 – Diagram of water and ice distribution in the heat accumulator
(black-mass of ice; gray – mass of water)

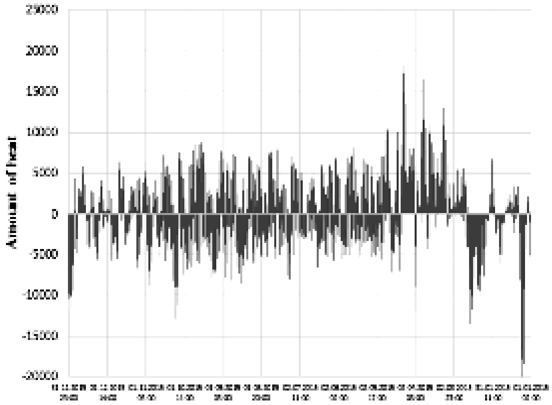

Figure 7 – Diagram of the heat accumulated in the heat accumulator amount distribution

Conclusions

1. The classification of the main seasonal heat accumulators is given. The diagrams of the work of the soil heat exchanger in different periods of the year are presented. The system consisting of a phase change accumulator and the heat transfer circuit of the heat pump enclosed in it are described; the main shortcomings of the above-mentioned engineering solutions are given.

2. With a more detailed hourly calculation, it is possible due to the alternating change of charge-discharge processes in the accumulator to reduce the need for heat-accumulating material more than twice.

3. Based on the calculations using the mathematical model of thermal balances of water, ice and air, a diagram of the distribution of air temperature after treatment in the accumulator, diagram of the formation of ice and the amount of accumulated heat was constructed and analyzed.

4. In the warm period of the year, the acumullator acts as an equalizer of the temperature fluctuation amplitude thereby creating the ability to reduce the need of the cold energy for the needs of the air conditioning system. The promising possibility of reducing the mass of heat-accumulating material from 9 to 4 tons is proved.

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The scientific and technical activity module development for the department of structures from metal, wood and plastics

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The problems of society informational support, which demand the creation of modern information management systems for different objects were considered. Particular attention was paid to the detailed description and analysis of existing services for keeping records of scientific activity in higher educational institutions. The necessity of module creating for management the scientific and technical activities, which will allow to optimize the procedure for conducting reports on scientific, technical and innovative activities, to expand the possibilities for the results analysis, to ensure transparency and objectivity in the procedure of evaluating the teachers' activities, was revealed and explained. The developed sections of the technical enquiry were given. On the basis of the conducted research the software implemented module for management the scientific and technical activity of the department.

Keywords: informational intellectual control system, intellectual module, semantic analysis of the text, scientific activity.

Розроблення модуля управління науковою та науково-технічною діяльністю кафедри конструкцій з металу, дерева та пластмас

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Розглянуто проблеми інформатизації суспільства, яке вимагає побудови сучасних інформаційних систем управління різними об'єктами. Проаналізовано теоретичні та методологічні аспекти вдосконалення ведення звітності у вищих навчальних закладах. Особливу увагу приділено детальному опису й аналізу існуючих сервісів ведення обліку наукової діяльності вищих навчальних закладів. Виявлено та обґрунтовано необхідність створення модуля управління науковою й науково-технічною діяльністю, який дозволить оптимізувати процедуру ведення звітів про наукову, науково-технічну та інноваційну діяльність, розширити можливості для аналізу результатів, забезпечити прозорість і об'єктивність у процедурі оцінювання діяльності викладачів. Наведено основні критерії автоматизованої інформаційної системи. Представлено розроблені розділи технічного завдання. Передбачено виконання вимог реалізації інформаційної системи у вигляді Web-додатка. Наведено опис концептуальної схеми за допомогою узагальнених конструкцій блоків та розроблену ER-модель. Розраховано мінімальну конфігурацію сервера. На основі виконаного дослідження запропоновано програмно реалізований модуль управління науковою та науково-технічною діяльністю кафедри конструкцій з металу, дерева та пластмас (КМДіП), у якому враховано недоліки існуючих інформаційних систем управління. Система забезпечує публікування наукових матеріалів, можливість перевірки тексту на унікальність і генерування автоматичних звітів, що значно полегшить роботу викладача чи наукового діяча, виконує розмежування користувальницької та адміністративної частин програмного комплексу, автоматичне формування звітів про наукову, науково-технічну й інноваційну діяльність та надає можливість розроблення чи додавання (поповнення) нових публікацій і їх категорій.

Ключові слова: інформаційна інтелектуальна система управління, інтелектуальний модуль, семантичний аналіз тексту, наукова діяльність.

Introduction

Nowadays the question of present interest is the system creation for management the departments of higher educational institutions scientific and technical activities. Universities in Ukraine and abroad are involved to solve this problem, but the exact analogues that meet all the requirements of the given technical enquiry, has not been found. Thus, this problem is becoming more and more important [6] due to the general rapid development of science and technology and the national education systems redeveloping which require up-to-date solutions and the introduction of advanced models, tools, methods and technologies for creation modern intelligent information management systems, in particular, management of scientific and technical activities of the department [7, 8].

One of the promising areas of informational support is the development of resource management systems for companies and enterprises (ERP-systems). Now, the question of present interest is the computer support systems for managing universities and education quality creation. The most of countries of the European Union and beyond it deal with this problem. This is due to the constant redevelopment of national education systems, the Bologna process development as well as the complexity of the subject area.

Considering the business process of quality assurance, it is based on the «Standards and Guidelines for Quality Assurance in the European Higher Education Area» developed by the European Association for Quality Assurance in Higher Education on the direct request of Education Minister of the European countries Conference signed by the Bologna Declaration. The European Association for Quality Assurance in Higher Education is an organization that aims to maintain and increase the quality of European education to a high level [9].

Review of research sources and publications

The investigation of theoretical and methodological aspects of improving the reports in higher educational institutions with the help of information systems is reflected in the works of many native and foreign scholars. The core and importance analysis of information systems in general has been carried out by a number of western authors [13], [15 – 19]. The current state of the information systems development and their classification are presented in the works [20 – 22]. The basic requirements for information systems are given in the works [23 – 26]. At the same time, there is a need to develop and implement a specific information system to automate the departments scientific work accounting.

Definition of unsolved aspects of the problem

For the effective work of the department teaching staff of higher educational institutions, there is a need to create an information intellectual system project to manage the scientific-technical activity.

The scientific and technical management module enables to optimize the procedure for keeping reports on scientific, technical and innovative activities, to reduce the costs of department scientific activity organizing by optimizing the use of all department resources, improving staff production performance and efficient management of the paid scientific research services provision. It promotes the development of academic freedom by ensuring the activity transparency of all parties involved in the system.

Problem statement

The task of this research is to design and implement the module for the department of structures from metal, wood and plastics scientific and technical activities management. The main purpose of this development is to create a web-module that has the functions of publishing scientific materials and automatically generate reports for the department of structures from metal, wood and plastics.

The aim of the research is to improve the quality of subdivision research activity, to provide scientific materials publication, to check possibility of the text for uniqueness and automatic reports generating, which greatly eases the work of teachers or scientists. Target audience is teachers, researchers, students, school leavers, and people interested in this information, etc.

Basic material and results

The main criteria for the project of automated information system are shown in Figure 1.

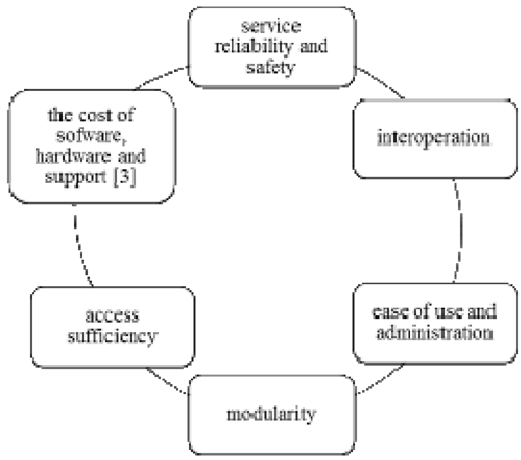

Figure 1 – The main criteria of automated information system

For the first time the application of software content management system with advanced set of tools of information and training Web-portals, which contains software means for department scientific and technical activity management has been developed during conducting this research. The overview of existing information resources for creating the key model features of the program module for scientific and technical activities of the department, an analysis of the existing services (Figure 2) has been carried out and their features and disadvantages [4, 5] have been determined.

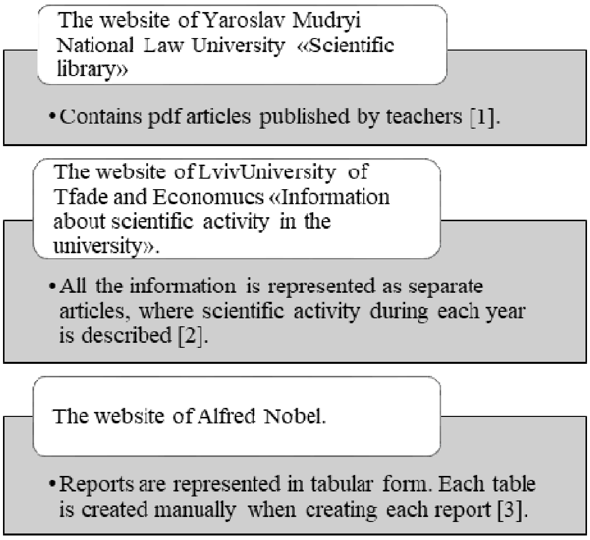

Figure 2 – The examples of existing services

At the stage of the information system designing, the main sections of the technical assignment have been defined (Figure 3).

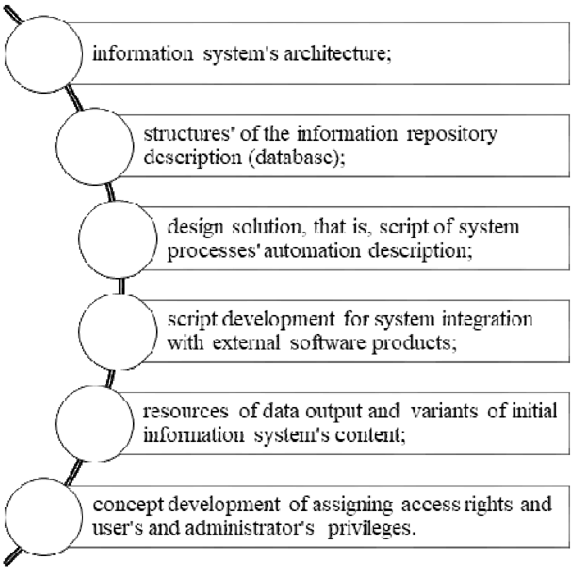

Figure 3 – Main sections of technical specification

During system development meeting the requirements of the information system implementation as a Web-application has been anticipated. The content management system should provide the administrator with the ability to perform the actions shown in Fig. 4.
An important function of interface design is the usage prediction of a specific function, the description of the function itself and the error messages processing, the search for the necessary information.

As designing of useful and spacious information is an important thing for the user, and then it should be evaluated at the same level as the system architecture or program code. Messages design takes a lot of time and effort.

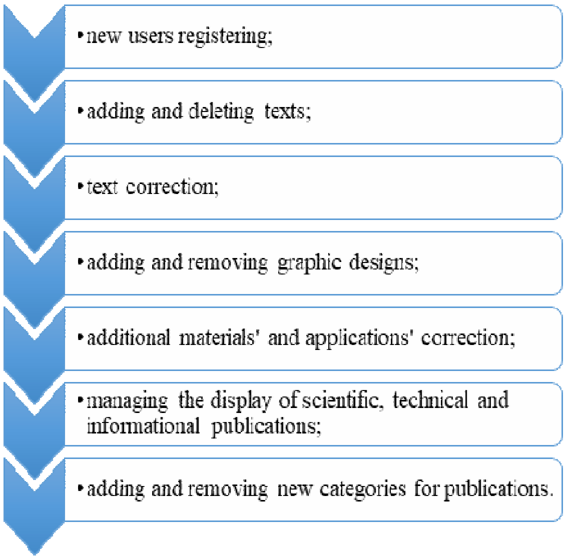

Figure 4 – Administrator functions

Error massages handling is one of the means of user training to work with the system. Having accidentally executed a certain error action, the user should receive a message in an intuitive form, what exactly the user did wrong and what to do to fix this error.

Searching for information is a mean of providing the user with the information in a large amount of other information. Therefore, this function should be provided in the system interface in a convenient place. An example of its usage in the system is the form of logging into the system, having entered data, the user receives the corresponding message and the hyperlink with the help of which the problem can be solved.

To accomplish the assigned task, a convenient module structure has been designed and developed, which is presented in Figure 5. Technical activities of the department, UML models were used.

For more precise presentation of the projected software, it is necessary to make a general model diagram of precedents for further expansion (Use Case diagram).

The precedent diagram visually reflects a variety of interaction scripts between actors (users) and precedents (cases of use); describes the system functional aspects (business logic).

The module for managing the scientific and technical activities of the department for the information intellectual system «Portal-Department» is implemented in the language of PHP-7, MySQL, Open Server, CMS Wordpress [14]. The number of users can vary from 1 to 100 (depending on the number of teachers or scholars who are registered on the website). The peak of activity on the server occurs during end-of-term exams. Therefore, in order to determine the hardware configuration, the peak traffic on the information system was determined. Also, such parameters as the number of hardware cores (processors), the amount of RAM, the required traffic-carrying capacity of the network channel.

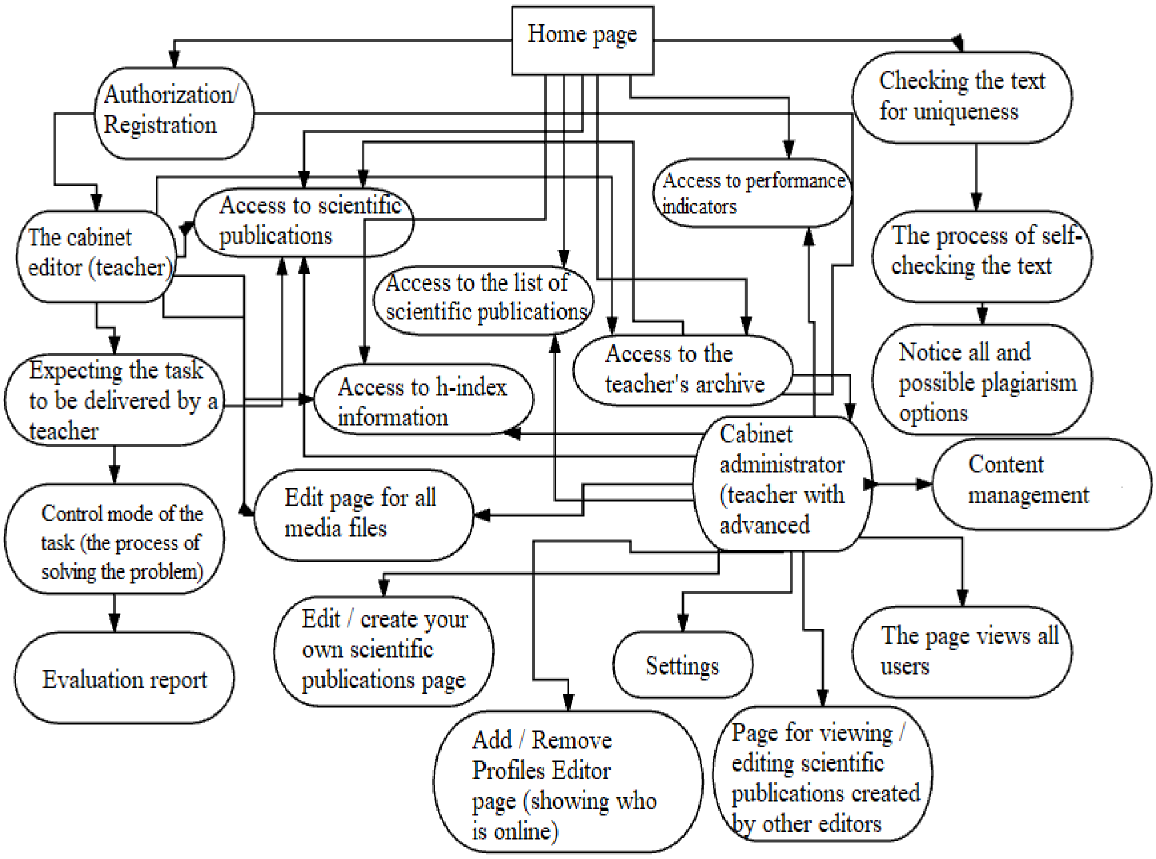

Figure 5 – The automated system structure

For more detailed module design and to simplify the calculations, the following simplifications are introduced (Figure 6).

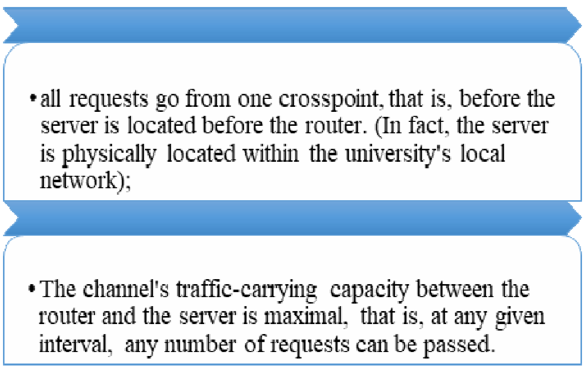

Figure 6 – Simplification for calculations

According to this, the minimum server configuration was calculated [10]: the number of cores - 4-8, 8 GB of RAM, 2-10 Mb/s traffic-carrying capacity, web server Apache 32 MB, DBMS-MySQL 32 MB.

Reliable operation of the software complex is provided by (Figure 7).

The diagram of the system usage options is shown in Figure 8.

For the usage variants the following notions are used:

-- subject as an external entity, interacting with the system; it can be a man, a device or another system;

-- usage aspect as a mean provided by the system;

-- one-sided association, as an interaction, directed from one subject or aspect to another;

-- generalization from one subject or aspect to another.

To display the series of objects and the messages which they exchange with each other, the interaction diagrams were made.

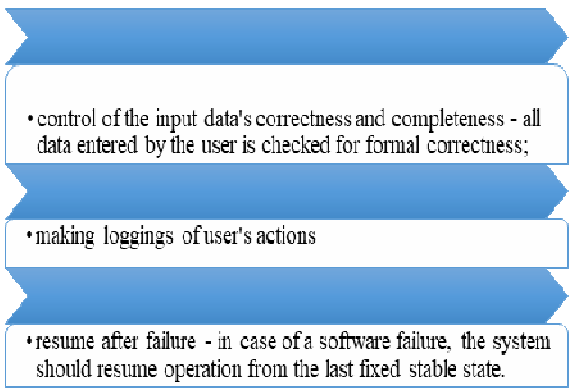

Figure 7 – Ensuring the reliability of the software system

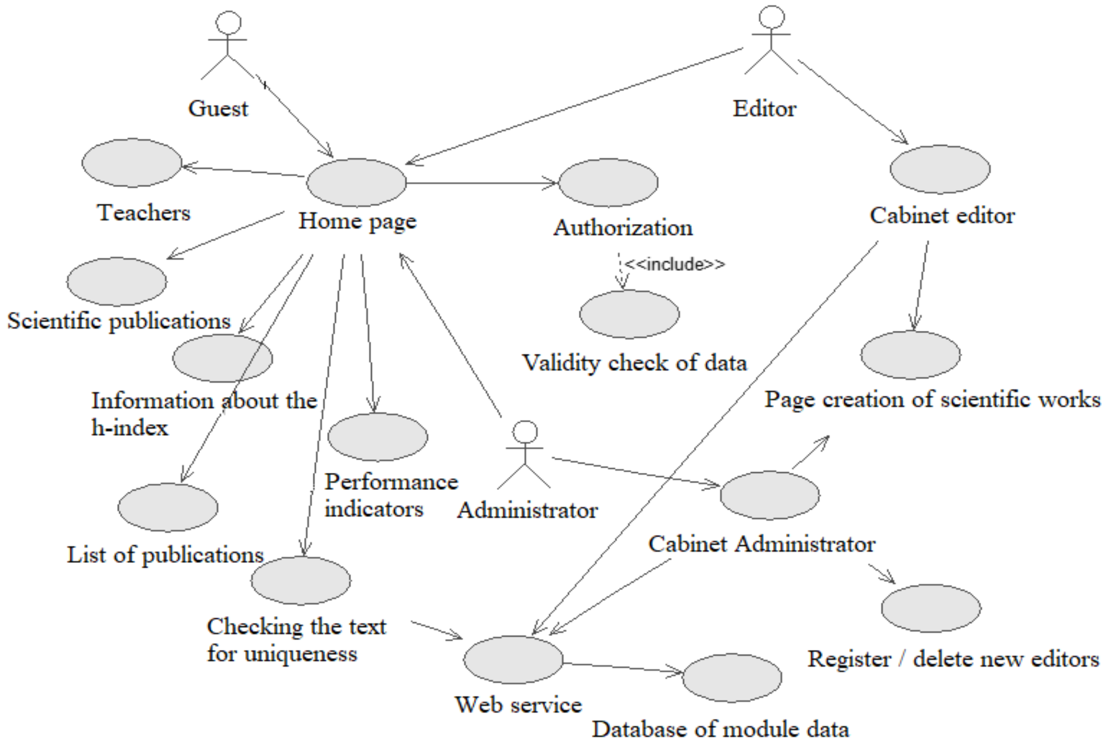

Figure 8 – The Use Case Diagram

Activity diagram is a diagram that shows the decomposition of some activity into its constituent parts. Figure 9 shows an activity diagram for the administrator's website.

The presented system of the department of structures from metal, wood and plastics is aimed at teachers and scientists, students and ordinary users. The purpose of its creation is to provide management of the department scientific and technical activities, which greatly facilitates the work of the above-mentioned users. During the software product development, modern web-technologies have been selected and substantiated, which enable to create interactive web- pages.

During an information system database designing the ER-model (entity-relationship diagram) has been developed, which enables to describe conceptual diagrams with the help of general block constructions. On its basis the data diagram has been made (Fig. 10) [11, 12].

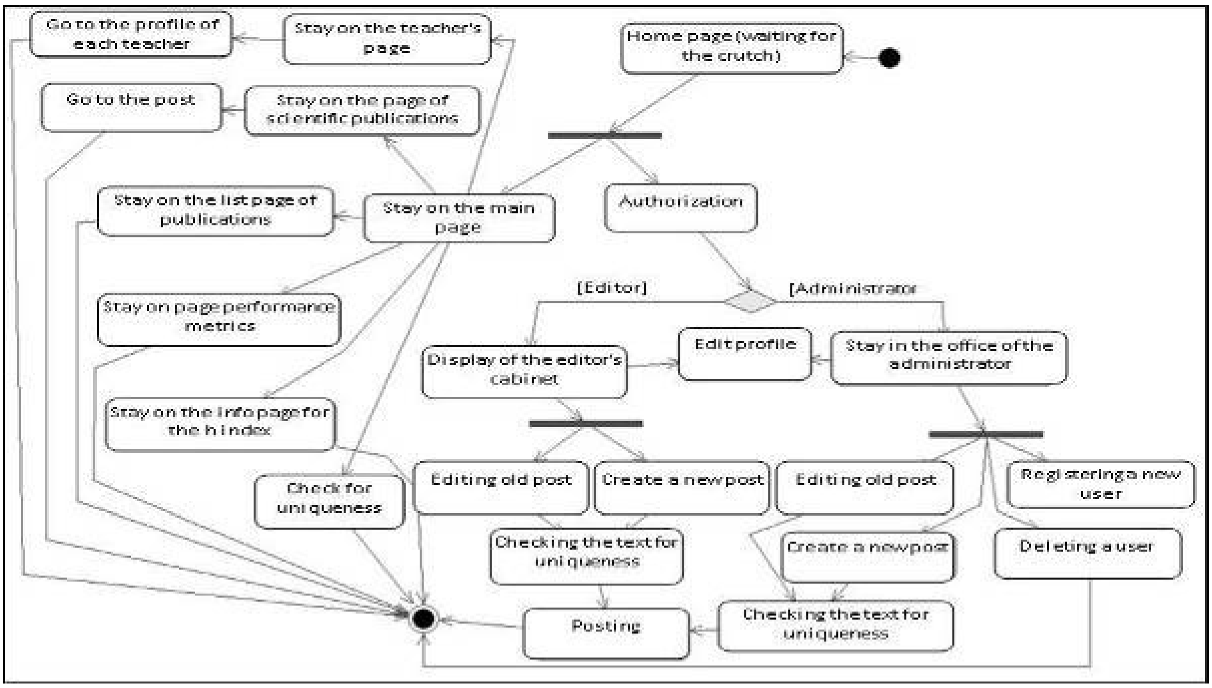

Figure 9 – Activity Diagram

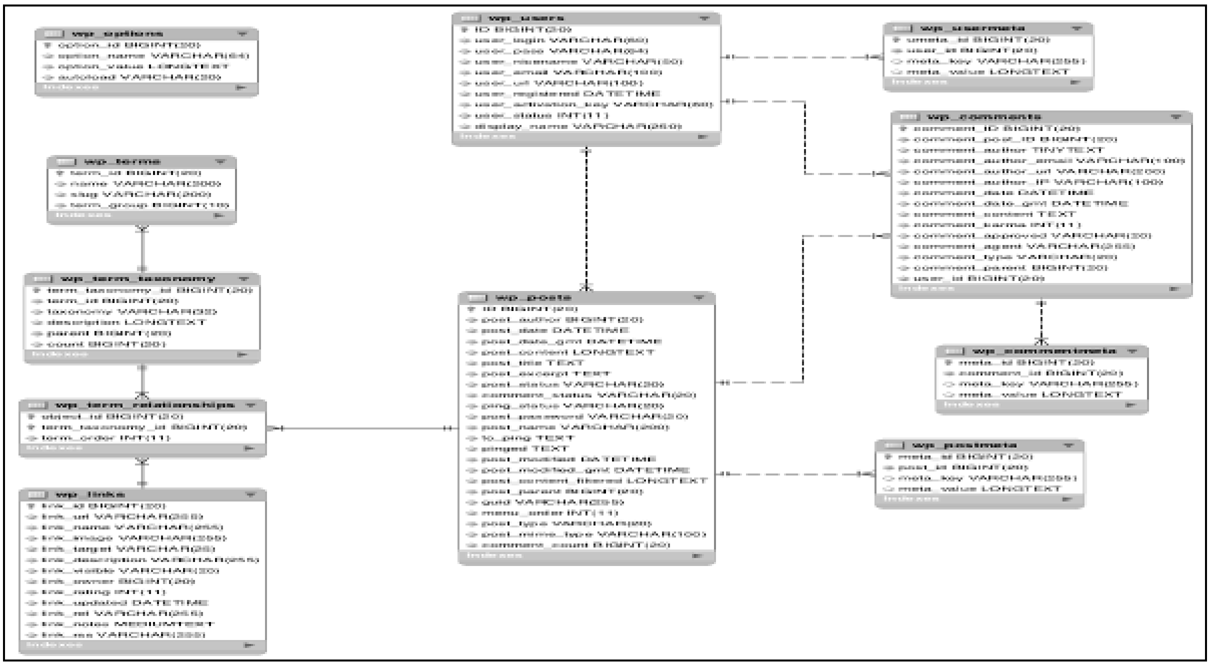

Figure 10 – The «intent -link» model

Conclusion

The result of the research was the development and program implementation of the module for the department of structures from metal, wood and plastics scientific and technical activities management.

The disadvantages of existing information management systems have been considered in this system, namely it provides scientific materials publication, the checking possibility of the text for uniqueness and automatic reports generating, which greatly eases the teacher's or scientist's work.

Information system provides the implementation of the following functions:

1. Division of the user and administrative part of the program:
   1.1. Editor (can add / delete / edit scientific publications, check the text for uniqueness, available to teachers or scientists who have been registered by the administrator);
   1.2. Administrator (can add / delete / edit scientific publications, check the text for uniqueness, and has the rights to add / remove new users, namely editors);
   1.3. Ordinary users (they can view publications, reports on scientific, technical and innovative activities of the department)
   1.4. Registered teacher or scientist should have the possibility of free access to the management module of scientific and technical activities using individual unique logins and passwords. When logged in, he/she accesses scientific publications (where he/she can add a new publication, with the ability to add a co-author, select the appropriate categories and other details);
2. Automatic generation of reports scientific and technical and innovative activities;
3. Providing development or addition of new publications and their categories;
   3.1. Status changes of published articles - private status (the publication dol not appear on the website, even if it has been published previously) or public status (the publication is displayed on the website, even if it has not been published previously).
4. An administrator's ability to clean the database from irrelevant data
5. Information presence of the department on the Internet, availability of reports on scientific and technical and innovative activities and scientific materials;
6. Simplicity and comfort of navigation, that is, an intuitive interface.

Among the possibilities of this intellectual module, the following can be highlighted:

- automatic generation of reports on scientific, technical and innovative activities;
- checking the text for uniqueness.
- automatic publication of the article at the specified date and time.

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Optimization of shopping center construction under organizational and financial constraints

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The article presents the results of optimizing the construction duration of the object and the average monthly financing intensity of the shopping center construction under organizational and financial constraints, namely: the work processes alignment – 68-76%; the maximum monthly financing intensity – 40 million UAN; number of work brigades – 1-2; the maximum construction duration – 360 days. There were developed the method for optimizing and the results of numerical modeling of organizational and financial decisions were obtained. The most efficient models of construction were identified by graphical way under limited conditions of their implementation.

Keywords: organization of construction, civil engineering, trade and entertainment center, the duration, the intensity of the funding.

Оптимізація моделі будівництва торгово-розважального центру при організаційно-фінансових обмеженнях

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Представлено результати оптимізації тривалості зведення об'єкта і середньомісячної інтенсивності фінансування будівництва торговельно-розважального центру в умовах організаційно-фінансових обмежень, а саме: сумісність процесів – 68-76%; максимальна місячна інтенсивність фінансування – 40 млн. грн.; кількість бригад – 1-2; максимальна тривалість робіт – 360 днів. Аналіз інформаційних джерел показав, що умови житлового будівництва надзвичайно мінливі, тому важливо дослідити вплив зміни організаційних рішень на основні показники, насамперед на тривалість будівництва та інтенсивність фінансування. Розроблено методику оптимізації організаційних рішень будівництва торговельно-розважального центру із використанням сучасних програмних продуктів з галузі управління проектами. Шляхом організаційного моделювання у програмі MS Project та економіко-математичного моделювання у пакеті MS Excel побудовані достовірні моделі процесу будівництва. Згідно плану експерименту зафіксовані значення наступних показників: тривалість виконання будівельно-монтажних робіт, максимальна місячна інтенсивність фінансування проекту та середньомісячна інтенсивність фінансування будівельних робіт.Для подальших досліджень була вибрана поліноміальна модель другого ступеню, що відповідає плану експериментів. На цій основі побудовані експериментально-статистичні моделі зміни показників від факторів, що варіюються: інтенсивність використання робочого часу, кількість робочих бригад та суміщення робіт. Графічним способом визначені найбільш ефективні моделі будівництва в обмежених умовах їх реалізації: «тривалість виконання будівельно-монтажних робіт» дорівнює 244 дні (60 робочих годин у тиждень, 2 робочі бригади, суміщення робіт 68%); «середньомісячна інтенсивність фінансування» дорівнює 15000 тис. грн. (80 робочих годин на тиждень, 1 робоча бригада, суміщення робіт 68%).

Ключові слова: організація будівництва, цивільне будівництво, торговельно-розважальний центр, тривалість, інтенсивність фінансування.

Introduction

The volume of civil construction in Ukraine was increased by 3.4 times (from 19 659,1 million UAN to 66 791,6 million UAN) for the period 2010-18 years. At the same time, the conditions of the civil construction are more complex than other types of construction for two main reasons: complicated engineering facilities as well as the instability of the financial situation at the macro and microeconomic levels. In the examination of the regulatory and reference literature comprehensive systematic recommendations for the choice of organizational and financial decisions on the topic has not been found. Research topic is extremely relevant, given the high social, economic and technical effect of solving the problem of rational selection of organizational solutions for civil construction.

Review of research sources and publications

Now there are 144 shopping centers in Ukraine with the corresponding lease area of 2.5 million m^2 by ICSC standards [1]. ICSC Ukraine Research Group has identified a concept of "shopping center" – an object of commercial real estate, which is planned, built and operated as a single entity, including shops and gross leasable area (GLA) of not less than 5000 m^2. According to the study, most of the retail space in the largest cities of Ukraine is presented in the format of "traditional/large" (27.9% of gross leasable area of shopping centers), "traditional/average" (23.2%), and the "traditional/small/with day to day – trade dominant" (24.7%). Another 15.8% have a format of "specialized/thematic center/entertainment without dominant" [2-3]. Shopping centers` market development has its own logic, and from year to year it is becoming more diverse. Under these conditions, the study of organizational and financial decisions of new shopping centers construction is relevant [4].

Analysis of works devoted to the optimization of organizational and technological solutions of construction and reconstruction [5-7] leads to the conclusion that the use of experimental statistical modeling is an effective way to solve such problems and can be used for modeling and optimization of operating activity of the construction companies. The papers [8-13] are devoted to optimization techniques applied by experimental statistical modeling. It is expedient [5-7] to create operating models of construction enterprise using specialized software for project management.

Definition of unsolved aspects of the problem

There is not considered the combined effect of a variety of organizational factors on the performance of the construction project in numerous studies aimed at the selection of efficient organizational and technological solutions. The study task is proposed to be solved by numerical simulation of construction processes and optimization of organizational solutions considering existing constraints, using modern software, theory of experiment planning and statistical processing of experimental results.

Problem statement

Purpose of the article is to optimize the construction duration and the average monthly financing intensity of the shopping center construction under the organizational and financial constraints. The following tasks were set to achieve this goal:

– Development of optimization method of indicators of shopping center construction project.

– Construction of experimental statistical dependencies of construction duration and average monthly financing intensity from the working time use intensity, the number of work brigades and the work processes alignment.

– Analysis and graphic interpretation of the numerical experiment`s results.

Basic material and results

There was proposed to use experimental statistical modeling for effectiveness evaluation of the organizational solutions of the shopping center construction. The essence of this simulation is to monitor the system under consideration by fixing the values of the outgoing parameters when specifying input values parameters. Thus, in the present study, the system is represented as a time schedule. Experimental statistical modeling algorithm is shown on Fig. 1.

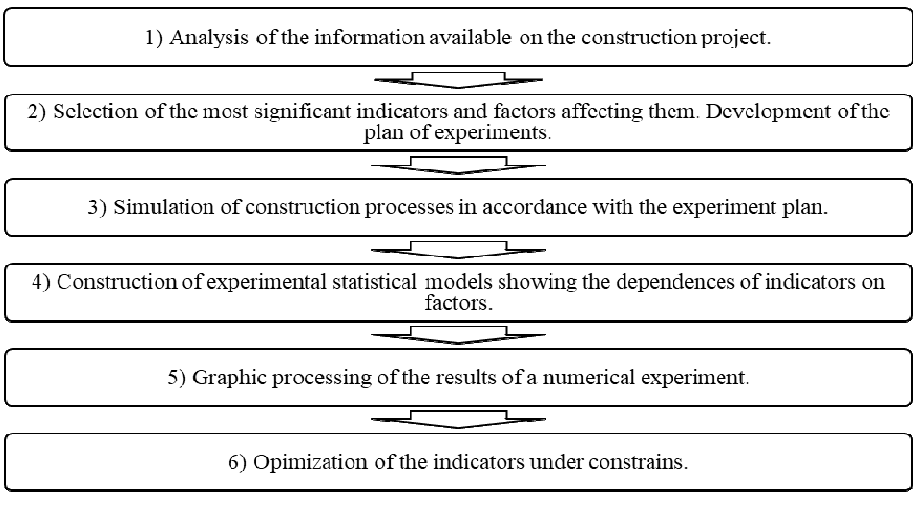

Figure 1 – Research algorithm

The key indicators are as follows:

– Y_1 – construction duration – the number of calendar time from the start of the first work until the end of the last work at all sections considering the schedule of construction works.

– Y_2 – maximum monthly financing intensity – the maximum amount of the monthly financing for the entire period of construction. It is defined as follows: construction of the work schedule with a cash distribution equal to month period; the resulting financing schedule is analyzed and month with a maximum funding is selected.

– Y_3 – average monthly financing intensity – the funds allocated for the construction of a facility are deleted on the duration of the construction work, expressed in months.

The selected indicators are most affected by the following factors:

– X_1 – working time use intensity – there was provided the following in the development of the experimental design: 40, 60, 80 hours per week;

– X_2 – number of work brigades –there was considered embodiment of the workflow involving 1, 2 or 3 brigades simultaneously;

– X_3 – work processes alignment – the ratio of the length of the construction period T_c to the total value of the working time of all the processes on all work sections $\Sigma^N{}_1\,\Sigma^N{}_1\,t_1$ (formula 1).

The transition to the coded factor levels was performed according to the standard formula 2, where:

– x_i – predetermined level of factor in its normalized form;

– X_i – predetermined level of factor in its natural form;

– $X_{i\ max}$ – maximum level of factor in its natural form;

– $X_{i\ min}$ – minimum level of factor in its natural form.

The polynomial experimental statistical model was selected to solve the problems of the present study. It is general form presented in formula 3. Numerical results of the experiment are shown in table 1. The results of the experimental statistical models calculation for the selected indicators are shown in formulas 4-6.

One of the tasks set by the customer was to determine the minimum duration of the construction work. The following restrictions were imposed while solving this problem:

– Work processes alignment – 68-76%;

– Maximum monthly financing intensity – 40 million UAN

$$K_s = \frac{T_c}{\Sigma^N{}_1\Sigma^n{}_1 t_i} - 1\ ; \tag{1}$$

$$x_i = \frac{X_i - \dfrac{X_{i\cdot\max} + X_{i\cdot\min}}{2}}{\dfrac{X_{i\cdot\max} + X_{i\cdot\min}}{2}}\ . \tag{2}$$

The restrictions are shown on the diagram by shading of following isosurfaces of construction duration and maximum monthly financing intensity values (Fig. 2). It enables to analyze these restrictions.

The effective value of the indicator "construction duration", equal to $C_{1\ \mathrm{limit}}$ = 244 days, was found after examining the diagram with restrictions. This model is possible at: X_1 = 60 hours per week, X_2 = 2 working brigades, X_3 = 68%. The indicator reduces by the increasing of the working time use intensity (X_1), the number of work brigades (X_2) and the work processes alignment (X_3).

$$\begin{aligned} Y_i = b_0 &+ b_1 X_1 + b_{11} X_1^2 + b_{12} X_1 X_2 + b_{13} X_1 X_3 \\ &+ b_2 X_2 + b_{22} X_2^2 + b_{23} X_2 X_3 \\ &+ b_3 X_3 + b_{33} X_3^2 \end{aligned} \tag{3}$$

$$\begin{aligned} Y_1 = 243{,}64 &- 73{,}3\, X_1 + 35{,}94\, X_1^2 + 47\, X_1 X_2 + 19{,}5\, X_1 X_3 \\ &+ 121{,}3\, X_2 + 57{,}94\, X_2^2 + 31\, X_2 X_3 \\ &+ 63{,}8\, X_3 + 9{,}56\, X_3^2 \end{aligned} \tag{4}$$

$$\begin{aligned} Y_2 = 36692{,}83 &+ 5619{,}81\, X_1 - 2047{,}95\, X_1^2 + 618{,}65\, X_1 X_2 + 1407{,}68\, X_1 X_3 \\ &+ 11427{,}04\, X_2 - 1998{,}13\, X_2^2 + 2043{,}24\, X_2 X_3 \\ &+ 6450{,}97\, X_3 + 209{,}38\, X_3^2 \end{aligned} \tag{5}$$

$$\begin{aligned} Y_3 = 20377{,}52 &+ 3106{,}2\, X_1 - 1812{,}37\, X_1^2 - 282{,}55\, X_1 X_2 + 23{,}64\, X_1 X_3 \\ &+ 6104{,}18\, X_2 - 1470{,}06\, X_2^2 + 438{,}58\, X_2 X_3 \\ &+ 3227{,}25\, X_3 + 1575{,}09\, X_3^2 \end{aligned} \tag{6}$$

Table 1 – The results of a numerical experiment

#	Working time use intensity, hours a week (X_1)	Number of work brigades, (X_2)	Work processes alignment, % (X_3)	Construction duration, days, (Y_1)	Maximum monthly financing intensity, thsd. UAH, (Y_2)	Average monthly financing intensity, thsd. UAH, (Y_3)
1	40	1	61	710	15 171, 713	7247, 944
2	40	1	76	445	20 788 647	11 487 308
3	40	3	61	278	32 012, 947	18 039 328
4	80	1	61	395	21 591 218	12 817, 417
5	40	3	76	190	43 728 903	25 907, 546
6	80	1	76	261	30 764, 955	19 025, 854
7	80	3	61	204	38 833 149	24 353, 094
8	80	3	76	141	58 253 785	30 441 367
9	80	2	68	224	44 725 436	22 139 176
10	40	2	68	335	26 268 165	15 032 774
11	60	3	68	194	50 425 694	25 367 806
12	60	1	68	409	20 667, 557	12 488, 766
13	60	2	76	190	47 045 372	25 907, 546
14	60	2	61	278	28 462 916	18 039 328
15	60	2	68	244	33 285 173	20 294 245

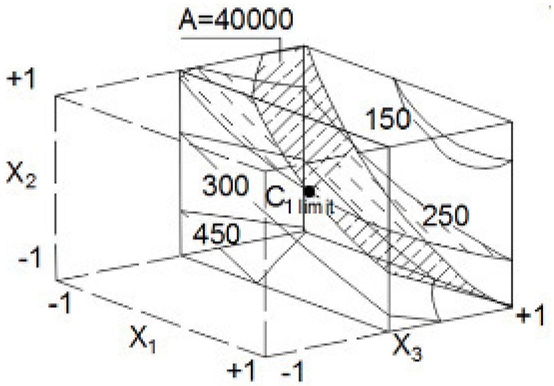

Figure 2 – Optimization of the construction duration under limitations of the work processes alignment and the maximum monthly financing intensity

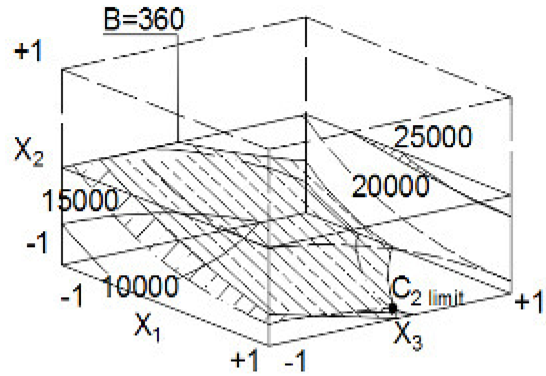

Figure 3 – Optimization of the average monthly financing intensity under limitations of the number of work brigades and the maximum construction duration

Next task, set by the customer, was to determine the minimum of the average monthly financing intensity. The restrictions for this task were:

– the number of work brigades – 1-2;

– the maximum construction duration – 360 working days.

These restrictions were shown on the diagram of the average monthly financing intensity (Fig. 3).

The minimal value of the indicator "average monthly financing intensity", equal to $C_{2\ \text{limit}}$ = 15,000 thsd. UAN was found after considering the limitations. This model is available when X_1 = 80 hours per week, X_2 = 1 operating brigade, X_3 = 68%. The average monthly financing intensity reduces with increasing levels of working time use intensity (X_1), the number of work brigades (X_2) and the work processes alignment (X_3).

Conclusions

1. The developed methodology and the obtained results confirm the possibility of using the proposed approach to the optimization of the construction duration and average monthly financing intensity for the facilities under consideration.

2. The efficient construction model into the investigated range of the factors has the following parameters: duration of construction – 244 days, the maximum monthly financing intensity – 40 million UAN, the average monthly financing intensity – 15 million UAN. These rates are achieved at 60 working hours a week, using two working brigades, with work processes alignment equal to 68% under constraints (work processes alignment – 68-76%; maximum monthly financing intensity – 40 million USD; number of working brigades no more than 2; construction duration – 360 working days).

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Oil and gas complex of Ukraine: analysis and prevention of electrical traumatism

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The work is devoted to the analysis of injury cases from the electric current influence on the oil and gas complex objects. The principles, the main provisions of the system analysis and the classification of electric traumatism cases at power enterprises are presented. Main causes and patterns of the electric trauma occurrence on dependence of the various production features, as well as for various categories of workers are determined in the article. The structural scheme of the relationship between the elements of the system "man – electrical engineering – production environment" is given. A set of preventive measures and practical recommendations that can be successfully applied to assess occupational risk and reduce the level of injury from electric current in the energy sector of Ukraine is proposed.

Keywords: electrical safety, traumatism, electrical traumatism, industry, oil and gas complex, safety management.

Нафтогазовий комплекс України: аналіз і профілактика електротравматизму

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Проведено аналіз випадків травматизму від дії електричного струму на об'єктах нафтогазового комплексу України. У статті представлені принципи, основні положення системного аналізу і класифікація випадків електротравматизму на енергетичних підприємствах та визначені основні причини і закономірності виникнення електротравм у залежності від різноманітних особливостей виробництва, а також для різних категорій робітників. Виявлені основні організаційно-технічні, технічні, організаційні, санітарно-гігієнічні та психофізичні причини виникнення нещасних випадків і професійних захворювань у робітників нафтогазового комплексу. Системний аналіз випадків електротравматизму дозволив сформулювати основні принципи системного підходу до проблеми наукового обґрунтування системи управління електробезпекою на промислових підприємствах. Розглянуті основні етапи розробки, погодження, затвердження і функціонування системи управління електробезпеко. Приведена структурна схема взаємозв'язку між елементами системи «людина – електротехніка – виробниче середовище» і запропоновано комплекс профілактичних заходів і практичних рекомендацій які можуть бути успішно застосовані для оцінювання професійного ризику та зменшення рівня травматизму від дії електричного струму в енергетичному секторі України. Запропонована методологія аналізу та оцінки ризиків аварій на об'єктах нафтової та газової промисловості, що дозволяє порівнювати дію небезпечних чинників різної природи, визначати, з урахуванням вкладу кожного окремого фактора, інтегральну ступінь небезпеки будь-якого промислового об'єкта. Застосування даної методології оцінки ризику дає можливість розробляти механізми і стратегії різних заходів щодо підвищення безпеки об'єктів нафтової і газової промисловості; встановлювати межі величин ризику і невизначеностей, пов'язаних з обмеженістю вихідних даних або з невирішеністю наукових проблем.

Ключові слова: електробезпека, травматизм, електротравматизм, промисловість нафтогазовий комплекс, управління безпекою.

Introduction

Every year, the electricity production and consumption increase, thus, the number of people, who use (operate) electric devices and installations during the life-cycle of their lives, increases too. According to [1], the approximate forecast for electricity production by 2035 year can be 195 billion kWh that is 31.3 billion kWh more than in 2015 year (163.7 billion kWh), so the issues of electrical safety are on particular importance.

Exploitation of electrical equipment by the personnel is associated with the electric shock risk, as well as with the probability of electric injuries among technicians who do not connected directly with the operation of electrical installations. The risk of electric shock is significant increase during the direct conduct of repair, prophylactic, start-up and commissioning works with the voltage removal near the current-operated parts or without the voltage removal from electrical installation by electrical personnel. Therefore, the main task is to reduce the electric trauma in all spheres of production activity.

The analysis of industrial traumatism shows that the number of injuries caused by the action of electric current is about 1% of the total number of the injured persons. However, accidents with a lethal consequence of the electric current are equal up to 7% and occupy one of the first places in the industry. The largest number of injuries, including 80-85%, with fatal consequences, occurs when using electrical installations up to 1000 V, due to their distribution and relative availability for practically anyone working at the production site. Accidents during operation of electrical installations with a voltage over 1000 V are rare, which is due to the insignificant proliferation of such electrical installations and their servicing by highly skilled personnel.

Consequently, the questions devoted to establishing the main causes and patterns of the electric traumas occurrence, depending on the characteristics of industrial production, as well as the development of preventive measures, are definitely relevant.

Review of research sources and publications

Many publications are devoted to the issues of industrial objects electrical equipment safe usage [2, 3]. These publications provide methods for the safe usage of machinery and equipment. The increase of technological processes with the usage of electric current causes the need for continuous monitoring of the electric current influence on the workers body and the possible negative effects of this effect [4 -- 7]. However, a general analysis of the causes of electrical injuries on industrial objects and the oil and gas complex objects is not given.

The conducted analysis of recent researches and publications showed that today there is no single methodology that determines the principles of electrical safety management and the procedure for assessing the risk of electric trauma during the performance of technological operations in the extraction and transportation of hydrocarbons. Famous methods for assessing the electrical safety level are based on a comparison of the measured design values of the contact voltage, current strength passing through the human body, and the time of their action with the normalized parameters or on the methods of analysis of the electrical traumas statistical data without taking into account the electric trauma probable nature and the permissible level value of the electric energy absorbed by the worker body.

Definition of unsolved aspects of the problem

The limited information does not enable to implement effective measures on electrical safety in industry. Therefore, systematization and elimination of serious shortcomings in the existing analysis determines the relevance of research in this direction.

Problem statement

Considered the above mentioned, the purposes of this article are to systematize and classify cases of electrical injuries at the objects of the oil and gas complex, to identify the main causes of workers injury, to develop the main directions of preventive measures, and to improve the control and management of the electrical safety system for construction organizations workers, taking into account the combined approach of professional assessment risk. The choice of a certain alternative between ensuring the continuity of the production process and achieving the necessary level of electrical equipment operation is followed by the development and implementation of innovative technologies in the field of electrical safety production management.

Basic material and results

Modern construction site can not be imagined without mechanisms and mechanized tools, which are driven by electric current. Widespread of the electrical current was obtained by heating the concrete, stone masonry, finishing works, defrosting of the ground, as well as lighting the construction worksite.

Violation of electrical safety rules when using machines and mechanisms, that is, direct contact with the conductive parts of the electrical equipment, which is under voltage, creates the danger of human damage by electric current. The energy sector is ranked second by the injuries among all types of economic activity.

An analysis of the industrial safety state in Ukraine shows that in 2016, the number of industrial accidents increased by 4% compared to last year, or by 168 accidents (at the enterprises of Ukraine in 2016, 4428 people were injured, in 2015 - 4260 people). The number of fatal accidents involving the production increased by 7%, or by 25 accidents, compared to the same period last year (400 people were fatal in enterprises in Ukraine in 2016, 375 people in 2015). Electric traumas accounted for approximately 15.6% of the total number of accidents at enterprises of the oil and gas complex. This is the second rank in the number of electric traumas after the agro-industrial complex (27.7%) [8 – 10].

Electrical injury accidents among workers of different professions show that workers of non-electrical occupations are injured 6.2 times less frequently than electric ones. But the frequency of electrical traumas among the workers of some non-electric professions is very high.

For example, the frequency of electrical injuries is higher for locksmiths and mechanics. For electric locksmiths, this frequency is lower, although according to the nature of their activities, the former are much less likely to deal with electrical installations than the latter. A similar inconsistency is observed when comparing the data from electrical injuries to electric welders and drivers of motor vehicles and mechanics of various aggregates.

The main causes of electric shock in industrial enterprises are:

– accidental contact with conductive parts that are under voltage;

– false actions during work or malfunction of protective devices, through which the victim has been touched by the conductive parts;

– the appearance of voltage on the metal structural parts of the electrical equipment due to damage of the conductive parts insulation;

– the short-circuit of the network phase to the ground, the voltage wire drop on the structural elements of the equipment;

– the appearance of voltage on the switched-off conductive parts due to the incorrect activation of the device or the short-circuit.

Data analysis shows that the maximum frequency of electric traumatic accidents comes on workers aged 26 ... 40 and young workers aged 18-25 years (Fig. 1).

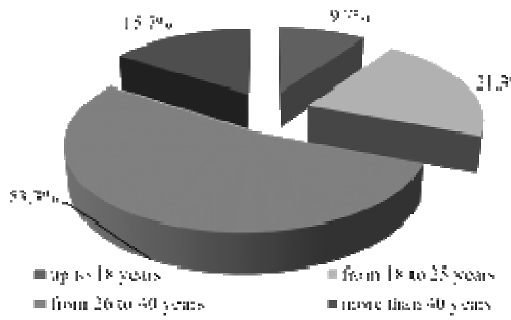

Figure 1 – Electrical traumatism at workers of all ages in % of the number of electric traumas for all types of economic activity

It is suggested that workers in these age groups are either lacking in certain experience or caution and accountability, which is typical for older people, confirming injury data, depending on the production experience (Fig. 2).

Workers with a work experience of up to 5 years have a frequency of electrical traumatism 2 times more than workers with a work experience of 5 ... 10 years, and 5 times more than workers with a work experience of more than 10 years. Accordingly, young and inexperienced workers should be constantly at the attention center of persons who are responsible for the safety work.

The analysis of electric traumatism cases enabled to reveal the main organizationally-technical, organizational, sanitary-hygienic and psychophysical causes of accidents at the workers of the oil and gas complex.

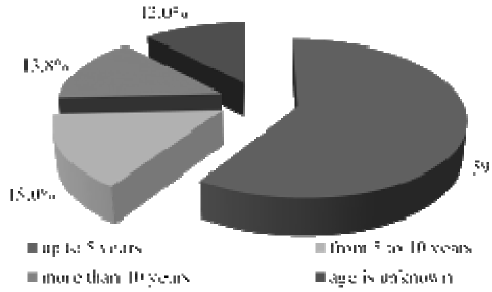

Figure 2 – Electrical traumatism at workers of different production experience in % of the number of electric traumas for all types of economic activity

For sanitary-hygienic causes include: increased content of harmful substances; insufficient or irrational lighting; increased levels of noise, vibration, infra- and ultrasound; unsatisfactory microclimatic conditions; the presence of various radiation above the permissible levels, etc.

Psychophysical causes include the following factors: false actions of workers due to fatigue, illness, carelessness; discrepancy between the psychophysical and anthropological data of the employee to the performed work.

Distribution of organizationally-technical and technical reasons is shown on Figure 3.

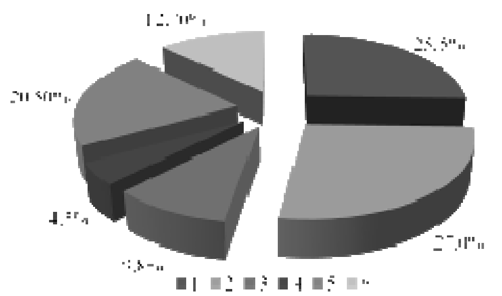

Figure 3 – Electrical traumatism from organizationally-technical and technical causes in % of total electric trauma:

1 – forbidden work or non-use of precautionary technical measures when working under voltage;
2 – violation of the air lines protection zone, transportation of oversize cargoes, non-use of fences and locks, mechanical damage to the insulation;
3 – use of inappropriate voltage, incorrect inclusion, switching, inclusion of zero conductors per phase;
4 – convergence of conductors of the included and off network, phase and zero conductors of one line, breakage or sagging of wires;
5 – defects of the structure and installation of electrical equipment; 6 short circuit to the frame, the short circuit between the electrical windings of transformers, the inverse transformation.

The organizational causes of electric trauma can be attributed: work without admission or job-order, or improperly executed job-order; absence or irregularity of the safety briefing; absence or non-use of protective equipment; mismatch of accident preventions qualification to the performed work; illegal combining of professions; work in extra-ordinary time; unsatisfactory work organization of personnel on a business trip; wrong or prohibitive actions of work performers; violation of discipline.

As it can be seen from the above data, the usage of protective equipment only by the enterprises provided by the Rules for the installation of electrical installations (RIEI) can not create conditions for complete safety during the installation, operation and repair of equipment. It is possible only when these measures are supplemented by other organizational measures (instructions, studying, verification of knowledge, etc.) and take into account: voltage of electrical installations, neutral regime, environment conditions, etc.

It is also necessary to consider insufficient level of safe organization of labor during the operation of equipment, machinery, mechanisms, vehicles, violation of the technological process, poor condition (deterioration) of production facilities, structures, and production goods.

Based on the analysis of the causes and cases of electrical traumatism, it should be noted that the problem of electrical safety and prevention of electrical traumatism refers to mixed problems, which consist of both qualitative elements and less well-known, which are of a random nature. It is suggested that solving the problem of providing electrical safety at the enterprises of the oil and gas industry should be carried out on the basis of system analysis and development of the electrical safety management system.

Since the main parameters of electrical safety are regulated by rules and standards, within the framework of the management system, proposed complex and technical measures aimed at ensuring the electrical safety, is preventive, should be additional to the normative work regulations and should not conflict with the current normatively-legal acts on supervision and control of work safety. The operation of the control system must be coordinated with the energy services that carry out direct operation, technical maintenance, repair of electrical installations and authorized by the relevant energy monitoring services.

The direct management of measures and decisions on the use in operational practice of the electrical safety management system is carried out by the Chief Mechanical Engineer Service (Power Engineer) of the enterprise. Feasibility report on the need for implementation of control technical means, effective means of protective shutdown, information and analytical system for managing the database on electrical safety, training of qualified specialists and technical staff studying for protection systems maintenance is carried out at the same time. The prepared normative and technical documentation is approved by the management of the enterprise.

The management of the measures and decisions implementation on the management system accomplishment is carried out in accordance with the approved stage-by-stage program under the direct supervision of the responsible person (Chief Power Engineer).

The problem solution efficiency of providing electrical safety at the enterprises of the oil and gas complex is determined in the process of the equipment direct operation according to the criteria, which laid down in the basis of the management system functioning in accordance with the applicable normative and legal documents.

The system analysis of electric traumatism cases enable to formulate the basic principles of a systematic approach to the electrical safety management system scientific substantiation problem [11 – 12].

1. Electrical safety is a system of organic measures and technical facilities that protect workers of all professions from harmful and dangerous effects of electric current, electric arc, electromagnetic field and static electricity.

2. The control management system of electrical safety by structure, internal content and external communication has all the signs of a multi-level system.

3. The control system includes a set of the system elements, which are defined both within the framework of internal interconnections, and in the implementation of external links with subsystems of operation, technical maintenance and repair, technical control, etc.

4. Within the framework of a specific organizational management structure, not only analysis of the system elements is carried out, but also the establishment of interrelationships between them. In this process, the functioning of the control system is equally due to the properties of its individual elements, as well as the properties of the structure itself.

5. The description of the system separate elements and their interrelation within the framework of the current electrical safety control system is carried out by many models: physical, mathematical, cybernetic, statistical and economic.

The system approach to the problem of electrical safety involves the studying and practical usage of the following aspects:

1. System-element or system-complex aspect involves the identification of elements that make up the system of electrical safety, taking into account the specifics, technological processes organization and management in the conditions of construction and building materials and structures manufacturing.

2. The system-structural aspect is the establishment of internal connections and interconnections between elements of the control system, which allow identifying the internal organization of the system under investigation.

3. System-functional aspect involves the establishment of functions implemented by each element of the control system.

4. The system-target aspect involves the definition of target management settings and their interconnection.

5. The system-resource aspect is to determine the resources necessary for the functioning of the management system.

6. The system-integration aspect involves the integration of the electrical safety management system into the general system of occupational safety.

In accordance with the aforementioned principles of a systematic approach, the structure of the electrical safety control system in general is presented in Fig. 4.

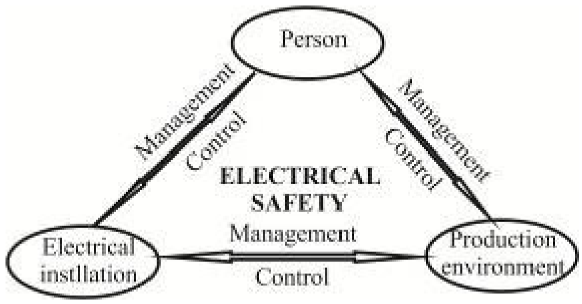

Figure 4 – The general structure of the electrical safety provision at the enterprise

The electrical safety management system has a complex internal structure, which elements are interconnected both in a hierarchical plan and at each of its levels.

The operation of the electrical safety management system should be considered directly in connection with its component subsystem "man-electrical installation-environment". This is due to the fact that the dangers of industrial electrical traumatism impairment arise precisely in this subsystem, when there is a direct contact between a person and an electrical installation. At the same time, the environment affects both the person and the electrical installation.

The reduction of the electric trauma probability in the specific conditions of the electrical equipment operation at the enterprises of the oil and gas complex through the usage of organizational and technical measures is a direct task that is solved in the electrical safety control system.

There is an interconnection between the system ensuring elements of the electrical safety at the enterprise. The definitive functioning of this subsystem, the person as a subject, affects the electrical installation by providing it with safe properties both at the stage of its design (development, creation), and directly in the process of operation. Such properties are: constructive execution, level of explosive protection, spark protection, level and class of insulation, etc. All these characteristics and electrical installations characteristics are directly connected with the operating condition that is directly connected with the environment, which specificity is related to the technology of drilling and other works.

A person performs certain operations that affect both the electrical installation and the environment. The influence on the electrical installation is determined by the instructions for operation, repair, technical maintenance and other normatively-technical and legal documents. The impact on the environment is carried out by creating and maintaining a comfortable microclimate, which are regulated by sectoral norms and rules at the enterprises of the oil and gas complex.

Electrical installation, as an object of part man influence, also has its influence. In the process of operation, aging and wear of electrical equipment (reduction of insulation resistance, mechanical wear of hanging parts, their damage, etc.) leads to decrease in reliability and safety. Electrical installation also affects the environment, as the deterioration of the protective characteristics of the electrical equipment can lead to emergency situations and electrical traumatism.

The environmental impact of the electricity management system is also interrelated with the specifics of production, governed by applicable norms and rules, has its own parameters and characteristics. The negative impact of the environment is strongly manifested in the specific field conditions of extraction, preparation and transportation of oil and gas.

Thus, the system "man-electrical installation-environment" is a complex structured subsystem of interconnected elements, which interaction in the production environment is regulated b electrical safety control system.

Since the complete damage elimination from the technological -industrial and environmental hazards associated with electrical installations in industrial enterprises engaged in exploration, extraction, processing and marketing of oil and gas is impossible, therefore, in order to minimize the risk of electric trauma for the personnel. This personnel serves electrical installations. It is necessary to choose the way of minimizing the total socio-economic costs. There is a set of measures and means of electrical safety should be chosen from the possible variety, which introduction reduces the risk of electrical traumatism in electrical safety to an acceptable level.

The implementation of a risk-based approach to management can be seen as a functioning of a system or “control system”. That is, specially developed on the basis of the system accounting, analysis, valuation, planning, control and management traditional methods integration. That provides obtaining, processing and aggregation of information about enterprise activity, continuous monitoring with the use of technical diagnostics means of electrical equipment operation regime parameters. Also it provides application of special protection means against harmful and dangerous effects of electric current, electric arc, electromagnetic field and static electricity.

For an effective controlling electrical safety system for oil and gas workers, it is proposed to use a closed information contour on the electrical energy. This contour contains a sequence of logically related control functions: the risk parameters estimation of personnel damage by electric energy; planning and implementation of planned measures on electrical safety in order to eliminate the chain of preconditions for the appearance of electrical trauma aimed to the risk minimization of electrical traumatism and professionally determined disease; control over the implementation of planned activities; assessment and analysis of electric traumas risk parameters after the taken measures; making decisions to improve the electrical safety system, which allows to constantly compare the actual state of the controlled process, in order to minimize the risk of electric traumatism.

Conclusions

The methodology of analyzing and assessing the risks of accidents at the objects of the oil and gas industry is actively developing, therefore, the development of new and improved existing approaches, models and methods for assessing the risks of accidents, their computer realization remains a topical task for our state. The determination of the accidents risk assessment should be based on the results of monitoring the technical condition of potentially hazardous objects, statistics on accidents and man-caused emergencies, integrated monitoring of dangerous geological and hydrometeorological processes, the natural systems state, as well as the results of the relevant hazardous situations simulation and their influence on public health. The application of the risk indicator enables to compare the different nature dangerous factors effect, determine, considering the contribution of each individual factor, the integral degree of any industrial object danger. The application of the risk assessment methodology enables to develop mechanisms and strategies for various regulatory measures to improve the safety of oil and gas industry objects; to set the limits of the risk and uncertainties associated with the limited source data or the unresolved scientific problems.

Summing up, it can be determined that the main preventive measures for workers of non-electric specialties are: improving their professional level, explaining the work danger with electrical equipment, systematic and qualitative safety training and control strengthening by officials in compliance with safety measures at the enterprise, obsolete equipment replacement or its modernization in accordance with modern labor safety requirements.

Vocational education is of great importance for providing electrical safety on the oil and gas enterprises. The analysis of injuries shows that individuals who have graduated from special educational institutions, including vocational and technical, are much less likely to be injured than those ones without special education.

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Formation of multifunctional nano-layered oxide REE-containing materials using nitrate precursors

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Complex system research of the interaction of structural components in nitrate systems of rare-earth and IA, IIA elements of the periodic system – precursors of modern multicomponent oxide polyfunctional materials on their basis – established the formation of a class of lanthanides alkaline coordination nitrates. The obtained data are the basis for identifying, controlling the formed phases in the preparatory stages of processing in innovative technologies using nitrate predecessors of different electronic structures elements and various combinational methods of their activation, establishing technological and functional dependencies, controlling the modification of the synthesis products properties.

Keywords: alkaline coordination nitrates of lanthanides, formation conditions, crystal structure of compounds, properties.

Формування багатофункціональних наношаруватих оксидних РЗЕ-вмісних матеріалів з використанням нітратних прекурсорів

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Комплексним системним дослідженням взаємодії структурних компонентів у системах нітратів рідкісноземельних і IA, IIA елементів періодичної системи встановлено утворення цілого класу лужних координаційних нітратів лантаноїдів, які синтезовані в монокристалічному виді. Їхній склад, атомно-кристалічну будову, форми координаційних поліедрів Ln, типи координації лігандів, ряд їхніх властивостей досліджено з використанням комплексу фізико-хімічних методів. З'ясовані об'єктивні закономірності поведінки цього типу сполук поглиблюють розуміння про: хімічні і фізичні властивості Ln, їх комплесоутворюючу здатність; можливість утворення й існування в аналогічних системах асоційованих нових фаз і їх стійкість; вплив природи лантаноїдів і лужних металів, магнію на структуру комплексних аніонів і сполук у цілому; індивідуальність Ln комплексів; існування ізотипних за складом і структурою груп сполук за природними рядами лантаноїдів і лужних металів; роль NO_3^--груп у стереохімії цього класу нітратів; роль води у формуванні найближчого оточення іонів Ln^{3+}- комплексоутворювачів. Одержані дані є основою для виявлення, ідентифікації, контролю утворюваних фаз, визначення елементного складу і вмісту проб, проведення аналізу і порівняння фазового стану об'єктів у підготовчих стадіях перероблення в інноваційних технологіях з використанням нітратних попередників елементів різної електронної структури і різними комбінованими способами їх активації, встановлення технологічно-функціональних залежностей, керованого модифікування властивостей продуктів синтезу. На перспективність цих прекурсорів указують існування достатньо представницького класу комплексних нітратів лантаноїдів, виявлення серед них ізотипних за складом і структурою груп сполук представників Y, La – Lu; Li – Cs, прояв комплексу цінних у технологічному відношенні притаманних їм властивостей.

Ключові слова: лужні координаційні нітрати лантаноїдів, умови утворення, кристалічна будова сполук, властивості.

Introduction

The report continues the discussion and results analysis of the structural components joint behavior feature study in the precursor systems of nitrate lanthanides, alkaline, alkaline earth metals in the preparatory stages during the formation of multifunctional RE-containing oxides with the structure of perovskite, garnet using the methods of soft chemistry and thermal activation.

One of the most promising classes of complex oxide materials of rare-earth elements and titanium are nanostructured layered perovskite-like compounds (the Ruddlesden-Popper phases $A'_2[A_{n-1}B_nO_{3n+1}]$ [1], Diona-Jakobson $A'[A_{n-1}B_nO_{3n+1}]$ [2], where A', A – atoms of alkaline, alkaline or rare earth metals, B – transient metal atom (most often Ti, Ta, Nb) and solid solutions on their basis. For such compounds, the alternation of perovskite layers with layers of another type is characteristic. In this case, perovskite blocks may contain one, two or more (n) octahedron layers, with a thickness of one layer of about 0.5 nm and characterized by a different deviation from the stoichiometry of the perovskite oxygen. In their structures, the octahedra are joined by vertices and, in the ideal case, form a volumetric frame with cubic symmetry. Their structural difference lies in the orientation of the layer, separated from the perovskite sublattice, in relation to its axes and in the interlayer space.

In the elementary cell of the phases of Ruddlesden-Popper, alternation of perovskite blocks takes place $[(A_nB_nO_{3n})]_\infty$ with balls such as rock salt [AO]. The structure of the Dion-Jacobson phases is characterized by alternation of perovskite blocks $[(A_nB_nO_{3n})^{4+}]$ and layers of type $[(B_2)^{4-}]$. Such REE-containing oxide phases, revealing a set of valuable physical properties, high activity and stability, satisfy the functional, technical, technological complex of requirements for modern materials in many new applications.

Available information on the state and possible ways of improving technology for creating such materials; current requirements for their stability and reproducibility of properties; expansion of their use; new information on reactivity and the transformation of layered perovskite-like oxides, stabilization of photocatalytic and sensory-active crystalline modification of anatase by ions NO_3^- [3], doping Ln_2O_3 [4, 5] upon receipt TiO_2 from solutions [6], trends in the technical facilities devlopment initiated the continuation of the study on this topic. And today the ways are found to control the technical parameters of the target products through the choice of composition, synthesis conditions and method of treatment.

Review research sources and publications

Now due to the technological methods of "soft chemistry" reactions, the possibility of creating substances with various structural features, the production of metastable compounds through the sequence of low-temperature topochemical syntheses has appeared. Synthesis methods that have recently appeared are combined considering the advantages of each used method (elements of pyrolysis and hydrolysis synthesis methods, the method of Pecuni (Fig. 1, [7]), combustion of liquid nitrate precursors (Tab. 1, [8]), sol-gel method) and use liquid nitrate precursors of different electronic structures elements.

For formation of target phases ion exchange [9], intercalation and deintercalation [10], various processes of substitution and condensation [11], splitting processes [12] and mutual transformations of one structure into another [13] (for example, transition from the phases of Ruddlesden-Popper in the Diona-Jacobson phase; transition within one type of phase with an increase or decrease in the number of layers).

The most common reactions of "soft" chemistry include reactions of ion exchange, during which there is the replacement of weakly bound cations of the interlayer space, while perovskite layers are sufficiently stable mainly due to covalent bonds of metal-oxygen and act as carcass in a layered structure. It enables to substitute some interlayer cations to other, without affecting the basic structure of the layered oxide. Such reactions can be used to obtain a wide range of new perovskite-like structures.

So layered oxides with Dion-Jacobson phases in solutions [14] and the Ruddlesden-Popper phases in the melt [9,15] of the corresponding nitrates salts are subjected to ionic substitution reactions of larger-dimensional interlayer cations such as Cs^+, Rb^+ i K^+, on cations of smaller size – Li^+, Na^+, NH_4^+. For the Ruddlesden-Popper phases, it is also characteristic to replace two monovalent interlayer cations with one bivalent cation (Mg^{2+}, Ca^{2+}, Sr^{2+}, Ba^{2+}; Ni^{2+}, Cu^{2+}, Zn^{2+} and others) with the transformation in the Dion-Jacobson phase (shown in the scheme of such ionic transformations, Fig. 2).

Now, to increase the effectiveness of these reactions and reduce the time expenditures, methods of physical influence on reactionary systems, such as microwave heating [16] and ultrasound treatment [17], have become more commonly used.

For technological reception of active phases of perovskite type, leading manufacturers use immersion (impregnation) of structured carriers of cellular or ball-like type in aqueous solutions of precursors - salts of nitrates of a given composition (possible spray application or other) in the absence of any additives or in the presence of chelating agents [18] (for example, citric acid (Pecuni method)), enabling to obtain bulk-linked (polymerized) high-concentrated homogeneous solutions of nitrate precursors. Thus, there is subsequent two stage heat treatment - drying in a conventional furnace (80–200 °C) or in a microwave oven (it prevents the redistribution of active components and ensures their good distribution on the substrate) and there is subsequent calcination at 700–1100 °C – converts predecessors into given oxide of the perovskite type [19]. Generally, phase-clear materials are obtained. Their active phase layer, 2–160 µm thick, covering the walls of a structured carrier, demonstrates high productivity through good accessibility to its active sites.

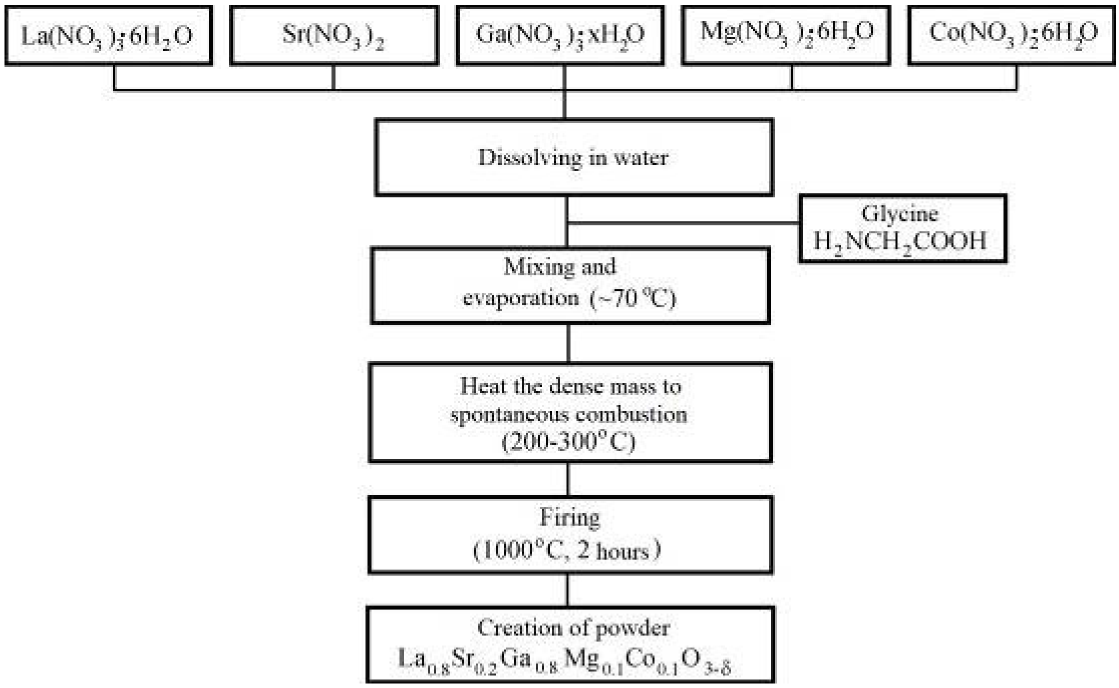

Figure 1 – The sequence of operations of formation of perovskite-like oxide phases $La_{0,8}Sr_{0,2}Ga_{0,8}Mg_{0,1}Co_{0,1}O_{3-\delta}$ glycine-nitrate method [7]

Table 1 – Components that are most often used in the preparatory stages during the formation of multicomponent REE-containing oxide phases by burning liquid nitrate precursors [8]

Oxidizer	Fuel	Solvent
metal nitrates or their hydrates: $Me^{v}(NO_3)_v \cdot nH_2O$, v - valence of metal	urea (CH_4N_2O) glycine ($C_2H_5NO_2$) saccharose ($C_{12}H_{22}O_{11}$) glucose ($C_6H_{12}O_6$) citric acid ($C_6H_8O_7$)	water (H_2O) kerosene benzene (C_6H_6) alcohols: ethanol (C_2H_6O) methanol (CH_4O) furfuryl alcohol ($C_5H_6O_2$) 2- methoxyethanol ($C_3H_8O_2$) formaldehyde (CH_2O)
ammonium nitrate (NH_4NO_3)	fuels based on hydrazine: carbohydrazide (CH_6N_4O) oxalyl dihydrazide ($C_2H_6N_4O_2$)	
nitric acid (HNO_3)	hexamethylenetetramine ($C_6H_{12}N_4$) acetylacetone ($C_5H_8O_2$)	

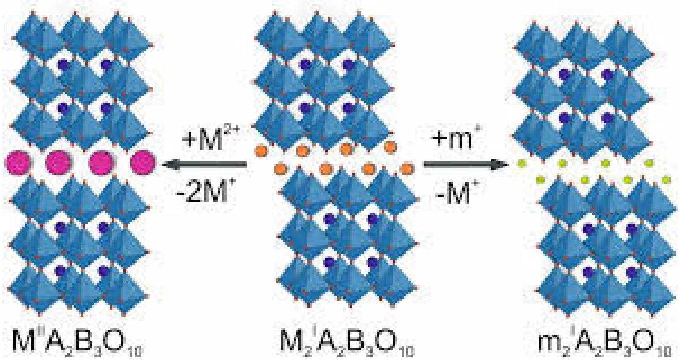

Figure 2 – Ion exchange of phases in Ruddlesden-Popper

A good alternative to the procedures that have been described previously is synthesis by the method of burning the solution [8]. It enables to receive a porous well-fixed perovskite layer with a smaller number of technological steps. After immersion in an aqueous solution containing precursors - nitrates of the relevant elements (oxidants), urea, glycine (fuel) and ammonium nitrate (combustion accelerator), the structured carrier is subjected to heated initiation to self-ignition of the system. For several minutes, the heat released during combustion of fuel (glycine), enables to convert the precursors of nitrates into a phase of pure perovskite type oxide. The removal of large quantities of gaseous products over a very short period of time during combustion gives a thick, porous and spongy coating of 40–100 μm in thickness with a high specific surface area (4–30 $m^2 \cdot g^{-1}$). Such a structure minimizes the drop in the pressure of the gas stream and improves the mass transfer. Coatings obtained by this procedure, show excellent adhesion. Their thermal aging at temperatures up to 850 °C in the presence of SO_2 or water does not cause significant deactivation.

Definition of unsolved aspects of the problem

It should be noted that the synthesis of nanosized oxide materials by combustion of solutions (or modification of the method) is a complex exothermic chemical process that self-sustains and propagates in the aqueous or sol-gel medium of precursor components. Such transformations are fleeting multistage; begin with the dehydration and thermal decomposition of liquid systems of initial precursors and include a series of thermally coupled exothermic reactions that lead to the formation of a solid product and a large amount of gas. Due to the variability of the source constituents nature, the conduction conditions, the activation methods, the course nonequilibrium, the actions of other specific factors of the target products formation in the aspect of knowledge of their mechanisms, the clarification of the phase transformations features, it is not simple scientific and technological object, which is now intensively investigated. Such attention to the method is explained by the ease of its implementation technology, the versatility of the approach and the impressive number of important applications: new materials for energy technologies – accumulators, super-capacitors, fuel and solar cells, efficient devices for mutual conversion and energy conservation; heterogeneous catalysts – for reforming hydrocarbons, cleaning from organic pollutants of solutions and air, oxidation and treatment of exhaust gases of cars / diesel engines; semiconductor and optical materials for photo catalytic and optical applications; thin films for the development of special types of transistors and electronic structures; nanoceramics (electro-, magneto-, bio-) and others.

Synthesis of solution combustion also positively differs from other varieties of self-propagating high-temperature synthesis, namely: 1) mixing of the initial components in an aqueous solution at the molecular level (ion sizes, ligands in solution 0.1–1 nm, size of solid powder particles by ceramic technology 102–105 nm); 2) the reaction responsible for the formation of a solid product may differ from the transformation of self-sustaining combustion; 3) a large amount of gaseous by-products leads to a substantial expansion of the solid product, a rapid drop in the temperature of the system after the reaction, which makes the solid product porous and slightly dispersed (it plays the main role in the synthesis of nanosized powder).

And modern studies are aimed at studying the thermodynamics and kinetics of reactive technological systems used in innovative solutions; the role of the process parameters responsible for self-sustaining combustion speed; the basic principles of composition management; physico-chemical characterization of composition, size, structure and microstructure, properties of both intermediate and final transformation products; elucidation of the influence factors, their degree of importance on the processes of formation and regulation of the size and morphology of nanosized phases, characteristics of target materials, etc.

Problem statement

The purpose of this work is to study cooperative processes of interaction between structural components during the formation of perovskite-like oxide phases of rare-earth and transitional elements in the preparatory stages using nitrates of different electronic structures elements and finding possible methods of influence on rare-phase and solid-phase systems based on the reagents thermal activation, from purpose of their structural-sensitive characteristics reproduction.

The most recent data on the results of such studies are far from unequal, in some cases they are due to large experimental constraints, sometimes contradictory and do not give a complete picture of the complexing ability of rare-earth elements in such objects. The unambiguous interpretation of the flow above these processes is often hindered also by the incongruous nature of the solubility (melting) of the intermediate phases formed, the simultaneous coexistence of several metastable forms of thermolysis product; the formation of heterophases, the dependence of their form on the prehistory of the process itself (the possible amorphous or poorly crystallized state of precursors [20]), the complexity of processes occurring on the boundaries of grains in polycrystalline systems, which are determined by the peculiarities of the chemical interaction of system components, the unevenness of the occurrence of transformations and the presence limiting their stages and other existing factors. The need for a deeper understanding of the above-mentioned features and the effective use of the possibilities of specific technological schemes for the formation of perovskite-like oxide phases, which nowadays become widely used, led to the study continuation.

Basic material and results

To evaluate the possibility of controlling the synthesis processes in the multistage production of products by iso-and heater-valent replacement of components and thermal processing; the study of the water-salt systems of nitrates has been studied as the modeling of the mechanisms of phase formation as a model using a complex of physico-chemical methods $Me^{I}NO_3 – Me^{II}(NO_3)_2 – Ln(NO_3)_3 – H_2O$, ($Me^{I}$ – K; Me^{II} – Mg, Ca; Ln – Nd). The choice of the research objects, temperature sections are determined by a number of factors. Among the elements of the rare-earth series, the higher complex-forming ability is found by representatives of the cerium subgroup; among them the greatest changes in composition, structure, properties of their compounds - elements of its middle, Pr and Nd. Selected system components specify the specifications of the target product or modifiers of its properties. And the presence of large quantities for the use of potential electronic analogues (representatives of natural rows of rare earth, alkaline, alkaline-earth elements) causes a considerable variation and breadth of the range of modification of their characteristics. The temperature cross sections are due to the domains of the crystalline hydrate forms of the initial components.

To find out the nature of the chemical interaction and phase equilibrium in the water-salt systems of the investigated nitrates (precursors of multicomponent oxide polyfunctional materials) in the full concentration ratios in the temperature range of the existence of solutions, the method of additives described in [21, 22] has been used. It is based on the solubility study as one of the properties that are most "sensitive" to the detection of phase transformations in systems that are both a parameter of their state, and moreover, the simple available experiment now thorough methods. The method enables to find the limits of self-development, to which, in specific conditions, an isolated system of a given composition goes in an equilibrium state. In order to increase the reliability of the data obtained, a simultaneous comprehensive study of the system components solubility, density and relative refractive index of light of their solutions is used. Good coherence of the obtained results is revealed.

Phase equilibrium has been achieved within 2–3 days. The starting salts used hydrated and anhydrous nitrates of the specified elements of the brand «ch.d.a.».

Chemical analysis of liquid and solid phases, "residues" has been carried out on the ion content Nd^{3+}, Mg^{2+}, Ca^{2+}, nitrogen. Contents Ln^{3+} have been determined trilonometrically; Mg^{2+} – volumetric method; Ca^{2+} – complexometric titration of the substituent in the filtrate liberated from Ln^{3+} ammonium buffer; nitrogen – the method of distillation; ion K^{+} – calculation of the difference, based on the total content of nitrates and partly on the dry residue.

The obtained data for individual ions have been transferred to the salt content and, according to the principle of compliance; have been applied to the solubility graph. The graphic representation of the solid phase composition formed in the system was carried out by Skreinemakers [21, 22]. Their individuality has been confirmed, in addition to chemical, X-ray, crystalloptical, thermographic, IR-spectroscopic, etc. methods of analysis.

The experimental data from the study of systems are obtained $KNO_3 – Me^{II}(NO_3)_2 – Nd(NO_3)_3 – H_2O$, ($Me^{II}$ – Mg, Ca) and summarized in the table. The complexity of transformations in the objects of the study clearly demonstrates the presented horizontal projections of their corresponding spatial isotherms of solubility (Fig. 1, 2).

For these systems, in the regions of phases coexistence, the positions of the quasi-equilibrium equations lines, the composition of the electron and transition points are determined in isothermal conditions; the quantity, composition, the nature of solubility, temperature and concentration limits of the phases formation of phases; regularities and features of complex formation, heterogeneous equilibria are established. Isothermal horizontal and frontal projections of their multidimensional solubility charts are constructed. Lines of equal water content are drawn. The relative sizes of the crystallization fields of all formed phases are established.

Triple systems, which are part of the quaternary magnesium-containing system, $KNO_3 – Nd(NO_3)_3 – H_2O$, $Mg(NO_3)_2$ - $Nd(NO_3)_3 – H_2O$ at 50 °C have been studied by the authors and described in previous publications [23–25]. The first system is characterized by the formation of incongruently soluble compounds of the composition $K_2[Nd(NO_3)_5(H_2O)_2]$ and $K_3[Nd(NO_3)_9]$; The second is a congruent soluble complex nitrate $[Mg(H_2O)_6]_3[Nd(NO_3)_6]_2 6H_2O$. In the system $KNO_3 – Mg(NO_3)_2 – H_2O$ the limits of the crystallization of only the initial salts have been established [26].

Isotherm solubility of the system at 25 °C has 4 crystallization fields KNO_3, $Mg(NO_3)_2·6H_2O$, $Nd(NO_3)_3·6H_2O$, $[Mg(H_2O)_6]_3[Nd(NO_3)_6]_2 6H_2O$. As the temperature rises, the nature of the interaction between the structural components of the system is greatly complicated. And with 50°C the solubility isotherm of the system, in addition to the crystallization fields of the initial salts, contains 2 more solid-phase separation fields $K_2[Nd(NO_3)_5(H_2O)_2]$ and $K_3[Nd(NO_3)_9]$. In the system, the largest area in horizontal projections is potassium nitrate. To carry out quantitative calculations of evaporation, crystallization, its front water projections are constructed; On horizontal projections, an insulating net of equal water content is applied. The smallest contents of water are solutions saturated with coordination nitrates $K_2[Nd(NO_3)_5(H_2O)_2]$, $K_3[Nd(NO_3)_9]$.

Investigation of phase equilibria in a four-component system $KNO_3 – Ca(NO_3)_2 – Nd(NO_3)_3 – H_2O$ preceded the study of heterogeneous equilibrium

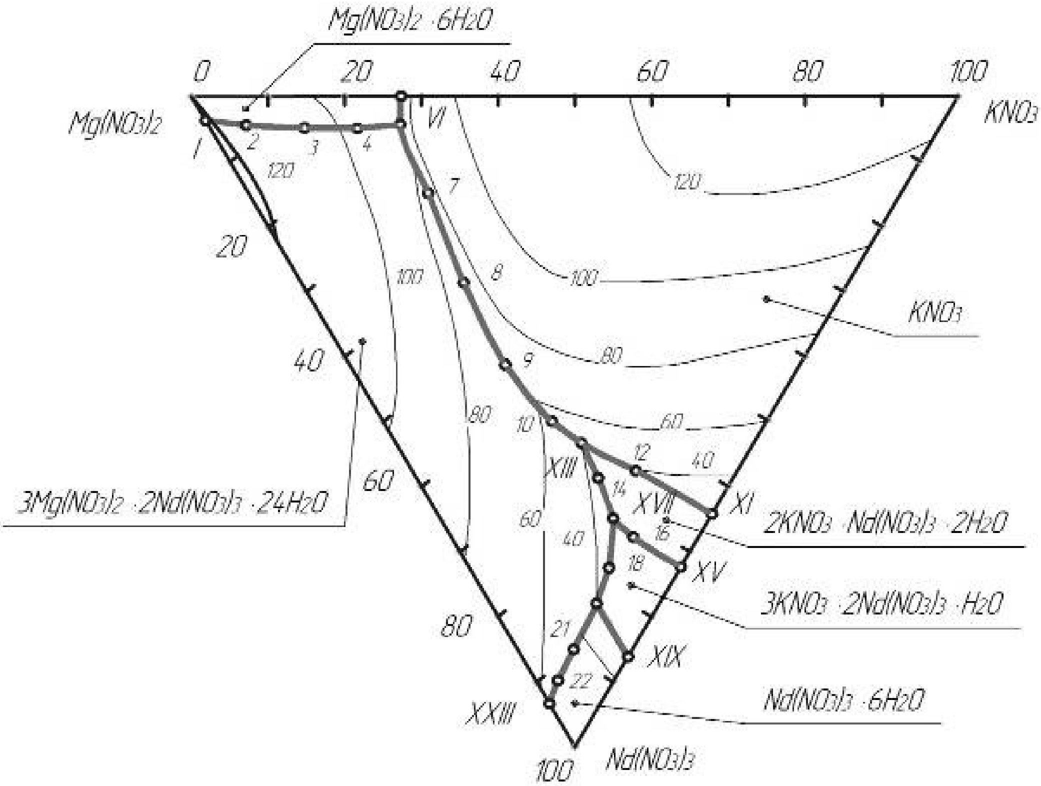

Figure 3 – Horizontal projection of the solubility spatial isotherm of the system $KNO_3 – Mg(NO_3)_2 – Nd(NO_3)_3 – H_2O$ at 50 ^oC.

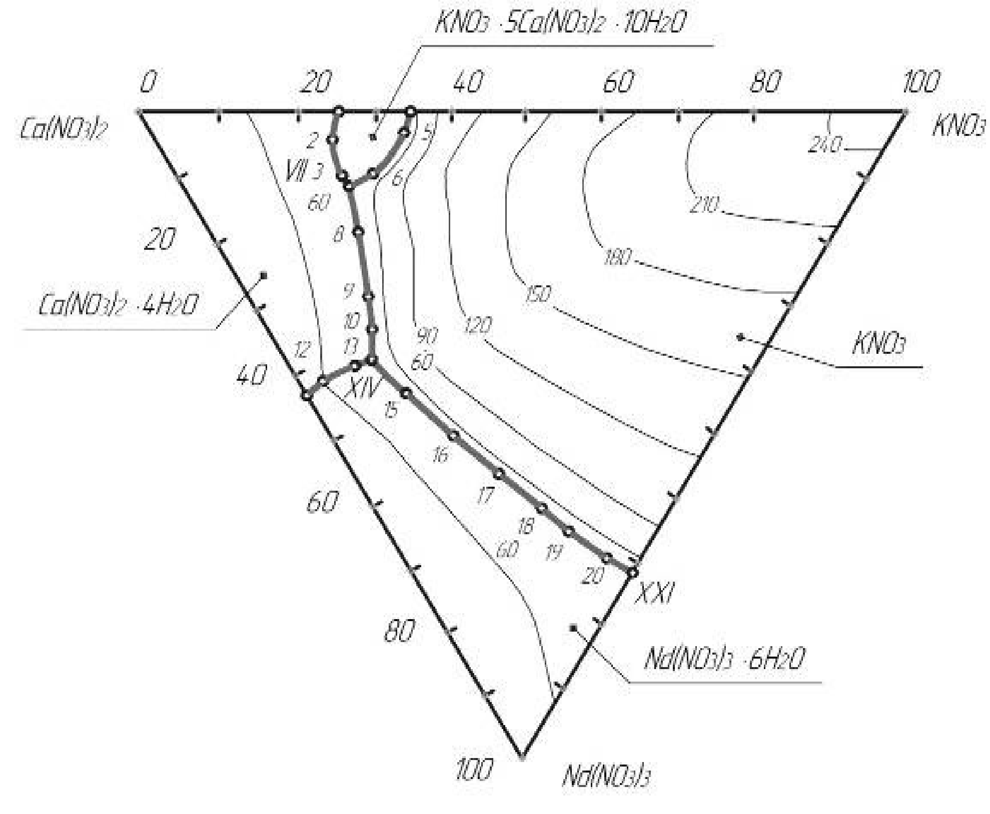

Figure 4 – Horizontal projection of the solubility spatial isotherm of the system $KNO_3 – Ca(NO_3)_2 – Nd(NO_3)_3 – H_2O$ at 25 ^oC.

in systems KNO_3 – $Nd(NO_3)_3$ – H_2O, $Ca(NO_3)_2$ – $Nd(NO_3)_3$ – H_2O, KNO_3 – $Ca(NO_3)_2$ – H_2O for the possibility of preparing refills of triple systems with saturated solutions corresponding to the contents of non-invariant points. Data from the study of the authors of the first two systems were discussed earlier in [23]. Existing system information KNO_3 – $Ca(NO_3)_2$ – H_2O are controversial, so it has neeb studied again. On the diagram of this system solubility (25 °C) there are separated fields of crystallization of the source components KNO_3, $Ca(NO_3)_2 \cdot 4H_2O$ and incongruously soluble compound $KNO_3 \cdot 5Ca(NO_3)_2 \cdot 10H_2O$; certain warehouses of electron and transition points. Synthesis is carried out with double nitrate, its chemical analysis confirms the ratio of components in the formula above.

Isotherm solubility of this four-component system at 25 °C has 4 crystallization fields $Ca(NO_3)_2 \cdot 4H_2O$, KNO_3, $Nd(NO_3)_3 \cdot 6H_2O$, $KNO_3 \cdot 5Ca(NO_3)_2 \cdot 10H_2O$. The increase in the temperature of the study narrows and, to this end, a limited number of the quantities ratios of the liquid and solid phase, which corresponds to the possibility of the system being in the given conditions in an equilibrium two-phase state, complicates the dehydration processes of the structural components, and the system near the non-invariant points passes into a metastable, syrup state.

All of these complex nitrates are synthesized in a monocrystalline form, their optimal conditions of formation and forms of growth are studied. The research of their atomic-crystalline structure, forms of coordination polyhedrons, type of coordination of ligands, a number of physical properties by chemical, X-ray diffraction, X-ray structural, IR-spectroscopic, crystalloptic, thermographic, GDG laser radiation and other methods have been carried out.

The conducted multidimensional studies using a set of indicated methods enabled to determine the tendencies of the joint behavior of structural components in the system of nitrate precursors of REE and elements of IA, IIA groups of the periodic system. It was established that in the studied water-salt systems of nitrate precursors, which are integral components of more complex multicomponent systems; exchange chemical transformations starts in the liquid phase, from the moment dissolution of the components in water – a strong polar solvent. The mechanism of complex formation can be explained from the positions of competing substitutions of water molecules in the immediate environment Ln^{3+} for NO_3^-- groups, disordering the structure of solutions by introducing one - (Li^+ – Cs^+) and two-charge (Mg^{2+} – Ba^{2+}) cations, influence of temperature factor. The degree of completeness of substitution depends on nature Ln^{3+}, present cations Me^+, Me^{2+}, the properties of the electron donor oxygen atoms and the spatial structure of the ligands, the concentration of the electrolytes and temperature. Significant influence on these processes of the thermal factor and their stage is revealed. The presence of certain values of isolation start temperature in the solid phase of complex compounds indicates the existence of an energy barrier and the need to provide some additional energy to the system for the possibility of such transformations.

According to the results of previous studies [23] and available scientific data, there are differences in the complexing cerium and yttrium subgroups elements ability, Y, as well as among RHES within the first subgroup. In the formation of nitrate complexes to a large extent, the requirements of symmetry are fulfilled, and planar small-sized ligands NO_3^- is "convenient" for the formation of a high-symmetric environment of ions Ln^{3+}. The structure of such compounds is based on rare-earth coordination polyhedrons, which are somehow connected in space [28]. The influence of nature is revealed Me^+, Mg^{2+} on the form of coordination polyhedrons, a way of packing complexes into a spatial structure, properties of compounds.

The publication also summarizes the systematized information provided by the authors (see Table 2) on the peculiarities of internal organization and physico-chemical characterization of tumors [29] – coordination nitrates of REE for the possibility of combining and directing modern scientific, technological and technical efforts to solve urgent tasks. on the formation of perfect multicomponent oxide polyfunctional materials with mixed electronic and oxygen conductivity, rapid ion transport for systems of mutual transformation p ofthem forms of energy, oxygen-conducting materials in the conversion of natural gas, fuel cells, many catalytic and magnetic systems, oxygen membranes, high-electrodes, heating elements in gas sensors and others.

Conclusions

The empirical data obtained by the authors as for the atomic-crystalline structure formation and existence conditions. its features and patterns, the inherent properties, mechanisms and kinetics of thermal transformations (25–1000 °C) of the representatives of alkaline anionic coordination nitrates of lanthanides play an important role in optimizing the development of technologies for the production of new nanosized multicomponent oxide materials with the structure of perovskite, pomegranate in the form of powders, thin films, bulk ceramics, and components of composite systems. They help to clarify the relationship between the method of formation, the variability of the systems activation methods, the methodology of manufacturing and phase composition, lattice parameters, the magnitude of the specific surface, the morphology of the particles, the catalytic activity of the samples in the photoinduced reactions of water decomposition for the purpose of obtaining hydrogen (as an alternative species fuel), decomposition of toxic organic substances, incomplete oxidation of carbohydrates; when receiving other perovskite-like phases through ion exchange reactions, which can significantly simplify the procedures for the target products synthesis.

Table 2 – Characteristic properties of the representatives of isoslast groups of coordination nitrates of rare earth, yttrium (^oC)

Compounds; spatial group of crystals	Representatives	Dehydration	Melting in crystallization water	Polymorphic transitions	Melting anhydrous form	Note
$Li_3[Ln_2(NO_3)_9 \cdot 3H_2O$ cubic.; $P2_13$	La–Sm	65 183 216	183	–	274	Temperature data for the coordination compound Nd
$Na_2[Ln(NO_3)_5] \cdot H_2O$ monocl.; $P2_1/a$	La–Sm	81 148 236	–	271	328	Temperature data for the coordination compound Nd
$K_2[Ln(NO_3)_5 (H_2O)_2]$ rhomb.; Fdd2	La–Sm	95 111	95	219	314	Temperature data for the coordination compound Nd
$K_3[Ln_2(NO_3)_9] \cdot H_2O$ cubic.; $P4_332$	La–Sm	126	–	–	347	Temperature data for the coordination compound Nd
$K[Ln(NO_3)_4(H_2O)_2]$ prim. rhomb.; $P2_1cn$	Y, Gd–Lu	138 172	138	–	–	Temperature data for the coordination compound Gd
$Rb_2(Ln(NO_3)_5 (H_2O)_2]$ monocl.; Cc	La, Ce	93 144	93	–	327	Temperature data for the coordination compound La
$Rb_5[Ln_2(NO_3)_{11}] \cdot H_2O$ monocl.; C2/c	Pr–Sm	172	172	–	–	Temperature data for the coordination compound Nd
$Rb_3[Ln_2(NO_3)_9] \cdot H_2O$ cubic.; $P4_332$	La–Sm	136	–	–	302	Temperature data for the coordination compound Nd
$Rb[Ln(NO_3)_4(H_2O)_2] \cdot H_2O$ monocl.; $P2_1/n$	Y, Gd–Lu	98 165 231	98	–	–	Temperature data for the coordination compound Yb
$Cs_2[Ln(NO_3)_5(H_2O)_2]$ monocl.; 12/a	La	99 148	148	–	263	Temperature data for the coordination compound La
$Cs_2[Ln(NO_3)_5(H_2O)_2]$ monocl.; C2/c	Ce–Nd	122 156	156	–	235	Temperature data for the coordination compound Nd
$Cs[Ln(NO_3)_4(H_2O)_3]$ threelc.; $P\bar{1}$	Pr–Sm	62 90 162	162	–	355	Temperature data for the coordination compound Nd
$Cs[Ln(NO_3)_4(H_2O)_2] \cdot H_2O$ monocl.; $P2_1/n$	Y, Gd–Lu	155 175 252	155	–	–	Temperature data for the coordination compound Yb
$[Mg(H_2O)_6]_3[Nd(NO_3)_6]_2 \cdot 6H_2O$ hexagon.	La–Sm	109	109	–	–	Temperature data for the coordination compound Nd

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Influence of molybdenum on corrosion and mechanical properties of carbon steel joint welds

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The results of experimental studies of the molybdenum impurity influence on corrosion and mechanical properties of carbon steel (Grade 20) joint welds are presented in the article. In particular, it has been found that the highest and stable values of the impact strength and resistance properties of cracks (K_{1s} and δ_s), as well as the resistance to sulfide corrosion cracking of the metal of tubular steel joints, are achieved at a concentration of molybdenum in it from 0,2 to 0,4%, which is realized by putting a molybdenum powder in the electrode coating in the amount of 0,5-1,0%. On the basis of the obtained results an optimal chemical composition of the weld metal was determined which is characterized by a fine-grained structure with a small amount of non-metallic impurities of globular shape. Optimal content of the doping micro-additive - molybdenum should be selected, based on both the influence of molybdenum on the size of structural components, but, most importantly, on its effect on the corrosion and mechanical properties of metal joint weld.

Keywords: welding, corrosion, crack resistance, strength, sulfide, hydrogen.

Вплив молібдену на корозійно-механічні властивості зварювальних з’єднань вуглецевої сталі

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Наведено результати експериментальних досліджень впливу молібденової домішки на корозійно-механічні властивості наплавленого металу зварювальних з’єднань вуглецевої сталі марки 20. Встановлено, що найбільш високі й стабільні значення ударної в’язкості й характеристик спротиву розвитку тріщин (K_{1c} і δ_c), а також стійкості проти сульфідного корозійного розтріскування метала шва трубних сталей досягаються при концентрації молібдену в ньому від 0,2 до 0,4%, яка реалізується вводом в електродне покриття молібденового порошку в кількості 0.5-1.0%. На базі отриманих результатів визначено оптимальний хімічний склад наплавленого метала, який характеризується дрібнозернистою структурою з незначною кількістю неметалевих вкраплень глобулярної форми. Такий хімічний і структурний склад зварювального шва реалізується оптимальним вмістом і співвідношенням феросплавів (FeMn, FeSi і FeTi) в електродному покритті. Оптимальний вміст легуючої мікродобавки – молібдену слід вибирати, виходячи одночасно не лише з впливу молібдену на розмір структурних складових, але, головне, з її впливу на корозійно-механічні властивості метала зварювального шва. Покращення механічних властивостей, зокрема, ударної в’язкості і параметрів в’язкості руйнування метала шва, легованого молібденом, можна пояснити його сприятливим впливом на структурну та хімічну неоднорідність наплавленого метала. Для оцінювання ступеню цього впливу стосовно електродів з основним покриттям, проведено дослідження сучасними методами металографічного аналізу. Порівняння даних структурного та мікрорентгеноспектрального аналізу дозволяє передбачити, що покращення пластичних властивостей метала шва при легуванні молібденом зв’язано з тим, що молібден зміщає область $\gamma - \alpha$ - перетворення вбік більш низьких температур, сприяючи тим самим утворенню достатньо дисперсної та однорідної структури нижнього бейніту з мінімальною шириною доевтектойдної феритної оторочки.

Ключові слова: зварювання, корозія, тріщиностійкість, міцність, сульфіди, водень.

Introduction

The highest and most stable values of impact strength and crack resistance, as well as resistance to sulfide corrosion cracking of weld metal joint of pipe steels are achieved by adding molybdenum powder to the electrode coating. On the basis of this approach, an optimal chemical composition of the weld metal, which is characterized by a fine-grained structure with a small number of non-metallic globular form impurities, was determined.

Review of research sources and publications

The analysis of literature data [1, 2, 5, 7, 9, 10] showed that high and stable values of the impact strength of metal joint s on carbon and low-alloy steels are to a large extent provided by deoxidation and doping of the metal joint with manganese, silicon and molybdenum. As follows from works [1, 2, 9, 10], thus the content of silicon and manganese in the weld metal should be in the following limits: from 0,10 to 0,60% Si and from 0,60 to 1,50% Mn.

In order to determine the optimal content of the molybdenum powder in the coating, ensuring the required concentration of molybdenum in the metal joint and high impact strength of the joint s at temperatures up to -30 °C, additional research was required on the steel pipe of the steel grade 20, which is most widely used in industrial production in conditions of sign-changing temperatures, pressures and loads.

Definition of unsolved aspects of the problem.

It is known [1 – 4, 8, 12 – 14] that the choice of pipe and welding materials for pipelines of TES industrial plants is carried out in accordance with the Rules and Standards of Ukraine's boiler control. Reliability of pipelines largely depends on the corrosion and mechanical properties of steel pipe and welding materials, but existing scientific and technical and technological developments concerning the increase of operational reliability and durability of pipelines (P) reveal contradictions and uncertainty both as among researchers so among operators, lack of a clear idea of the factors that cause the failure and destruction of the P, as well as scientifically substantiated practical recommendations regarding the optimal choice of welding materials, the technology of welded tubular steels, which are exploited under conditions of chemical aggressive media under variable temperature-baric regimes of industrial production [3 – 5, 10].

Problem statement

In connection with this, there was a need to find ways to increase the operational reliability of pipelines by ensuring high corrosion and mechanical properties of joint welds, which will serve as the basis for the development of technological and operational measures to improve the safe resource of pipelines, in particular oil and gas.

Basic material and results

As experimental, the electrodes with the main type of coverage of the brand ANO-26, were used. In the process of manufacturing the electrodes in the charge, a microfertilizer of molybdenum was put in the form of powder in quantity (in %): 1,0 (P1); 2,5 (P2); 3,0 (P3); 4,0 (P4). Welding was carried out by electrodes of 4 mm diameter from the rectifier VDU-504 in the modes: U_d = 23 – 24V; I_{sv} =180 A (DC, reverse polarity). Before welding, the electrodes were calcined in a thermo-oven at a temperature of 4000C for 1 hour. In the weld metal, the molybdenum content varied from 0 to 0,60%. The chemical composition of the weld metal was (in %): 0,071 – 0,075 C; 1,08 – 1,20 Mn; 0,32 – 0,40 Si; 0,016 – 0,023 S; 0,020 – 0,024 P.

Initially, using a standard method [2, 5], the critical tensile stresses of the samples cut from the welds in the longitudinal direction were determined. Test results are presented in Fig. 1.

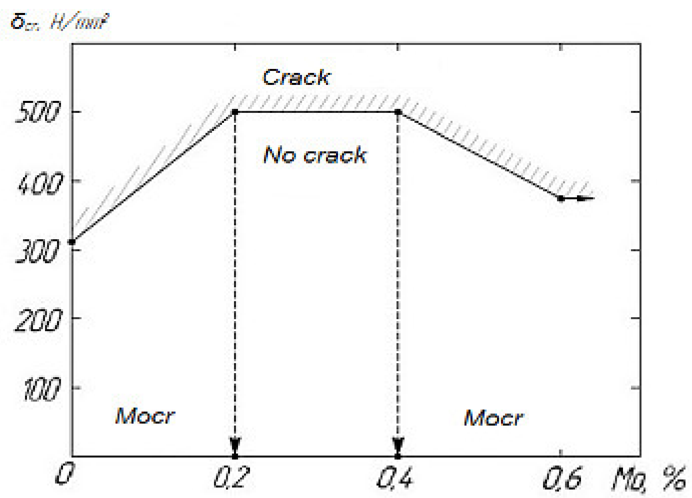


Figure 1 – Dependence of the critical tensile stress on molybdenum concentration in the weld joint. joint 20

It is seen that the limiting values of molybdenum content in the deposited metal are in the range of 0,20 – 0,40% for steel 20. The value of σ_{cr} corresponds to the critical value of the tensile stresses, where the origin and growth of the crack occurs up to destruction. As the research has shown, doping of a metal with molybdenum allows to improve also other widely used in fracture mechanics the viscosity characteristics of weld metal, in particular, parameters of the critical stress intensity (K_{1s}, MPa·m$^{1/2}$) and the critical crack opening (δ_s, mm), which characterize the resistance of the metal joint s to the opening of the crack [6].

For the manufacture of samples, the steel joint s were heated by 20 electrodes of diameter 4 mm (variants P1 – P4) at constant current (I_{sv} = 180 A, U_d = 23-24 V) from the power source – VDU-504. A notch on the welded specimens was applied along the joint. Electrodes ANO-26 were used as experimental electrodes, in the coating of which instead of iron powder, molybdenum was used in the amount of 0; 1,0; 2,0 and 4.0%. The fatigue cracks in the specimens were grown by means of a hydropulsator CDM-10 (Germany) at a load frequency of 10 – 15 Hz and a coefficient of asymmetry of the cycle R = 0,1 – 0,2. Tests to determine the parameters of the viscosity of destruction were carried out at the installation of UME-10 according to the standard method [2, 6, 9].

Results of measurements are given in fig. 2.

It can be seen that the metal of welded joints, which is doped with molybdenum (0.2-0.4%), has higher coefficients of K_{1s} and δ_s over the whole range of temperatures than the main metal (steel 20), that is, it is characterized by a higher fracture failure. The highest values of the critical magnitude of the stress intensity factor K_{1s} and the coefficient of crack opening δ_s are obtained for welds with a molybdenum concentration of 0,2 – 0,3%.

The improvement of mechanical properties, in particular, the impact strength and viscosity parameters of the metal joint doped with molybdenum, can be explained by its beneficial effect on the structural and chemical heterogeneity of the deposited metal. In order to assess the degree of this effect with respect to the electrode with the main coating, additional research was carried out using modern methods of the metallographic analysis.

a) b)

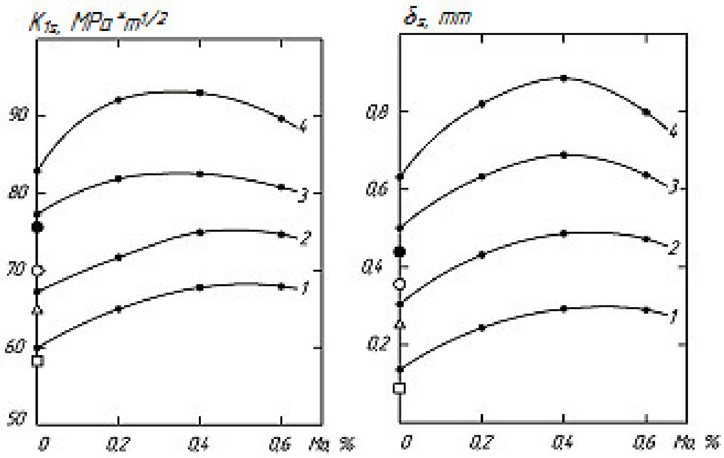


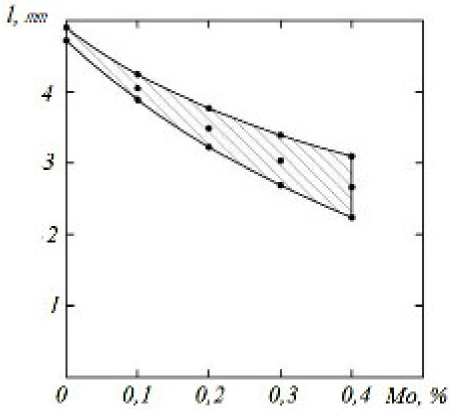
Figure 2 – Dependence of crack resistance coefficients K_{1s}(a) and δ_s(b) on the concentration of molybdenum in the welding joint.

Temperature (in °C): 1 – (-30); 2 – (-20); 3 – (-10); 4 – (+20). Steel 20

The metal single-layer welds made by test electrodes with varying molybdenum content in the coating (electrodes with indexes P1, P2, P3 and P4) were studied. The chemical composition of the welded metal is given in the text above. Welded on a direct current at the return polarity from the straightener model VDU-504 in the mode: I = 180 A, U_d = 23-24 V.

The structure of the metal joint was studied on a scattering electron microscope of the model "JSM-35CF" (firm "Jelol", Japan). The results of studies have shown that the structure of the metal joint of new electrodes is characterized by the following features. Non-equilibrium grains of upper bainite (diameters of 200 – 600 µm and length 0,5 – 1,6 mm) are surrounded by polycrystalline hypoeutectoid ferrite plain in the width of 15 – 25 µm, which does not retain the secretion phase of the introduction, but with non-metallic inclusions and perlite colonies along its borders. In the body of grains there are plates of carbides (mainly iron carbides) in the thickness of 10-15 microns, small perlite colonies and non-metallic inclusions, usually spherical shapes with a diameter of 0,5-2,5 microns.

From the data in Fig. 3 it follows that in the metal of the joint weld, the micro-additive - molybdenum causes a decrease in the length of columnar dendrites (ℓ), and at the same time their width is reduced (h). It is noteworthy that with an increase in the concentration of molybdenum in the metal up to 0,4%, the length of dendrites decreases by about 1,5 – 2,5 times, and their width to 1,5-2 times. So, when the length and width of the dendrites of the metal joints, which do not contain molybdenum, is 4,7 – 5,6 mm and 3,7 – 3.9 μm respectively, then, when doping with a micro-additive – molybdenum in volume 0,25% of columnar dendrites have the following parameters: ℓ = 3 – 3,7 mm and h = 25 – 3 μm. In addition, it is evident that the change of these quantities is already subtracted when doping a joint weld with molybdenum in the amount of 0,1 – 0,2%. For these values, the microstructural parameters have the following values ℓ = 3,5 – 4,1 mm and h = 28 – 32 μm. It should be noted that at the same time, the micro-hardness of molybdenum positively affects the fragmentation of equilibrium dendrites – the parameter d (see Fig. 3). It is seen that with increasing content of molybdenum in weld metal, such as 0,1 to 0,25% dendrites diameter decreases from 30 – 36 to 18 – 22 μm, that is, the value of dendrites diameter is reduced by an average 1,5 – 2,0 times. It should be noted that the optimal content of the doping micro-additive - molybdenum should be selected, based on both the influence of molybdenum on the size of structural components, but, most importantly, on its effect on the corrosion and mechanical properties of metal joint weld.



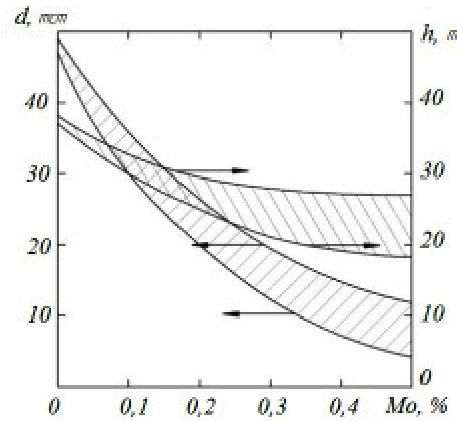


Figure 3 – Influence of microadditives of molybdenum on the length of dendrites (ℓ) and size in a welding joint.
Denomination: h – width of stained dendrites; d – the diameter of equilibrium dendrites

Comparison of the data of structural and micro-ray spectral analysis allows predicting that the improvement of the plastic properties of the metal joint in molybdenum doping is associated with the fact that the molybdenum displaces the region of γ-α-transformation towards lower temperatures, thereby contributing to the formation of a sufficiently dispersed and homogeneous structure lower bainite with a minimum width of a dovetectoid ferrite flange. Such a structure, as it is known [7, 9, 10], contributes to the high mechanical properties of the metal joint , in particular its impact strength.

Data of fractographic analysis of samples tested on impact bend (in the range of temperatures + 20 ... – 30 °C), showed that metal breakage of the joint doped with molybdenum, are viscous parts of the dimple type. In this case, the fraction of the viscous component in breaking these samples is not less than 90%, while in samples without molybdenum, does not exceed 40 – 50%.

At the same time, welds, which are doped with 0,2% molybdenum, collapse along the plane of the chip of the lower bainite packets. The above changes in the nature of the destruction of the joint as the molybdenum concentration increases from 0,2 to 0,4%, apparently, are the cause of the observed growth in its impact strength.

The increase of impact strength due to the doping of the joint with molybdenum is due not only to the fragmentation of the structure of the metal joint , but also to the effect of molybdenum on the dislocation structure of the ferrite matrix and bainite packets [5]. Molybdenum, which is part of the bainite packets, reduces their hardness, thereby contributing to plastic deformation [7].

Considering the fact that metal pipes, for example, chemical and agrarian production, in particular evaporators, steam boilers, steam lines, etc., for a long time have been in contact with a chemically aggressive medium containing sulfur, which causes sulfide corrosion, including its specific type - sulfide cracking of welded joints, then samples were tested for sulfide cracking according to the standard NACE TM-01-77 [11]. As a model medium served a saturated hydrogen sulfide solution containing 5% NaCl and 0,5% acetic acid. In this case, the content of H2S was 50 g/ liter. The initial pH value was 3,8, the final value was 4.1. The ambient temperature is 220C, the basic test time is 680 hours.

The results of corrosion-sulfide cracking tests are given in Fig. 4, from which it is evident that the content of molybdenum in the volume of 0,2 – 0,4% has a beneficial effect on the resistance of the metal against sulfide cracking, and this trend is manifested in the same way as in investigations on the fracture resistance of the metal joint (parameters К1s і δs).

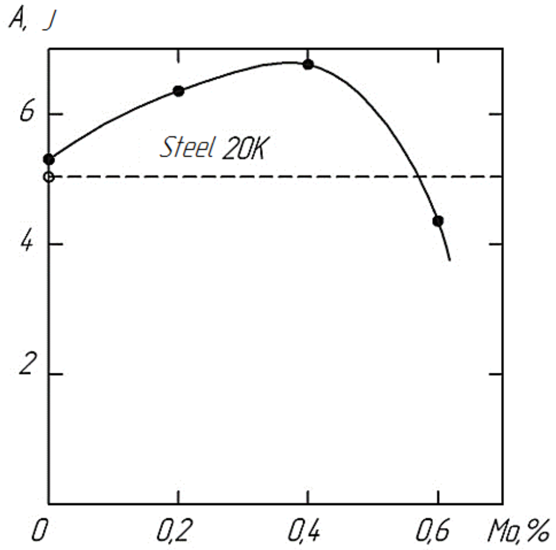


Figure 4 – The work of breaking welds depending on the concentration of molybdenum. NACE environment

Conclusion

It was established that the highest and stable values of the impact strength (65 – 77 J/cm^2 at t =-30°C) and the resistance properties of cracks (K_{1s}= 65 – 75 MPa·m$^{1/2}$ and δ_s= 0,23 – 0,47 mm at t =-30 °C), as well as resistance to sulfide corrosion cracking of tubular steels metal joint are achieved at a concentration of molybdenum in it from 0,2 to 0,4%, which is realized by introducing into the electrode coating molybdenum powder in the amount of 0,5 – 1,0%.

The optimum chemical composition of the welded metal is determined, which ensures the obtaining of fine-grained structure, containing a small amount of globular form non-metallic inclusions (in %): C < 0,18 – 0,22; 0,25 – 0,35 Si; 0,8 – 1,0 Mn; 0,2 – 0,4 Ni; S, P < 0,025, realized by the optimal content and ratio of ferroalloys in the electrode coating: 4 – 6% FeMn, 6 – 8% FeSi, 8 – 10% FeTi; FeTi : FeSi : FeMn = 2 : 1,5 : 1.

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