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AUTONOMOUS AUTOMATED ACCUMULATING COMPLEX FOR THE ROBOTIZED PUNCHING PROCESS

The accumulating complex was designed for automated loading with discoid billets of unified single-crank presses in robotized punching processes. Advanced variants of the task practical implementation by means of technical, diagram, design upgrading and processing equipment retooling are suggested. Raising the productivity and reliability of the facility, simplifying and cheapening its construction is based on the use and integral combination of possibilities provided by «active», «passive», information parameters influence on the formation and correction of operational production flows of its function supporting systems.

Keywords: *special tooling, accumulating complex, billets loading, robotized punching processes, automation, innovations.*

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АВТОНОМНИЙ АВТОМАТИЗОВАНИЙ НАКОПИЧУВАЛЬНИЙ КОМПЛЕКС ДЛЯ РОБОТИЗОВАНОГО ПРОЦЕСУ ШТАМПУВАННЯ

Створено накопичувальний комплекс для автоматизованого завантаження штучними дископодібними заготовками уніфікованих однокривошипних пресів у роботизованих процесах штампування. Запропоновано сучасні варіанти практичної реалізації поставленої задачі технічним, схемним та конструкторським модернізуванням і переоснащенням технологічного обладнання. Підвищення продуктивності й надійності роботи засобу, спрощення й здешевлення його побудови засновано на використанні та комплексному поєднанні можливостей впливу «активних», «пасивних», інформаційних параметрів на формування і корегування операційних виробничих потоків систем забезпечення його функціонування.

Ключові слова: *спеціальне оснащення, накопичувальний комплекс, штучне завантаження заготовок, роботизовані процеси штампування, автоматизація, інновації.*

Formulation of the problem. One of the important tasks in automating the production processes of punching is arrangement of automatic loading and billets orientation. It is particularly important through the lens of reducing the unattractive manual labor and increasing the efficiency of technological equipment.

Despite numerous research and engineering developments, this problem can not yet be considered completely solved for the class of small and medium-sized articles such as plates with implicitly expressed design features (orientation keys) and distinctive physical and mechanical properties (which are expressed, for example, in the process of metal billets transportation by sliding as a result of structuring and the residual magnetization of their surface layer, in the case of using billets from materials of reduced rigidity and strength, friability, etc.) because such kind of articles is impossible to apply traditional automation operations on loading and orientation of billets with mechanical entrapment. Their designing is a complicated research and technical problem, and the search for the latest solutions involves the use of knowledge, experience gained on the need for a comprehensive set of measures in the individual operations automation using industrial robots in various production fields:

– engineering improvement of universal technological equipment;

– upgrading the construction of the part (billet) itself, which should be suitable for the conditions of its manipulation with the possibility of entrapment, transfer and accurate positioning in the operating space;

– changes in the technological process in order to adapt it to the technical capabilities of the existing industrial robot that is available or purchase this process for servicing;

– selection of the parts orientation devices (it should be noted that the choice of the parts orientation method and the choice of the bases for their entrapment during transportation are closely linked);

– selection of part magazine (billets) (the use of billets storage units reduces the time spent by personnel while robots work preparation, and allows to combine technological equipment of different efficiency into a single cycle);

– manufacture of new special or specialized equipment (design and manufacture of such additional equipment, as well as improvement of the technological equipment itself, normally require comparatively little time and can be performed by the own efforts of the manufacturer);

– special measures to provide safety (security);

– development of the layout scheme, planning mutual arrangement of the technological equipment, robots, operation panels, loading mechanisms, storage magazines, containers and vehicles in the exact compliance with the technological procedure of production operations performance and requirements to the accuracy of positioning;

– design and control over the processing sector operation algorithm (preceded by the development of technological processes and technological calculations (determining the processing mode and time, necessary and possible auxiliary and final time determination, the robot actuator operation best trajectories and velocities and cycles and cyclograms, determining the storage magazines capacity calculation).

Recent research analysis. The known similar solutions of such tasks are implemented in the modern equipment complexes for punching articles using the single-billet loading [1 – 14]. In the above works data are presented on their composition, structural architecture, operation, specifications and capabilities. Consideration of the existing developments features allows analyzing and characterizing them from the standpoint of analogues when selecting and comparing structural elements, the operation and configuration principle, interaction type; it gives them an assessment from a constructive, technological and economic point of view, unifying nodes and parts etc.

Such technological complexes are based on the use of magazine, stack and bunker types of accumulating (storage) devices [15, 16]. Analysis of their layout indicates the existence of several target nodes (tools) common to all of their three types: charge capacity, trapping device, accumulating device, cut-off devices, power supply, reset, activator of movement, manipulator of oriented billets supply to the punching zone, drives. The variety of designs is developed due to variations in the layout and ways of implementing these specified facilities. The simplest version of the technological complex is a unified press equipped with one of the supply or feeding types. Currently, slide gate devices, revolving or clamshell loaders, manipulators and industrial robots are generally used [see. 9 to 19].

Such developments are cinematically complicated, characterized by a large number of friction pairs and considerable dissipation of energy, they have limited lifetime and require preliminary preparatory and routine operations, require high precision of their components manufacturing and their adjustment using specialized processing equipment. In a number of cases, they are complicated, expensive in cost, provide for the maintenance by trained personnel. But their disadvantage is that they can not be universal due to the existence of manufactured objects wide variety and complexity; the broad spectrum of functional tasks to be solved, principles and methods of their realization; the use of materials that do not completely satisfy the whole set of technical and technological requirements, their compatibility and other factors.

Identification of previously unsettled parts of the general problem. High return of the equipment included into the flexible production systems requires, as a rule, its equipping with automated billets (parts) warehouse, as well as the tools and special equipment necessary for the uninterrupted operation of the studied manufacture types.

Development of production systems in order to improve the operation reliability and timely provision of all elements within the technological process includes the use of perfect upgraded systems to ensure their functioning. At the same time, experience shows that the increase in the level of automation is achieved by a significant complication of structures and technological equipment systems, and, consequently, increase in their cost. Therefore, the required level of automation must be economically justified.

Expansion of the robotic system spheres of application, emergence of compact and simultaneously powerful performing facilities, development of technical automation means, elemental base, circuitry and control systems enable the use of a higher level design, introduction of new technical solutions, approaches and principles of operation when creating adapted special technical equipment for the preparatory stages of particular industrial production processes, where the use of traditional means is inappropriate or does not permit to technically solve the problem as a whole. Such problems are solved by introducing constructive technical changes in the transport routes, orientation, sorting, billets discharge, performing nodes, entrapment devices, equipping the latter with auxiliary external information sensors that respond to the presence of the manipulation object, its shape, size, weight, surface condition, clamping force, sliding, positioning etc.

Statement of assignment and methods of its solving. The purpose of this processing tool is development of multi-product loading and accumulating automated complex for single-piece disc-shaped billets feeding that functions interactively in flexible manufacturing robotic connection with the universal single-crank presses and combines the possibility of coupling with the available processing equipment of the present-day machine builder, with minimal modifications of the existing auxiliary equipment, by the authors' own efforts, in compliance with emerging innovation trends.

Study results and their discussion. The paper suggests consideration of technical implementation options for autonomous accumulating complex, designed to serve single-crank presses of single-action, open, with a normal force of 250 kN, in compliance with the existing standard GOST 9408-89, and provides obtaining pressed articles (perforated parts from the disk-shaped billets) by means of loading billets in bulk into the storage hopper, their further orientation, sorting, single-piece feeding, synchronous with the press's operation feeding to the punching press, punching (hole perforation) and stipping articles off from the punching press. Performance of the operations given sequence and the nodes coordinated operation is provided by the electronic control unit.

The presented development in general represents the totality of functionally connected facilities and devices constructively united on the general sequential logic principle of operation, and includes the loading bunker device (storage-hopper), sloping storage magazine of billets, standard one-crank press, special structure punch, device feeding billets to the punch: pneumatic manipulator KMO.63 C 4212, device for removing products from the punch, device for controlling the technological complex.

Technical data and specifications of the development:

Number of billets loaded simultaneously into an empty bunker, pcs.	600
Loading lot weight, kg	24
Diameter of billets, mm	60
Thickness of billets, mm	1,5
Operating modes:	a) «manual»; b) «automatic»: – «single», – «cyclic»
Average productivity of the complex in the automatic mode at full loading ofthe bunker with billets, punched articles per minute (pcs. / min)	40
Mean duration of one cycle, sec.	6
Capacity of the slip storage magazine, pcs.	8
Positioning accuracy, mm	$\pm$ 0,2
Rotation angle of the feeding device rod, degrees	45 - 240
Supply voltage, V	220 / (-24)
Air pressure in the performing mechanisms' line, MPa (kgf / cm^2)	0.45 (4.5)
Billet loading bowl dimensions, mm:	
diameter	450
height	150

The components interaction of the developed complexes and their work by operations are described below and, for clarity, explained by the scheme of the bunker-loading vibration device (Fig. 1) and by the block diagram of the technological complex control device (Fig. 2).

Disks-billets, of the given diameter and the fixed thickness, are loaded «in bulk» into the bunker-loading device (Figure 1). By means of electromagnets (4) the bunker is brought into oscillatory motion. The bowl (1) of the vibro-bunker with the screw-shaped transport track is installed on the inclined elastic-spring supports (6). Due to such mounting, vertical and horizontally tangential perturbation components are transmitted to the billets. Under their influence, orientation (the billet lies flat), stratification and circular movement of the billets are performed. The profile of the accumulator's bottom (2) has a small slope from the center of the bowl to the walls and ensures the billets displacement during their movement to the periphery of the bunker and on their way to the transport track. The billets are aligned one by one and move along the spiral of the tray.

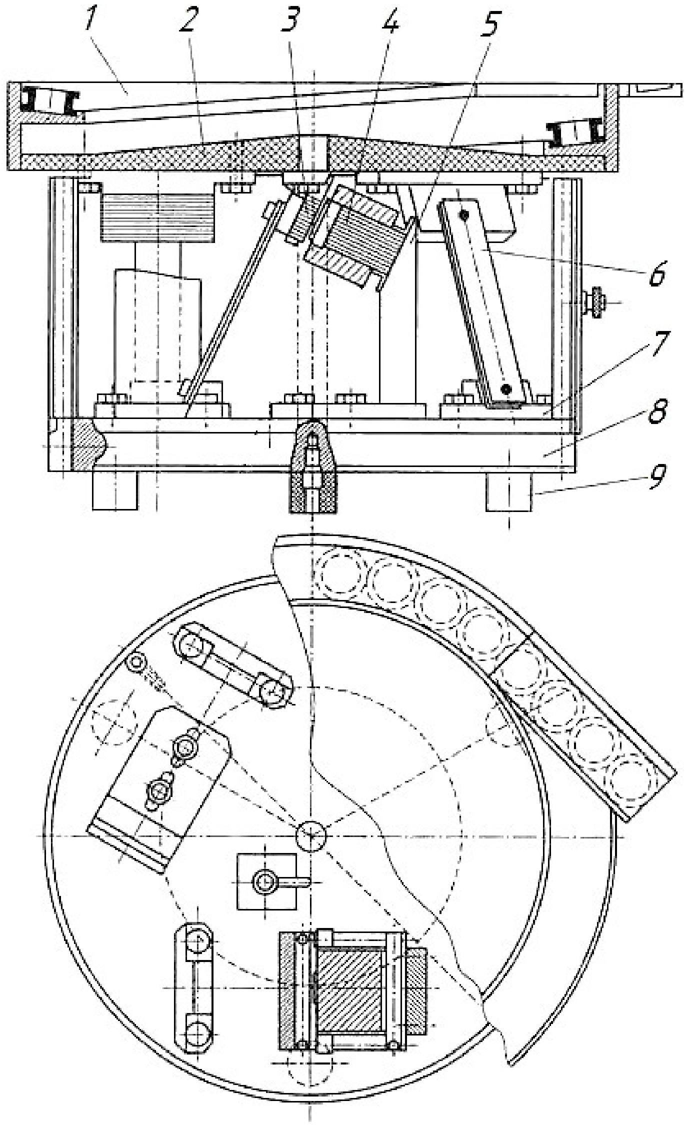

Figure 1 – Diagram of the bunker loading vibration unit:
1 – bowl for loading billets; 2 – textolite conical bottom;
3 – anchor; 4 – electromagnetic vibrator; 5 – vibrator mount column;
6 – suspension (elastic spring support) mounting the storage bunker bowl;
7 – bracket; 8 – foundation; 9 – shock absorber rubber

To eliminate the overlapping movement of the billets, sorting them from deformed disks and those with large burrs along their pathway, an «active» cramping-proof slot is made. It is formed by the rubberized ejector shaft, the axis of which is located downstream at a small angle (approximately 30 °) to the tangent of the transport track reference circle. The height of this slot is regulated and set about 1,5 of the billet thickness. The ejector shaft drive rotates at high speed, entraps the topping billet and ejacts it through the inner side wall of the tray into the storage bowl. The billet repeats the pathway covered. The rotation frequency of the ejector motor shaft has two discrete values and is set from the control panel.

The billets output from the vibro-bunker tool of the storage device is of a probabilistic, random nature. To ensure the coordinated operation of the storage device and the press, the complex is equipped with an intermediate inclined storage magazine.

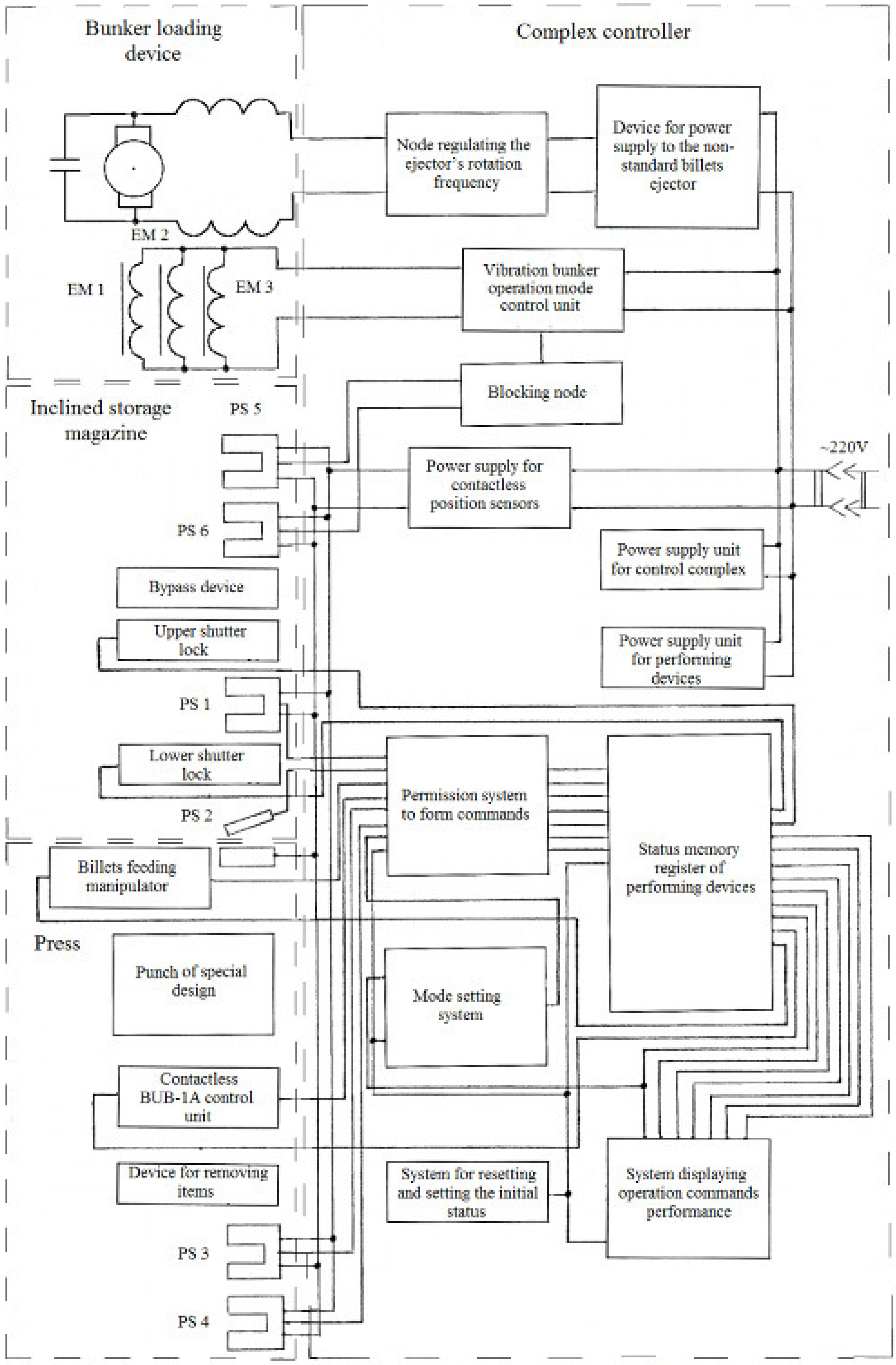

Figure 2 – Block diagram of the device for controlling the robotic processing punching complex

In the initial state (Figure 2), the lower electromagnetic shutter of the bypass device, the «feeder» of the inclined storage magazine, is closed, and the upper one is open. Billets follow one by one from the vibro-bunker device and, under the influence of the component gravity, fill the storage magazine. When forming a stack of 8 billets, using the contactless position sensors PS 5, PS 6 and the logical locking device, the drive of the vibro-bunker device is disconnected, feed of the billets stops and overflow of the inclined storage magazine tray is eliminated. When feeding the billet into the press punch and reducing the stack, the logical device switches on electromagnets of the vibro-bunker and carries out the additional filling of the magazine. The process is repeated. The filling of the inclined storage magazine does not depend on other proceeding operations, it is performed simultaneously with the press operation. The best accumulation mode is selected by the optimal conditions for vibrotransporting of billets into the vibro-bunker by choosing the vibration amplitude, by adjusting the voltage supplying electromagnets EM 1 – EM 3 (with the constant previously selected stiffness of the inclined elastic columns (6) and width of the gap: anchor (3) – electromagnet magnetic conductor (4) of the drive), visual control of which is performed by means of the voltmeter built-in on the control panel.

In the «operation» mode, if there is a billet in the initial position in the storage tray of magazine (sensor PS 1 between the electromagnetic shutters of the bypass device is set in the logical «one» position) and there are no interlock signals, the control device switches on the bypass device (the lower electromagnetic shutters’ stops are lowered, those of the upper ones are raised), and the billet slides from the storage magazine tray onto the surface of the matrix and moves slowly along it. At the time of slipping, the PS 2 sensor is activated, which trigs the single-shot pulses shaper with the constant duration sufficient to switch on the device of feeding a billet intothe punch. According to the circuit design, the bypass device is in the «on» position from the moment the «start» command given up to the moment when the position sensor PS 2 is activated. All this time the upper electromagnetic shutter stops of the bypass device are in the raised position, keeping the next-to-the-last and the above disposed billets from slipping and falling into the punch.

The ramming device, designed on the basis of the unified pneumatic manipulator KM0.63 C 4212, through the V-shaped entry by means of the rod, sends the billet until bumping into the opening of the matrix node and ensures the alignment of the billet geometric center with the vertical mandrel of the punch. The ramming device drive is actuated by the electro-pneumatic valve by the command of the PS 2 position sensor.

Moving the rod of the ramming device is regulated by the location of the stops, which the angle of the air engine shaft rotation is varied with. Speed of the drive shaft’s rotation in the forward and reverse direction is changed by means of the pneumodrossels Dr 1, Dr 2. Constructively, the device has a track microswitch, triggered from the camshaft mechanism at the end of the drive shaft backward stroke and signalling that the operation of the billet ramming to the punch is completed. After this command, the press start circuit generates a single pulse and actuates the press by the electronic control unit BUB – 1A. The article punching (perforation)is being performed.

BUB – 1A has a perfect control system and only performs the operation if all the functional units, press systems are properly operating, there is no interlocking signal and the energy carriers’ parameters are within the normal limits. Therefore, due to the presence of the press drive actuation pulse, we confirm the fact of the punching operation performing. This principle is the basis of the processing operation completion control and it allows to avoid installing additional position sensors. The punching completion control circuit is connected in parallel to the electropneumatic valve of the press drive and generates the command signal for the subsequent ejection operation on the trailing edge of the press start pulse, i.e., after the punching completion (holes perforation). Wherein, according to functional features of the press, this signal ends when the press rod rises to the upper position.

To be safe, the electropneumatic valve of the device for stripping the article off the punch is opened with a short delay. The pneumatic cylinder is activated. Its shaft pushes a punched (perforated) disk from the punching unit and sends some progressive pulse to it. The disc falls into the inclined chute and under the action of the component gravity slides into the receiving bunker of the finished parts.

In the process of ejection, the «flag» rigidly connected to the shaft in the extreme right position of the ejector enters the gap of the PS 3 generator sensor magnetic conductor, at the signal of which the direction of air supply to the cylinder changes. The shaft returns to its initial position. It is identified by the signal of the logical «one» from the position sensor of the PS 4 ejector initial state. This signal is simultaneous with the signal for the processing system cycle operation completion.

From the control panel, actuation of the system operation is envisaged in the «manual» and «automatic» modes.

In the «manual» mode, there is a possibility of individual units' sequential actuation: the bypass device, the ramming device, the press, the ejector, as well as one complete cycle of the system operation in the «single-piece feeding» mode when the «Start» button is pressed.

In the «automatic» mode, the signal from the PS 4 sensor at the end of the cycle is the command for performing the subsequent cycle by the system. Thus, its operation cycle-after-cycle is repeated according to the algorithm described above.

The electronic control device of the complex has a number of interlocks, ensuring its reliable operation.

Each subsequent operation can be performed only after the completion of the previous one. For this purpose, in the control device there is a register of memory cells to store information about the condition of the respective unit, which are set into the initial «zero» position before the beginning of each operation cycle. Information on the operations progress for visual inspection is output to the control panel via a display system. It is visual and necessary both in the setup mode and in the operating mode.

The start-up of the complex can only be performed when the ejector's shaft is in the extreme left position, derived from the matrix, and in the presence of the billet in the initial position in the inclined storage magazine.

The press can only be started when the ejector shaft is withdrawn in both «automatic» and «manual» modes.

In the «manual» mode, if two or more buttons are pressed simultaneously, the control command will not follow. Control can only be carried out by a single performing device.

At any time, at any stage of operation, it is possible to stop the complex operating by pressing the «Stop» button.

With the help of this development, high-performance, reliable, inexpensive, flexible, technology-based punching systems with piece loading of billets can be designed and equipped with auxiliaries, which are fundamentally different from the industrial and prior-art analogues. Such technological means can be easily implemented and operated both independently and within the functional lines. It is possible to modify the developed version depending on the functional tasks to be solved and the parts manufactured type changes, under the operations previous algorithm, by introducing structural changes in the vibration transport tracks, orientation nodes, storage magazine or by their replacement.

In order to improve the performance and reliability of the developed tool, special equipment and electronic position sensors, contactless control circuits for actuators and mechanisms are constructed; operating, diagnostics, control, and locking systems are designed.

Conclusions. The accumulation complex was designed for automated loading of discoid workpieces (billets) into the unified single-crank presses in the robotized punching processes. The present-day variants of the task practical solving by means of engineering, circuit and design modernization and retooling of the processing equipment are suggested. Improving the productivity and reliability of the device operation, simplifying and cheapening its construction is based on the use and integrated combination of the «active», «passive», information parameter possibilities influencing the formation and correction of operational production flows in systems ensuring the device functioning.

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FEATURES OF DURABILITY CALCULATION FOR MACHINE PARTS AND STRUCTURAL ELEMENTS UNDER HIGH ASYMMETRIC LOW-AMPLITUDE LOAD CONDITIONS

The study has been conducted by means of physical, mathematical and computer modeling integrated method use. To prove the adequacy of the results obtained the experimental procedure on the existing equipment and laboratory facilities has been applied. The method of carrying out asymmetric stress cycles with mean stress of stretching to symmetric using the proposed piecewise – linear equations for evaluating the material sensitivity to asymmetry of the cycle has also been improved. It has enabled pipe column element durability under the condition of typical asymmetric low-amplitude loading calculation.

Key words: *stress reduction, the boundary toughness, the asymmetry of the cycle.*

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ОСОБЛИВОСТІ РОЗРАХУНКУ ДОВГОВІЧНОСТІ ДЕТАЛЕЙ МАШИН ТА ЕЛЕМЕНТІВ КОНСТРУКЦІЙ, ЯКІ ПРАЦЮЮТЬ В УМОВАХ ВИСОКОАСИМЕТРИЧНОГО НИЗЬКОАМПЛІТУДНОГО НАВАНТАЖЕННЯ

Дослідження об'єкта проводилось за допомогою комплексного методу, що полягає в сумісному використанні фізичного, математичного та комп'ютерного моделювання. Для підтвердження адекватності отриманих результатів використано експериментальні методи на діючому обладнанні та на лабораторних установках. Удосконалено метод приведення асиметричних циклів напруж4

ень з середнім напруженням розтягу до симетричних із використанням запропонованих кусково-лінійних рівнянь для оцінки чутливості матеріалу до асиметрії циклів. Це дає змогу проводити розрахунок довговічності елементів трубних колон в умовах дії типового для них високоасиметричного низькоамплітудного навантаження.

Ключові слова: *приведення напружень, границя витривалості, асиметрія циклу.*

Introduction. The process of loading for a large number of structures and machinery parts is characterized by a large scatter of asymmetrical stress cycles both along its length and in timeframe. To the full extent it also concerns the elements of the drill string, particularly when drilling deep holes. Therefore, the vast majority of experiments determinating the fatigue resistance parameters is carried out at a symmetric cycle of stresses as a required stage for calculating column elements strength and bringing asymatrical cycles to the symmetrical equivalent.

The analysis of the latest research papers. The vast majority of machine parts and subassemblies in the process of operation is subjected to random loading [1 – 3]. In this case, when calculating the durability in the schematization process [4], conducing of stresses with different asymmetry R ratio to the symmetric cycle is recommended. Such a cast greatly simplifies further calculations conducing single-ended voltages σ_{max} with $-1 \le R \le 1$ for the symmetric cycle to σ_{ekv} with the recommended equation [5]

$$\sigma_{ekv} = b\sigma_{max} - (ab-1)\sigma_{-1}, \tag{1}$$

where σ_{-1} – is the endurance limit at symmetric loading;

a i b – are odds cast;

$$a = \frac{2}{2-(1-\psi)(1+R)}, \qquad b = \frac{1}{\dfrac{V_0}{V_{-1}}(1+R)-R}, \tag{2}$$

where $\psi = \dfrac{2\sigma_{-1}}{\sigma_0} - 1$ – is the sensitivity ratio to the asymmetry of the load cycle;

σ_0 – are the limits of endurance of the load;

V_0, V_{-1} – is the characteristic angle of the left branch fatigue curve in the semilogarithmic system according to zero and symmetrical load.

The analysis of equation possible use (1) for bringing symmetric asymmetric cycles of loading processes and drill rod to equivalency has been made.

Lines of equal damage are constructed on the Haigh diagram of the cycles with a positive mean stress (1).

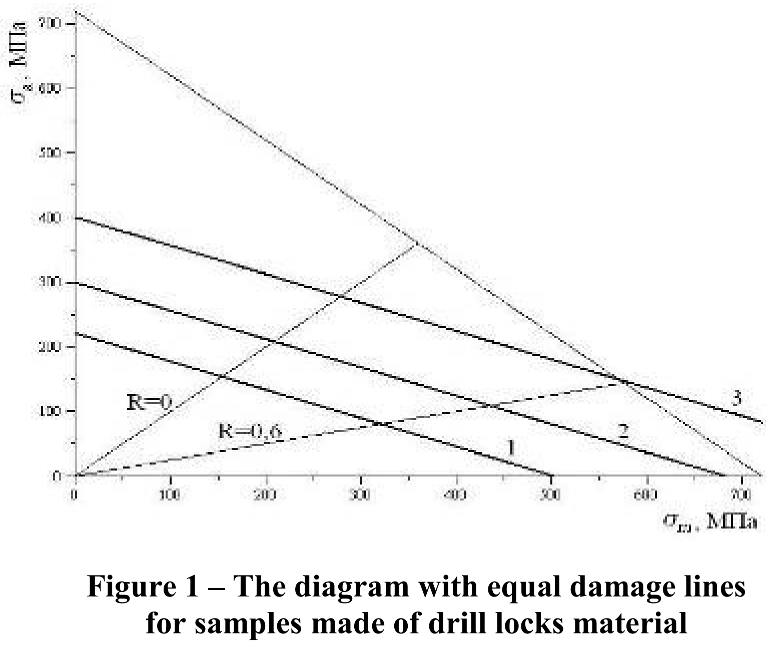

Figure 1 – The diagram with equal damage lines for samples made of drill locks material

On the graph line (Haigh diagram), which describes the cycle $R = const$, it is determined from equation (3)

$$y = \frac{1-R}{1+R}x \ , \tag{3}$$

where $R = 1$ is the x-axis, $R = -1$ coordinate axis, $R = 0$ – ray $y = x$.

Lines of equal damage are constructed in accordance with the equation (1) and data [6] for samples of steel 40XH, which is the material of drill pipe locks. Line 1 corresponds to the limit of endurance, line 2 is high fatigue and line 3 is low fatigue.

As it can be seen from Fig. 1, lines of equal distortion are in conflict with the real physical process for high asymmetric cycles with skewness 0.6 and above, which are characteristics for the load of the drill string upper part . So, lines 1, 2 may cross the x-axis in no case.

It can mean that a certain medium voltage level fatigue failure would occur for a certain number of cycles with infinitely small amplitudes that never happens in practice. Line 3 also has no physical meaning, because the outside of the diagram indicates a certain number of cycles before sample failures that are supposed to be broken down due to the stress exceeding tensile strength.

Therefore, it is necessary to adjust the corresponding equations (1) in case of asymmetrical stress cycles with high asymmetry. For this adjustment appropriate reduction equations have been developed [7].

In consequence, for aligning the asymmetric cycle asymmetry factor $-1 \le R < 0$, from the condition of invariance ψ of load levels ratio it was obtained [7] (4)

$$\sigma_{ekv} = \sigma_{max}\left(1 - \frac{(1-\psi)(1+R)}{2}\right) . \tag{4}$$

For conducing asymmetric cycles $0 \le R < 1$ the results of experimental studies of the asymmetry effect on the durability of materials and elements of aircraft structures given in [9, 1] (fig. 2) have been analyzed

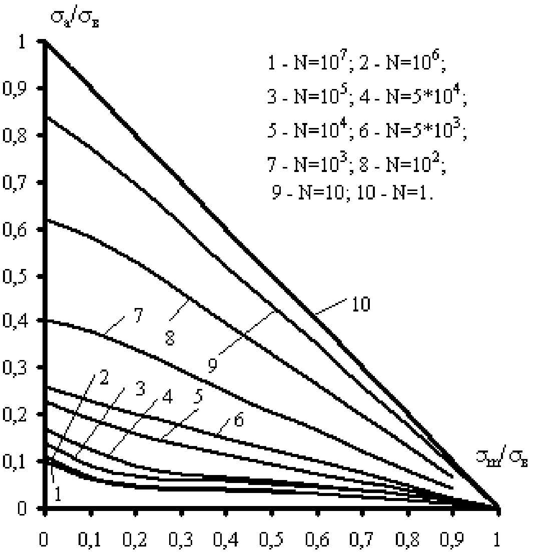

Figure 2 – Asymmetry loading impact on the durability of samples made of aluminium alloy 2024-T3 [7]

The authors have studied immense amount of information (over 1,000 experiments) which makes the results obtained extremely valuable and revealing under the conditions of inevitable statistical variance.

The results are illustrated by the Haigh diagram with no dimension coordinates

$$x = \frac{\sigma_m}{\sigma_b}\ ; \quad y = \frac{\sigma_a}{\sigma_b}\ .$$

Curves 1-10 are ones of equal damage for a certain number of stress cycles to failure of samples ranging from static destruction ($N = 1$) through low and high fatigue endurance limit ($N = 10^7$).

The inclination of line passing through the points (0, σ_{-1}) i ($\sigma_0/2$, ($\sigma_0/2$), to the x-axis represents the ratio of sensitivity to the asymmetry load ψ and is determined by the equation (5).

$$\psi = -\frac{y(R=-1) - y(R=0)}{x(R=-1) - x(R=0)}\ . \tag{5}$$

From the analysis of the data shown in Fig. 2, it can be argued that the angle of inclination of the equal damage curves within multi cyclic fatigue ψ factor is satisfactorily described only if $-1 \le R \le 0$ and, $0 < R < 1$ provided the tilt angle increases with decreasing N. Therefore, it is suggested to approximate curves of equal damage for asymmetrical tension with the average tension stretch with two straight lines. For tensions from $-1 \le R \le 0$ the coercion will be just according to (7).

Since all curves of equal damage converge at a point with coordinates (1,0) on the Haigh diagram, it can be used to bring cycles with mean stress of stretching ($0 < R < 1$).

A diagram of the suggested cast is shown in Fig.3.

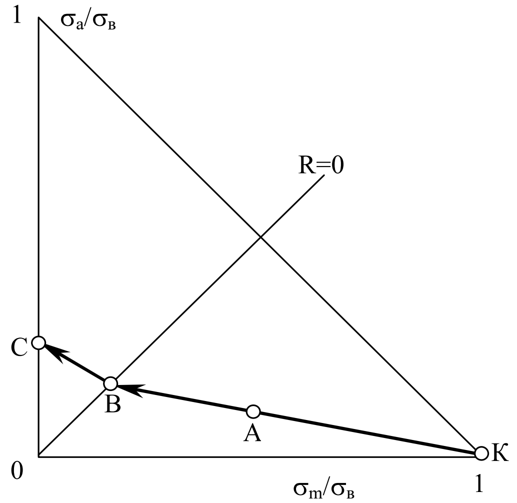

Figure 3 –The scheme of reduction to a symmetric cycle voltage with the average voltage stretch [7]

For example, let us consider the reduction to the symmetric loading cycle shown in figure 3 by point A (σ_m/σ_b, σ_a/σ_b). Through point A to the intersection with the straight line $R = 0$ (point B) we shall draw a ray, which is obtained from point K with coordinates (1,0). It was introduced a new coefficient specifying the influence of cycle asymmetry ψ_1. By analogy with (5) it is taken as (6)

$$\psi = -\frac{y(A)-y(K)}{x(A)-x(K)} = -\frac{y(B)-y(K)}{x(B)-x(K)} \ . \qquad (6)$$

Considering the coordinates of points A and K, it is obtained (7).

$$\psi_1 = -\frac{\sigma_a}{\sigma_b - \sigma_m} \ , \qquad (7)$$

$$y(B) = \psi_1 \,(1 - x(B)) \ . \qquad (8)$$

Since $y(B) = x(B) = \sigma_{max}(B)/2\sigma_b$, from equality (8) it is obtained (9)

$$\sigma_{max}(B) = 2\sigma_b \frac{\psi_1}{1+\psi_1} \ . \qquad (9)$$

Given that $R(B)$ = 0, further, the cast is performed according to (4). Obtained dependence is [7]

$$\sigma_{eq} = \sigma_b \psi_1 \frac{1+\psi}{1+\psi_1} \ . \qquad (10)$$

Proposed rate ψ_1 is determined by the equation [7]

$$\psi_1 = \frac{\sigma_{max}(1-R)}{2\sigma_b - \sigma_{max}(1+R)} \ . \qquad (11)$$

To justify the strength design of drill columns, the implementation analysis of the drill string operational load has been made using the Haigh diagram. Schematization of the loading processes has been carried out by the developed technique [4].

For instance, in (Figure 4) a general layout of the Haigh diagram with the imposed process of rod string columns loading is given.

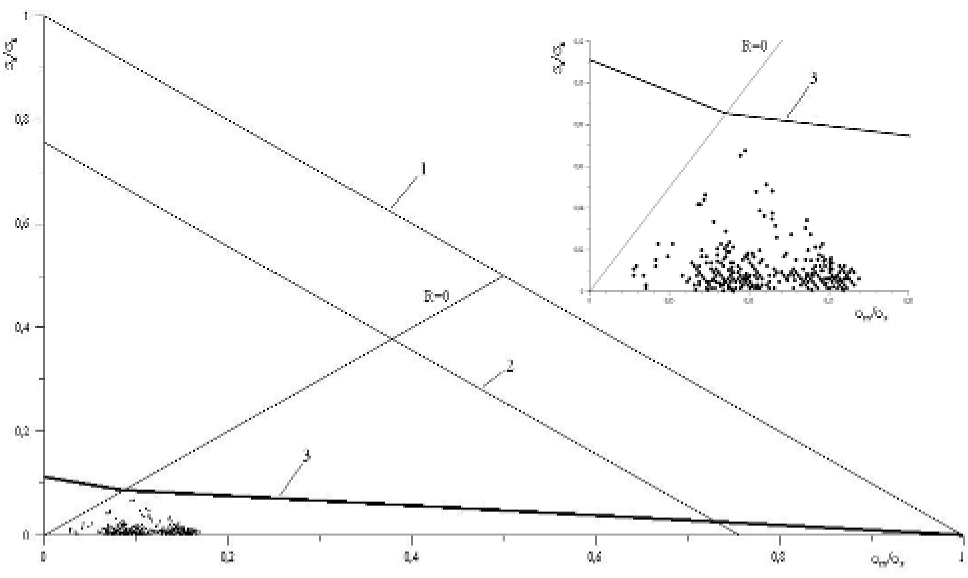

Figure 4 – A general layout of the Haigh diagram for the imposition of the process string rods columns loading

1 – static destruction $\sigma_{max} = \sigma_b$; 2 – line of border fluidity $\sigma_{max} = \sigma_m$; 3 – boundary line of endurance

In (Figure 5) treat processes over the drill string loading during lowering and for stitching during lifting are demonstrated.

It should be noted that in the case of stitches, the characteristic feature of all processes is the absence of cycles with stress amplitude above the corresponding border of endurance.

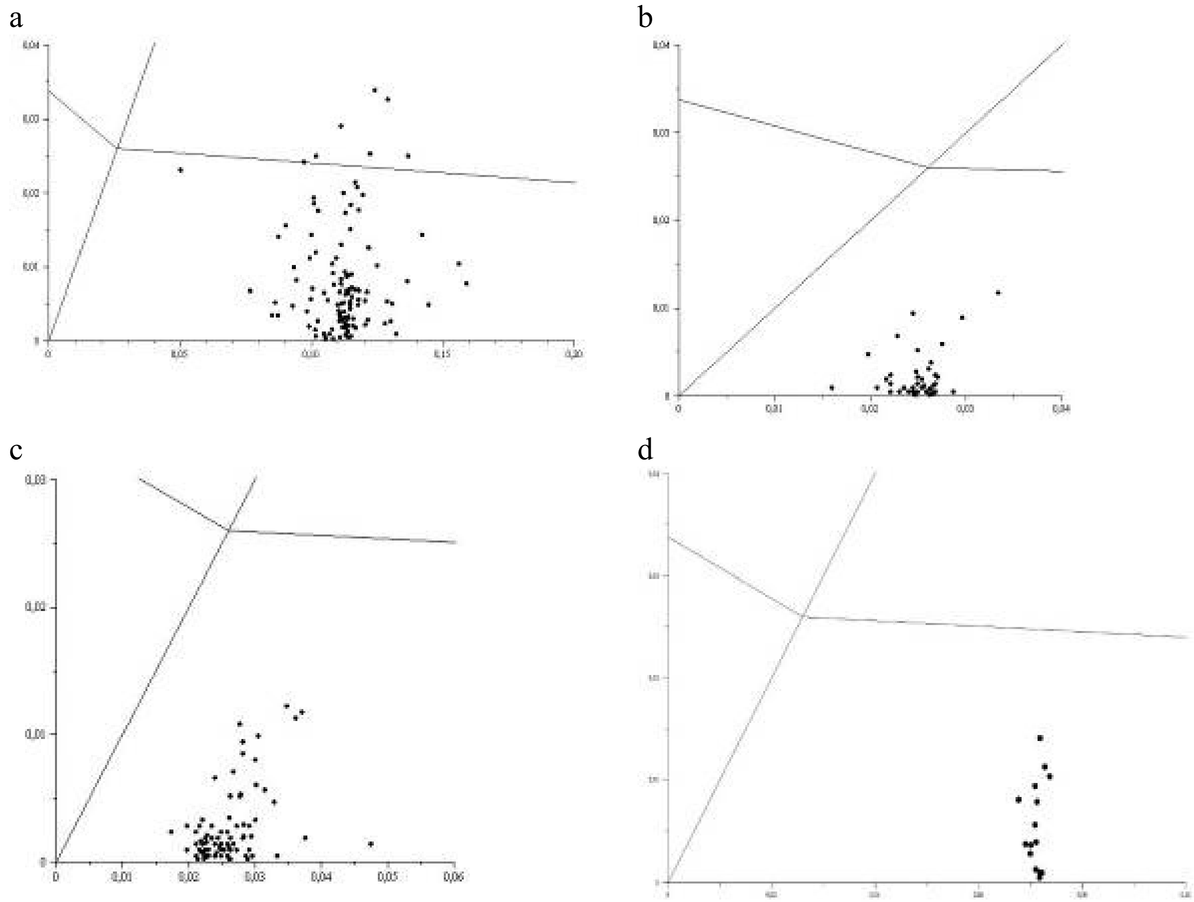

Figure 5 – The Haigh diagram with the process load of the drill string during lowering of the lifting operations

a – seams along the column length 500 m;
b – descent of the column length 190 m;
c – descent of the drill string length 500 m;
d – descent of the column length 1970 m

Generic problem unsolved parts selection. Thus, loading processes analysis occured under the operation of the drill string elements, shows that in the range of voltages the highest place is taken by low-amplitude voltage settings σ_{max}, R, which do not exceed the appropriate grants endurance σ_R. In this case, it is necessary to consider the inevitable reduction of the fatigue limit in the process of damage accumulation [8, 11, 12], caused by the action of the low amplitude voltages. So bringing σ_{max} to σ_{ekv} should extend to the stress cycles, which are smaller for the border of endurance. This again points to the particular importance of developing refined methods of bringing low-amplitude load cycles to assess drill string durability.

The use of equation (1) to bring $\sigma_{max} < \sigma_R$ has a significant limitation, namely, under the condition $\sigma_{max} < \sigma_R - \sigma_{-1}$ voltage σekv becomes less than 0. In this case, it is recommended not to consider this voltage as it is not producing any damaging effects [5]. But the neglect of low voltages under normal conditions (low amplitude loading) elements of the drill string is sure to have significant influence on their corrosion-fatigue life. It should be noted that the rejection of low voltage will lead to overestimation of the design life. It is also dangerous considering secure columns. The equations derived (4,11) also do not consider drill string loading elements specific.

The article goals. Therefore, the aim of this work is to develop the cast equations for asymmetric stress cycles drill string to the symmetric cycle with the features of their load.

Main material and results. For the development of such a refined method, it has been guided by the laws of low amplitude corrosion fatigue, characteristics and damage in Haigh diagram.

So, the decrease in the level of loading below the endurance limit reduces the sensitivity to unbalance loads. Damage accumulation is mainly due to dislocation mechanism, where the main factor is the amplitude. The effect of baushinger, which, presumably leads to a change in factor sensitivity of the load cycle asymmetry for the transition to exclusively tensile load on these stages is not completely working. But the accumulation of corrosion - fatigue fracture is not accompanied by decrease of kinetic endurance limit, which leads to the intensification of the process and the gradual increase of sensitivity to cycle asymmetry to typical cyclic fatigue level. Thus, for stress cycles below the fatigue limit, it is possible to make a model of the linear reduction factor in the sensitivity depending on the load level. The correctness of this model is confirmed by the fact that at low stress amplitude the damage line in Haigh diagram needs to be reborn in the x-axis.

As it is known, the drill string is quite often subjected to the actions of asymmetrical stress cycles with high amplitudes, even to the level of yield stress, for example during the elimination of sticking [13]. Even a small amount of stress is necessary to be considered for the calculation of longevity. For such stress cycles, on the contrary to low amplitude loading, there is an increased sensitivity to asymmetry. The level of damage is primarily controlled by the maximum stress of the cycle. Fig. 2 shows that the coefficient of sensivity of the asymmetry to a high level of load increases to unity for a single fracture.

So, according to the damaging effects of asymmetrical load on the drill and rod columns, three areas should be distinguished: low amplitude, high amplitude and the area of conventional multi cyclic corrosion fatigue. For normal stress cycles, for example, in fig. 5a it is located above the border line. The use of equation (4) for cycles $a - 1 < R \leq 0$ and (10) for $R > 0$ is recommened.

The corresponding equations for the other two regions are to be derived.

Assuming that low amplitude asymmetric cycle with a coefficient of skewness is to be reduced to the equivalent symmetric cycle $R \geq 0$ (point A in Fig.6).

In Fig. FED – is the line of endurance boundaries. Zero cycle equation (9) is used. Further enforcement conduct is a subject to linear reduction of the sensitivity coefficient to cycle asymmetry ψ_B depending on the load level

$$\psi_B = \psi \frac{OB}{OE} = \psi \frac{\sigma_{max}(B)}{\sigma_0} = \psi(1+\psi)\frac{\sigma_b}{\sigma_{-1}} \cdot \frac{\psi_1}{1+\psi_1} \quad . \tag{12}$$

Considering geometrical maintenance of coefficient sensitiveness to asymmetry of cycle in Haigh diagram according to (9), the equalization is obtained (13)

$$\psi_B = -\frac{y(C)-y(B)}{x(C)-x(B)} = \frac{\sigma_{ekv} - 0{,}5\sigma_{max}(B)}{0{,}5\sigma_{max}(B)} \quad . \tag{13}$$

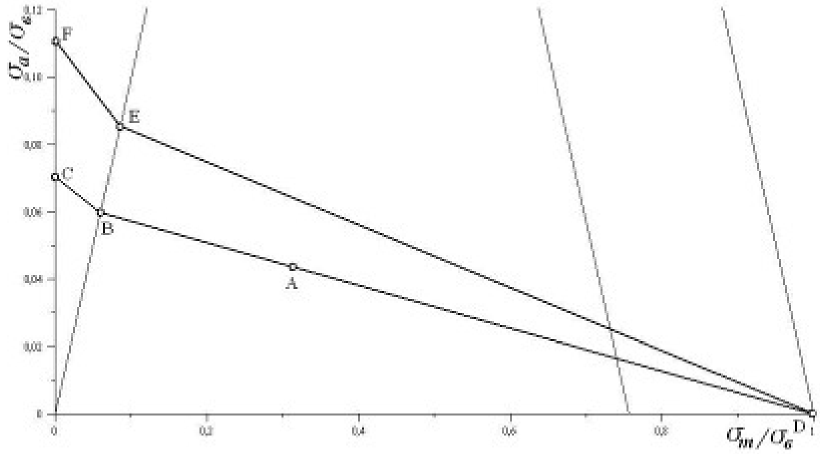

Figure 6 – A chart of bringing the amplitude asymmetric cycle with a coefficient of skewness $R \geq 0$

Thus it yields the final equation

$$\sigma_{ekv} = \sigma_b \psi_1 \frac{1+\psi_B}{1+\psi_1} \ . \tag{14}$$

where ψ_1 is determined by (10), and ψ_B from (12).

Given low amplitude asymmetric cycle with a coefficient of skewness $-1 < R < 0$ (the point A in Fig.7) is reduced to the equivalent symmetric cycle.

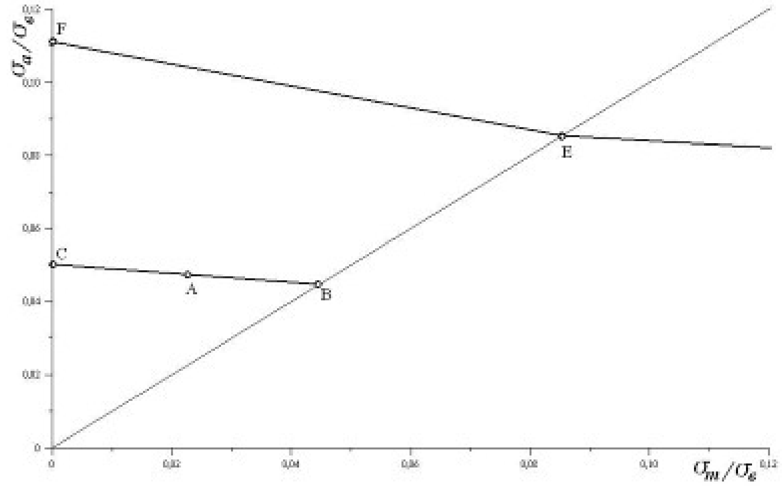

Figure 7 – Scheme of bringing the small-amplitude asymmetric cycle with a coefficient of skewness *-1 < R < 0*

In this case

$$\psi_B = -\frac{y(A)-y(B)}{x(A)-x(B)} = \psi \frac{x(B)}{x(E)} .$$

If $\psi = \psi\ x(B)/x(E) = k \cdot x(B)$ and $x(B) = y(B)$, the resulting quadratic equation with unknown $x(B)$. The solution has the form

$$x(B) = -\frac{k \cdot x(A) - 1 \pm \sqrt{[1-k \cdot x(A)]^2 - 4k \cdot y(A)}}{2k} .$$

The analysis shows that the physical sense has the solution with the sign « + ».

Given (13), it is finally obtained

$$\sigma_{ekv} = k \cdot x(B) \cdot (1 + x(B)), \qquad (15)$$

where $x(B) = -\dfrac{k \cdot x(A) - 1 + \sqrt{[1 - k \cdot x(A)]^2 - 4k \cdot y(A)}}{2k}$;

$k = \psi \dfrac{1+\psi}{\sigma_{-1}}$;

$x(A) = \sigma_{max} \dfrac{1+R}{2}$;

$y(A) = \sigma_{max} \dfrac{1-R}{2}$.

In the case of high-amplitude load $\sigma_{max} \geq \sigma_m$ it is got the same equation cast with just a little difference in definition ψ_B. From the condition of sensitivity coefficient linear increase to load level cycle asymmetry of, the following equation is

$$\psi_B = \psi + \frac{\sigma_{max}(B) - \sigma_m}{\sigma_b - \sigma_m}(1 - \psi). \qquad (16)$$

To justify the proposed method of asymmetric construction of the curves -1 < R < 1 the object of study was 40XH steel, which is used as the material of the tool joints of drill pipes.

The results of the study by V. Ivasi for samples of steel 40XH yielded such parameters of fatigue curves [14]:

σ_{-1} = 408 MPa; V_{-1} = 29,82 MPa;
σ_0 = 662 MPa; V_0 = 54,91 MPa;
$N_0 = 2 \cdot 10^6$ cycle; ψ = 0,22·

Fig. 8 shows the curves based on experimental studies as well as the curves constructed according to equations (4) and (10).

As it can be seen, the results correlate quite strongly. It demonstrates the effectiveness of the developed method of bringing asymmetrical stress cycles to equivalent destructive actions and the fatigue curves parameters determination under asymmetric loads.

To assess the reliability of the suggested casting method and other critical structural elements operating under conditions of corrosion fatigue, the results have been analyzed on samples of steel 17G1S. The parameters of fatigue curves are:

σ_{-1} = 141,9 MPa; V_{-1} = 30,87 MPa;
σ_0 = 247,1 MPa; V_0 = 51,83 MPa;
$N_0 = 5{,}207 \cdot 10^5$ cycle; ψ = 0,209·.

Fig. 9 shows curves 1 and 2, constructed in accordance with the specified parameters according to the equation [12]

$$N = N_0 \cdot ln\left\{1 + \left[exp\left(\frac{\sigma_{max} - \sigma_R}{V_R}\right) - 1\right]^{-1}\right\},$$

as well as the curves 3 and 4, obtained by casting using Oding equation $\sigma_{np} = \sqrt{\sigma_{max} \cdot \sigma_a}$ [9] and equation (4), respectively.

Special software has been developed for such 'reversed' curves.

The suggested method is almost fully consistent with the results of the experiment in contrast to the widely applied method using the Oding equation.

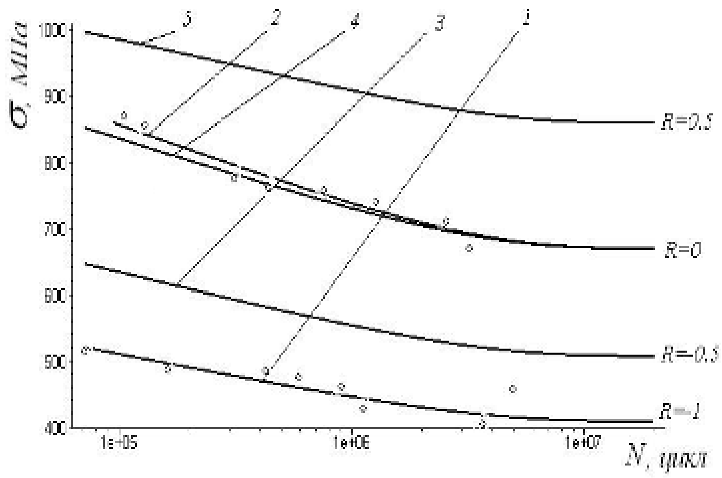

Figure 8 – Curves for the samples on steel 40XH:
1 – is experimental for symmetrical loading; 2 – is experimental with a pulsating load;
3 - is given by equation (4) (R=0); 4 – is given by equation (4) (R=-0,5);
5 – is given by equation (10) (R=0,5)

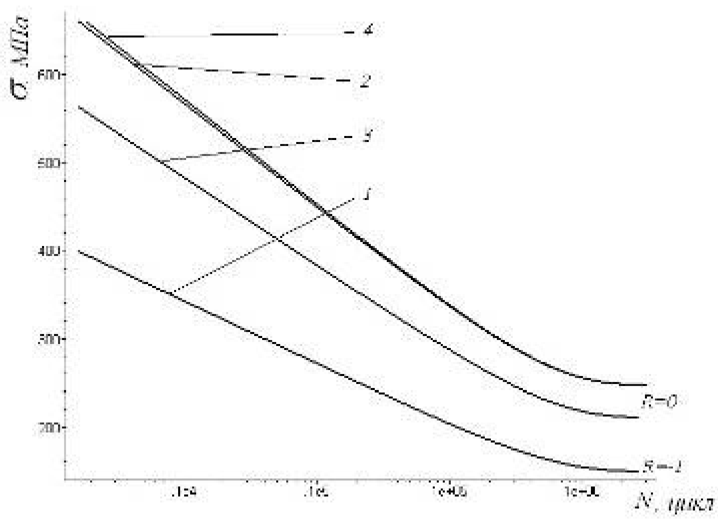

Figure 9 – Curves for samples of steel 17G1S [7, 15]:
1 – experimental for symmetrical loading; 2 – experimental with pulsating loads ;
3 – given in accordance with the Oding equation; 4 is given by equation (4)

Conclusions. Using the developed equations and software it can be built curves with symmetric loading and the coefficient of sensitivity to determine the parameters exactly. It is only necessary to consider the parameters of fatigue curves under symmetrical loads.

It greatly decreases the number of costly and time-consuming experimental studies that are required to assess the durability of the drill stem elements, operating in conditions of asymmetrical loading with mean stress of stretching. In the process of analyzing the load of the drill string at the stage of its reduction to an equivalent symmetric process there should be asymmetrical voltage range of the load.

The following research will focus on identifying features of the load during lowering-lifting operations in deep drilling using computer modeling and experimental studies of the durability of natural samples at high asymmetrical loads.

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LOADING OF STRUCTURAL ELEMENTS OF LARGE-SIZED ROTATING AGGREGATES DURING LONG-TERM OPERATION

Methods for determining the teeth total skew angle crown engagement considering errors from manufacturing and from mutual arragement of pinion and coronal wheel of an open gearing are implemented. On the basis of experimental data, the possible range of the total skew angle has been determined for different conditions under which large-sized rotating aggregates are operated.

Algorithm and techniques of calculation operated for a long time rotating aggregates during transitive and steady-state regimes of electromechanical drive mechanism operation, non-uniformity of the turn angle of the coronal wheel were considered. Practical recommendations for design improvement of an open gearing and for a system of kinematic parameters adjustment of large-sized rotating aggregates drive mechanism are suggested.

Keywords: *rotating aggregate, gear crown wheel and pinion, drive mechanism, skew angles of teeth, weared out involute profil.*

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НАВАНТАЖЕННЯ КОНСТРУКТИВНИХ ЕЛЕМЕНТІВ ВЕЛИКОГАБАРИТНИХ ОБЕРТОВИХ АГРЕГАТІВ ПІД ЧАС ТРИВАЛОЇ ЕКСПЛУАТАЦІЇ

Реалізовано методику для визначення сумарного кута перекосу зубців вінцевої пари з врахуванням похибок виготовлення і взаємного розташування зубчастих коліс відкритої передачі. На основі експериментальних даних визначено можливий діапазон сумарного кута перекосу за різних умов експлуатації великогабаритних обертових агрегатів. Розроблено алгоритм і методику розрахунку навантаження конструктивних елементів тривало експлуатованих обертових агрегатів під час перехідних і стаціонарних режимів роботи електромеханічного приводного механізму з урахуванням нерівномірності кута повороту вінцевого колеса. Запропоновано практичні рекомендації з удосконалення конструкції відкритої зубчастої передачі і системи регулювання кінематичних параметрів приводу великогабаритних обертових агрегатів.

Ключові слова: *обертовий агрегат, зубчасті вінцеве колесо і шестерня, приводний механізм, кути перекосу зубців, зношений евольвентний профіль.*

Formulation of the problem. Large-dimensional rotating aggregates, such as rotary kilns, tube mills, drying drums, etc. are widely used in the construction, mining, power, metallurgy and chemical industries. The desire to get greater productivity of technological lines has led to increase in the overall dimensions of their casing and the required power of electric motors for its rotation. In such technological lines one or two-motor electro-mechanical drive with open gear (crown pair) whose module is within the range 20 ... 60 mm and the pitch diameter of a crown wheel is up to 12000 mm is used. In this case, installation and repair work become more complicated, and the operating conditions deteriorate: it concerns fastening of the crown wheel to the steel case and the housing of the crown pinion to the basement, roller supports, bandages and other components of the rotating assembly. Insufficient reliability of individual structural elements of the electromechanical system leads to unplanned stops of the technological line, which adversely affects the utilization rate of the equipment and reduces the expected effect from the use of large-sized aggregates.

The practice of electromechanical drive mechanisms long-term operation of large-sized rotating aggregates testifies that one of the least reliable links in this case is the assembly of an open gear pair [1]. Thus, the problem of determining loads in the structural elements of drive mechanisms of large-sized aggregates and increasing their durability due to design, technological and operational measures in the course of their long-termed operation is an urgent task.

Analysis of recent research and publications. The complexities of the technological processes that accompany the long-term operation of rotating aggregates of continuous operation are covered in the works [2, 3]. The authors [4, 5] consider the problem of determining the loads of large-sized electromechanical drive mechanisms elements of rotating aggregates. Advantages of using the finite element method in various studies of the theory of gear mesh are given in [6]. The authors [7] conducted a study of the change in the wheels load capacity of the toothed pair for different variants of their geometric sizes. The modelling of the toothed pair coupling was carried out using the finite element method. As a result of the research, the method of increasing the load capacity due to the value of the module and the number of contacting wheels teeth is suggested. Issues of quasi-static analysis in gear pairs under the action of a torque momentum are investigated in work [8]. In the process of large-sized rotating aggregates operation due to specific conditions of their work there is intensive wearing of the open drive transmission teeth, which causes extra dynamic loads in the engagement [9]. The authors [10] have established that in the two-motor drive mechanism of the drum mills, there is forced oscillations emerge due to the accumulated error of the pitch in the open gear transmission. The paper [11] presents the study results of the time dependences of the normal force in the engagement and the change in velocity. From the obtained graphs, it is seen that the velocity of the wheel varies about a certain mean value.

In the conducted studies [12], it is shown that the non-uniform distribution of the load over the length of the contact lines of the conjugate teeth has the most significant impact on the accuracy of the assembly work, elastic deformation of the structural elements and the butt beating of the tooth crown. It is established that the distribution of the load over the width of the teeth crown of the cylindrical transmission is linear. To determine the structural elements rigidity of the ball mill drive electromechanical system, authors [13] used a two-mass calculation model, calculated analytical parameters determining the rigidity of gears, drive shafts, and elastic couplings of drum ball mills. The studies results of displacements in the cross sections of the roller-supports of large-sized rotary kilns are given [14]. Calculations show that the elastic displacement of the kilns body can be a significant part of its total displacement, which negatively affects the operation of the open gearing. As an alternative to the existing structures of drive mechanisms of rotary units, it is possible to note the patents [15, 16] and the author's certificate [17], where the crown wheel and the pinion are made with two additional support rollers at their butts which contact each other and maintain a constant radial clearance in the engagement of the open gearing wheels.

Unsolved earlier part of the general problem. The conducted analysis of studies and publications showed that during long-term operation of large-sized rotating aggregates there are many problems that are associated with their design and operational features. Preferably, research relates to drive mechanisms with closed gearings. There is much less attention paid to the study of drives with large-modal open gear transmission operation, although such mechanisms are operated under more heavy conditions. A number of new designs of rotating aggregates drive mechanisms are suggested [15-17], but due to the large material and time costs required for modernization, as a rule, their implementation in the production process is not carried out.

Setting objectives. The development of the algorithm and the technique of calculating the loads of structural elements of long-term continued operated large-sized rotating aggregates, considering the errors of manufacturing, installation, parameters of their operation, and electromagnetic processes in the electric motors of the drive mechanism were performed.

Main material and results. The electromechanical part of large-sized rotating aggregates consists of a body with mounted bandages rested on rollers. The torque moment is transmitted from the main electric motor to the main gear unit, the coupling, the pinion gear, the teeth of which are in the engagement with the crown wheel teeth, which is secured rigidly with the help of special rods to the body of the rotating assembly. Under the conditions of installation, the crown wheel consists of two halves. For example, for a rotating unit of the diameter of the body equal to 5 m, the pitch diameter of the crown wheel is 7.74 m, the width of the tooth crown is 0.85 m and weighs is about 50 000 kg. During a long-term operation, the body of the rotating unit and, with it, the crown wheel simultaneously move in the vertical and horizontal planes due to the eccentricity of the fit of the crown wheel and the radial beats, the ellipticity of the near-bandages and support rollers, the non-straightness of the axis of rotation of the body of the rotating unit and its temperature deformations. Mounting and checking of the drive elements occurs during the complete stop of the technological line, and at that time the temperature of the housing of the rotating unit is equal to the ambient temperature. When the rotary unit is started in the technological regime as a result of temperature and force deformations, the mutual arrangement of the gear wheels of the crown pairs significantly changes.

The value of the total skew angle of the crown pair teeth depends on the accuracy of the manufacture and installation of the large module gears and wheels and the deviations of the axis of rotation of the unit from the straight line. The pinion and wheel of the crown pair have no common place, therefore, in the process of the rotating unit operation; their mutual arrangement can vary considerably. The estimation of the constructive and operational factors influence on the gear wheels work of the rotating aggregate crown pairs can be carried out with the help of the average probable value of their total skew angle.

The angle value of open transmission teeth technological skew is determined according the A. I. Petrusevich’s formula, which is given in the paper [12]

$$\gamma_T = \sqrt{\Delta\beta_p^2 + \Delta\beta_g^2 + \gamma_\delta^2 \cos^2 \alpha_w + \gamma_n^2 \sin^2 \alpha_w}\,, \tag{1}$$

where $\Delta\beta_p$ and $\Delta\beta_g$ are the deviations from the given direction of the gears teeth angle and wheels inclination respectively;

γ_δ and γ_n are the angles of displacement and non-parallel of the pinion and wheels rotation axes respectively;

α_w are the angle of engagement.

For a rotating unit with a rectilinear axle of the body and an axial beating of the crown equal to the angle ξ_1 of inclination of the open gear wheels is of

$$\gamma' \approx \sin\gamma' = \frac{\xi_1}{d_{w2}}, \tag{2}$$

where d_{w2} is the pitch diameter of the crown wheel.

Movement of the crown wheel with the curved axis of the unit with two-motor drive rotation is shown in Fig. 1, where e is the value of the deflection of the unit body rotation axis in the section, which coincides with the position of the median plane of the crown wheel, ξ_2 is the axial displacement of the crown wheel beating which is caused by distortion of the unit body rotation axis.

At this case, the angle of inclination of the open gear wheels axes is determined depending on the value of its deflection in the section where the crown wheel is installed (Figure 1)

$$\gamma'' \approx \frac{e}{L}, \tag{3}$$

where $L = CB$ is the distance from the proximity of the rotating unit body rollers to the median plane of the crown wheel.

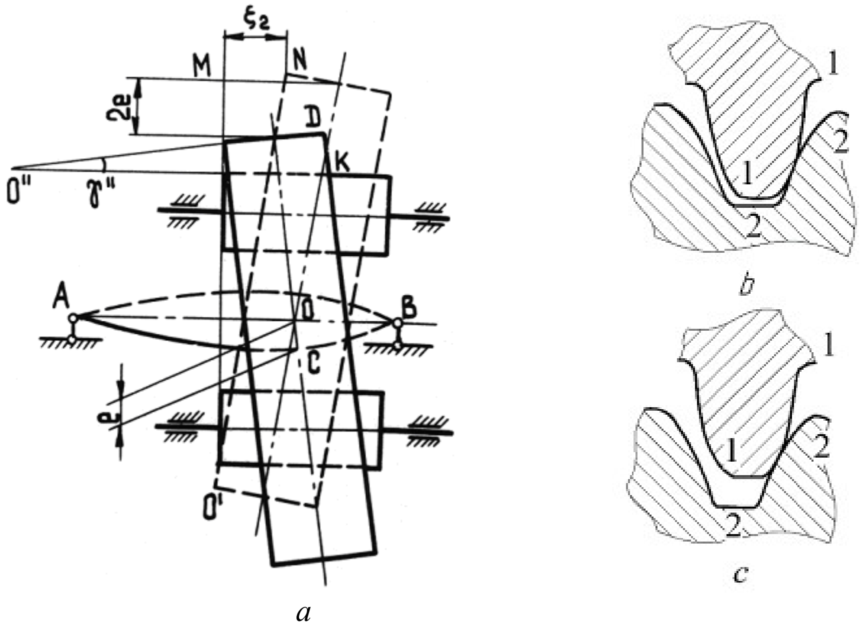


Figure 1 – Characteristic cases of the crown pair elements mutual arrangement in the case of the rotating aggregate body rotation curved axis

For the convenience of conducting further research, the angles of distortion the crown pair wheels axes are brought to the plane of engagement. Then, the dependencies for determining the variable components of the reduced angles of bias γ'_δ and the non-parallel γ'_n of the crown pair teeth from the axial beating of the crown ξ_1 in the function of rotation angle of the crown wheel ψ_κ have the form

$$\gamma'_\delta = \frac{\xi_1}{d_{w2}} \cos\alpha_w \sin\psi_\kappa; \tag{4}$$

$$\gamma'_n = \frac{\xi_1}{d_{w2}} \sin\alpha_w \cos\psi_\kappa. \tag{5}$$

Dependencies for determining the variables of bias reduced angles components γ''_δ and the non-parallelness γ''_n of the crown pair teeth from the value the unit body rotation axis deflection e in the function of rotation angle ψ_κ have the form

$$\gamma''_\delta = \frac{e}{L}\cos\alpha_w \cdot \sin\psi_\kappa; \tag{6}$$

$$\gamma''_n = \frac{e}{L}\sin\alpha_w \cdot \cos\psi_\kappa. \tag{7}$$

Substituting (4) - (7) into (1), it is obtained the formula for determining the total skew angle of the crown pair teeth

$$\gamma_\Sigma = \sqrt{\Delta\beta_p^2 + \Delta\beta_g^2 + \left(\frac{\xi_1^2}{d_{w2}} + \frac{e^2}{L^2}\right)\left(\cos^2\alpha_w \sin^2\psi_\kappa + \sin^2\alpha_w \cos^2\psi_\kappa\right)}. \tag{8}$$

In the course of experimental studies, the axial beating of the crown wheel and the deflection of the rotation axis of the rotating unit body from the straightness were determined. Investigation of the operating conditions of the drive mechanism at different values of the errors of manufacturing and assembling the elements of rotating aggregates have showed that in real conditions of operation the total skew angle of wheels crown pairs teeth may exceed the recommended maximum value more than two or three times, there may be cases of incomplete contact of crown pairs of teeth wheel width. For rotating aggregates of the sizes of 5×185 m, 4.5×170 m, 4×150 m, the value of the maximum total skew angle of the wheel teeth under certain operating conditions lies in the range of $4.0 \cdot 10^{-4}$ to $1.0 \cdot 10^{-3}$ rad.

All this indicates that in the process of rotating aggregates embedding of the crown pair teeth wheels long-term operation is not fully implemented. Due to the specific conditions of their work (open gear transmission, the presence of abrasive media, large non-uniform distribution of load over the length of contact lines, etc.), there is an intense wear of the teeth pinion and wheels (Fig. 2). This causes the unevenness of the rotary motion of the crown pair wheels, resulting in extra dynamic loads in the engagement.

a

b

Figure 2 – Profiles of the open gear wheels gear during their long-term operation:
a – the pinion and the crown wheel; b – pinion

The dependence of the variable component of the the crown wheel turn angle $\Delta\psi_\kappa$ in the function φ_m/u_2 of the reduced angle of the crown gear rotation during the time of one engagement of the conjugate teeth is approximated by the finite number of Fourier series terms [18]

$$\Delta\psi_K = A_0 + \sum_{j=1}^{S_i} E_{ij} \sin\left(jz_2 \frac{\varphi_m}{u_2} + \varepsilon_{ij} \right), \tag{9}$$

where $E_{ij} = A_{ij} + B_{ij}$ is the total coefficient of the Fourier series;

A_0, A_{ij}, B_{ij} are the coefficients of the Fourier series;

u_2 is the gear ratio of the gear wheels of the crown pair;

z_2 is the number of crown wheel teeth;

ε_{ij} is the initial phase j is its harmonics;

S_i is the number of terms of the Fourier series which is necessary to provide the required accuracy of the calculation.

The mutual relation of the crown pairs teeth wheels rotation angles with the worn out profile of the teeth has the form

$$\psi_K = \frac{\varphi_m}{u_2} + \Delta\psi_K. \tag{10}$$

Given the expression (10), we obtain

$$\psi_K = \frac{\varphi_m}{u_2} + A_0 + \sum_{j=1}^{S_i} E_{ij} \sin\left(jz_2 \frac{\varphi_m}{u_2} + \varepsilon_{ij} \right). \tag{11}$$

In this case, the variable transmission ratio of the wheels of the crown pairs of the worn out profile of the teeth has the form

$$\gamma = \left[\frac{1}{u_2} + \frac{1}{u_2} \sum_{j=1}^{S_i} E_{ij} jz_2 \, cos\left(jz_2 \frac{\varphi_m}{u_2} + \varepsilon_{ij} \right) \right]^{-1}. \tag{12}$$

To study the oscillation phenomena due to the kinematic errors of the crown teeth pair, due to the deviation of the teeth shape from the profile, the mechanical part of the rotating aggregates can be represented as a discrete calculation scheme (Fig. 3), where J_i*(*i=1, 2, …, m*)* are the moments of mass inertia of the drive mechanism; C_i*(*i=1, 2, …, m-1*)* are the elastic elements rigidity of the drive; v_i*(*i=1, 2, …, m-1*)* are the coefficients of linear resistance of the corresponding units. Metal construction of the rotating unit body by direct sampling is replaced by a set of masses with moments of inertia I_j*(*j=1, 2, …, n-1*)*; r_j*(*j=1, 2, …, n-1*)* and μ_j*(*j=1, 2, …, n-1*)* – respectively, the rigidity and coefficients of the linear resistance of the parts of the aggregate body; φ_i*(*i=1, 2, …, m*)* and ψ_i*(*j=1, 2, …, n*)* are the coordinates of movement of the corresponding masses of the drive mechanism and the body of the rotating aggregates.

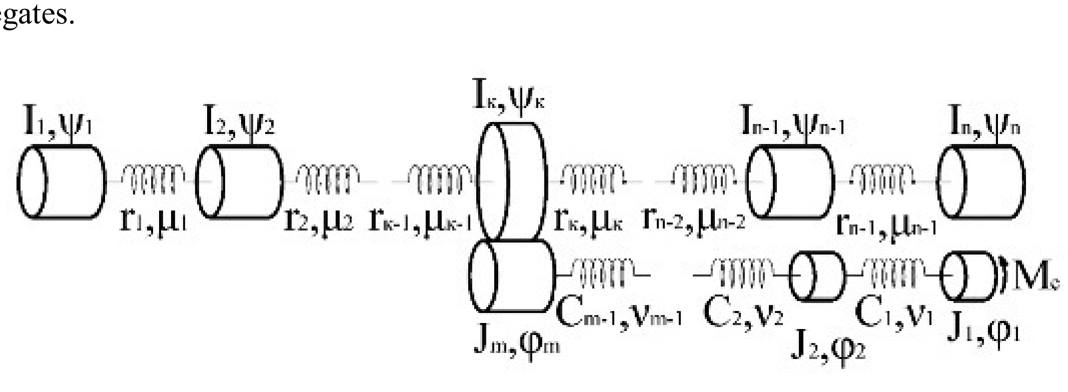


Figure 3 – Calculation scheme of a large-sized rotating aggregates

In the calculation scheme, the gear wheels of the crown pair are directly connected with the masses, which have moments of inertia J_m (for the pinion gear) and I_K (for the crown wheel). Assume that the values of the inertial, rigid and dissipative parameters of the drive elements are determined considering the reduction to the axis pinion gear rotation.

The equation rotating unit masses motion is made on the basis of the Lagrange equations of the second kind. Using expressions of kinetic and potential energies, Relay functions, and generalized forces, there can be obtained differential equations of motion.

For masses of the drive mechanism, these equations have the following form:

$$(J_1 p^2 + \nu_1 p + C_1)\varphi_1 - (\nu_1 p + C_1)\varphi_2 = M_e u_1; \tag{13}$$

$$\begin{gathered}[J_i p^2 + (\nu_i + \nu_{i-1})p + C_i + C_{i-1}]\varphi_i - (\nu_{i-1} p + C_{i-1})\varphi_{i-1} - (\nu_i p + C_i)\varphi_{i+1} = 0; \\ i = 2,3,...,m-1.\end{gathered} \tag{14}$$

where M_e is the electromagnetic moment of the motor;

u_1 is the gear ratio from the shaft of the pinion axle motor, which is equal to the gear ratio of the main gear unit;

p is the operator of differentiation with respect to time.

The equation of the open gear pair wheels motion directly is connected with the crown wheel of the rotating unit body section is represented by the dependence:

$$\begin{gathered}\left[\left(J_m + \frac{I_\kappa}{\gamma^2}\right)p^2 + \left(\nu_{m-1} + \frac{\mu_{\kappa-1}}{\gamma^2} + \frac{\mu_\kappa}{\gamma^2}\right)p + C_{m-1} + \frac{r_{\kappa-1}}{\gamma^2} + \frac{r_\kappa}{\gamma^2}\right]\varphi_m - \frac{2I_\kappa}{\gamma^3}\cdot\frac{d\gamma}{d\varphi_m}(p\varphi_m)^2 - \\ -(\nu_{m-1} p + C_{m-1})\varphi_{m-1} - \gamma^{-1}(\mu_{\kappa-1} p + r_{\kappa-1})\psi_{\kappa-1} - \gamma^{-1}(\mu_\kappa p + r_\kappa)\psi_{\kappa+1} = -\gamma^{-1}M_{OK},\end{gathered} \tag{15}$$

where M_{OK} is the moment of the forces resistant to motion, which is applied to the part of the rotating aggregates body connected with the crown wheel.

The equations of the body lumped masses motion of the rotating unit have the form:

$$(I_1 p^2 + \mu_1 p + r_1)\psi_1 - (\mu_1 p + r_1)\psi_2 = -M_{01}; \tag{16}$$

$$\begin{gathered}[I_i p^2 + (\mu_{i-1} + \mu_i)p + r_{i-1} + r_2]\psi_i - (\mu_{i-1} p + r_{i-1})\psi_{i-1} - \\ -(\mu_i p + r_i)\psi_{i+1} = M_{oi}; i = 2,3,...,\kappa-1,\kappa+1,...,n-1;\end{gathered} \tag{17}$$

$$(I_n p^2 + \mu_{n-1} p + r_{n-1})\psi_n - (\mu_{n-1} p + r_{n-1})\psi_{n-1} = -M_{on}. \tag{18}$$

where M_{oi} ($i = 1,2,...,\kappa-1,\kappa+1,\kappa+2,...,n$) are moments of motion resistance.

If the lateral gap in the engagement of the open gear is partially disclosed, the equation of motion of rigidity coupled masses with pinion and with a crown wheel takes the form

$$(J_m p^2 + \nu_{m-1} p + C_{m-1})\varphi_m - (\nu_{m-1} p + C_{m-1})\varphi_{m-1} = 0; \tag{19}$$

$$\begin{gathered}[I_\kappa p^2 + (\mu_{\kappa-1} + \mu_\kappa)p + r_{\kappa-1} + r_\kappa]\psi_\kappa - (\mu_{\kappa-1} p + r_{\kappa-1})\psi_{\kappa-1} - \\ -(\mu_\kappa p + r_\kappa)\psi_{\kappa+1} = -M_{OK}.\end{gathered} \tag{20}$$

It should be noted that considering non-rectlinearity of the body rotation axis and the specificity of the processed raw material movement along the axis of the rotating aggregate, the total moment of the body forces resistant to the motion is shown with the expression

$$M^*_{0\Sigma} = M_{0\Sigma} \cdot (1 + \varepsilon \sin \psi_\kappa), \tag{21}$$

where ε is the coefficient of resistance moment amplitude variation to the rotating aggregates body motion.

The dynamical properties of the asynchronous motor are considered by means of simultaneous integration of the rotating unit mechanical part motion equations with the equations of the motor electromagnetic state [19].

The mathematical model of an electric machine is described by the following equations

$$pi_s = a_s(u_s + \Omega_s\psi_s - r_s i_s + b_s(\Omega_R\psi_R - r_R \cdot i_R); \tag{22}$$

$$pi_R = b_R(u_s - \Omega_s\psi_s - r_s \cdot i_s) + a_R(\Omega_R\psi_R - r_R \cdot i_R). \tag{23}$$

In these equations, the subscripts s and R denote that the corresponding parameters are related to the stator or rotor windings, respectively i_s and i_R are the matrix-columns of projections of currents on to the coordinate axes x, y; u_S is the matrix-column of voltage source; ψ_s, ψ_R are the matrix columns of complete flow connections; Ω_s, Ω_R are the angular velocity matrices; a_s, a_R, b_s, b_R are the constant coefficients; r_s, r_R are the stator and rotor windings resistances, respectively.

The electromagnetic moment of the motor is found by the formula

$$M_e = \frac{3}{2} p_0 (i_{sy} \cdot i_{Rx} - i_{sx} \cdot i_{Ry}) / \tau_i, \tag{24}$$

where p_0 is the number of pairs of magnetic poles;

τ_i is the inverse working inductance of the motor.

Depending on the location of the two neighbour teeth of the crown wheels, the total angle of the crown pinion turn, relative to the axis the crown wheel rotation , considering the choice of the lateral gap in the engagement of the open transmission χ is determined by the expression

$$\psi_\kappa = \frac{\varphi_m}{\gamma} + \Delta\psi_\kappa + (q-1) \cdot \chi / \gamma, \tag{25}$$

where χ is the angle where the teeth of the crown pinion is turned to chose the lateral gap between the teeth of crown pair.

The torque M_m which is transmitted by the crown pinion determined by the formula

$$M_m = C_{m-1}(\varphi_{m-1} - \varphi_m) + \nu_{m-1} p(\varphi_{m-1} - \varphi_m). \tag{26}$$

Determining the loads that emerge in the elements of the mechanical system of the rotating aggregate under different operating conditions, there can be carried out the following algorithm:

Variant 1: in the case where the lateral gap between teeth of a crown gear pairs is completely chosen $q = 1$; $M_m > 0$.

The mathematical model of the drive system of rotating aggregates is formed from the equations (13) - (15), (16) - (18), (22), (23) considering the relations (12), (24), (25).

Variant 2: in the case where the lateral gap between the crown pairs is maximal $q = 2$ $M_m < 0$.

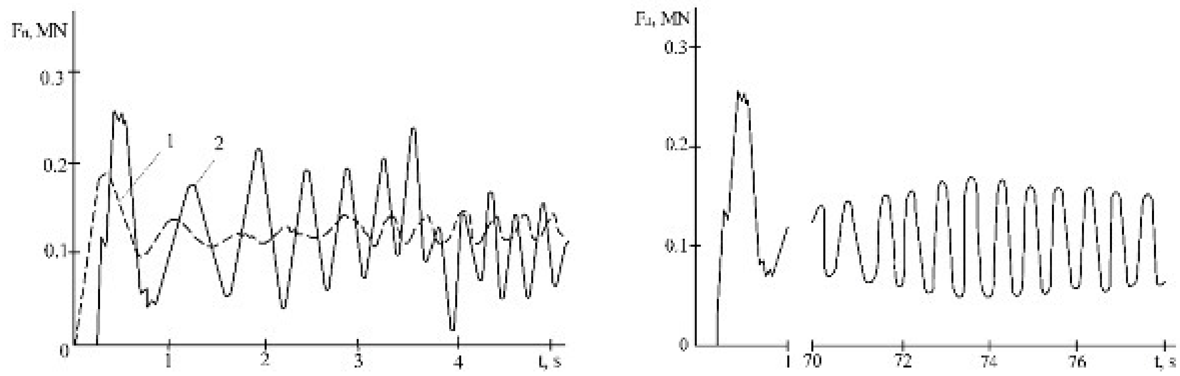
The mathematical model of the drive system of rotating aggregates is formed from the equations (13) - (18), (22) - (23) considering the relations (12), (24), (25).

Variant 3: in the case where the lateral gap between teeth of a crown pairs is partially chosen $1 < q < 2$; $M_m = 0$.

There is constructed a mathematical model from equations (13), (14), (16) - (23) considering the relations (12), (24), (25).

According to the variants number (1, 2 or 3), the system of the corresponding differential equations is determined. Their numerical integration is performed by the Runge-Kutta method. The program provides an opportunity to identify efforts of meshed wheels crown pairs, moments in elastic links, dynamic factors and output calculation results with a given time step. Using modern PC allows us to optimize the parameters of the drive mechanism according the criterion of minimum dynamic loads.

As an example, the drive system elements load of six-support rotating kiln Ø 4.0×150 m with roller support on roller bearings and welded bandages is considered. Technological frequency of the kiln body rotation, $n_b = 0.55 \ldots 1.4$ rpm. As an electric motor of the main drive, the motor AKZ-12-35-6 with a power of 320 kW and speed of $n_m = 980$ rpm are chosen. The main gearbox A 600×900×1400 with gear ratio of u_1 = 87,82. Parameters of open crown pair gear: module m = 45 mm; number of teeth $z_1 = 19$, $z_2 = 150$.



a *b*

Figure 4 – Graphical dependences of the normal force change in the engagement of the open gear crown pair transmission of the drive mechanism of the rotating kiln Ø 4.0×150 m

Fig. 4, *a* and Fig. 4, b show the dependences of the normal force change in the engagement of the open gear crown pair transmission of the rotation kiln drive mechanism Ø 4.0×150 m during the transition and stationary modes of operation. Curve 1 in Fig. 4 a, represent the dependence of the change in the normal force in the engagement of an open gear crown pair with an ideal involute profile, the teeth ($\chi = E = 0$), the curve 2 in Fig. 4, *a* – provided that the worn-out involute profile of the teeth is when $\chi = 0.004$ rad and $E = 0.0005$ rad. Comparison of the graphic dependencies 1 and 2 in Fig. 4 *a* shows that the increase of the angle χ and the uneven angles of wheels rotation of the crown pairs for the considered case lead to an increase in the normal force in their toothed engagement by 27%.

Fig. 4, b shows the dependence of the normal force change on the engagement of an open gear crown pair during stationary mode under condition of a worn-out involute profile of the teeth when $\chi = 0.004$ rad and $E = 0.0005$ rad. From Fig 4, b, it can be seen that in the stationary mode of the rotating kiln at lower frequencies of the rotation of its housing, which is required in accordance with the technological regime of clinker annealing, resonant states may occur in the drive elements. It is due to the fact that the range of the rotational speed of the kiln body according to the technological regime of the clinker annealing lies in the limits of 0.55…1.4 rpm, which corresponds to the range of the crown wheel pair frequency which lies within (1.3…3.5) Hz. At the same time, the frequency teeth of the crown pairs of wheels can coincide with one of the inner frequencies of the rotating aggregate mechanical system.

Conclusions. 1. Analysis of structures and operational conditions for traditional drive mechanisms of large-sized rotating aggregates have indicated that the total skew angle open gearing teeth can reach the value of $4{,}0\cdot10^{-4}$ rad, sometimes up to $1{,}0\cdot10^{-3}$ rad. During long-term operation, embedding of crown gearing wheels teeth is not complete. This causes great non-uniformity of load distribution along lines of contact, intensive wheel and pinion teeth wearing out, and extra dynamic loads of the structural elements rotating aggregate.

2. Algorithm and technique of structural elements calculation of operated for a long time large-sized rotating aggregates are suggested. According to this technique, the determination of loading in mesh of an open gearing and torque moments in elastic links is conducted considering the mutual influence of mechanical and electromagnetic oscillation phenomena. The kinematic parameters of open gearing are presented in terms of non-linear dependences considering the errors from manufacturing and the parameters of wearing out.

3. The results of investigations, which have been carried out by means the numerical modelling method and by means of experiments, have indicated that during operation of large-sized rotating aggregates resonance states can emerge in elements of their drive mechanism at the expense of the open gearing teeth re-conjugation frequency coincidence with one of the lower natural frequencies of the mechanical system. It is expedient to use of transducers for monitoring the vibroparameters of the drive mechanism, the transducers signalize for correcting into the aggregate body rotary speed smooth adjustment system. Carrying out the adjustment of the kinematic parameters of the driving mechanism in due time makes it impossible to operate large-sized rotating aggregates in resonance regime.

4. The application of the drive mechanism design of large-sized rotating aggregates with mobile «floating» pinion of an open gearing is in prospect. In such a drive mechanism, the pinion and the crown wheel are made with two additional rollers at their butts. The additional rollers with the help of elastic elements are pressed against each others. The application of such drive mechanism design enables to maintain more constant mutual position of the wheels of an open gearing during its long-term operation, including such parameters as interaxial distance, lateral and radial gaps.

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COATINGS DISCRETE SURFACES CONSTRUCTION BY SUPERPOSITIONS OF ADJUSTED MESH FRAMES

The method of curve surface discrete geometric modeling on the basis of two discrete frames superimpositions, formed by a static-geometric method, and on the basis of a single surface curve nodal points superimposition, also formed by static-geometric method, is considered. It has been determined that the suggested method allows to model balanced discrete structures formed on the specified contour nodes, as well as those passing through the specified nodal points without composing and solving equations systems.

***Keywords:** discrete geometric modeling, discrete surface frame, coating surfaces, static-geometric method, the external shaping load value, geometric apparatus of superimpositions, coefficients of superimposition.*

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ФОРМУВАННЯ ДИСКРЕТНИХ ПОВЕРХОНЬ ПОКРИТТІВ НА ОСНОВІ СУПЕРПОЗИЦІЙ ЗАДАНИХ СІТЧАСТИХ КАРКАСІВ

Розглянуто спосіб дискретного геометричного моделювання кривих поверхонь на основі суперпозицій двох дискретних каркасів, сформованих статико-геометричним методом, та на основі суперпозицій вузлових точок однієї кривої поверхні, сформованої також статико-геометричним методом. З'ясовано, що запропонований спосіб дозволяє моделювати врівноважені дискретні структури, сформовані на заданих контурних вузлах, а також ті, що проходять через задані вузлові точки без складання і розв'язання систем рівнянь.

***Ключові слова:** дискретне геометричне моделювання, дискретний каркас поверхні, поверхні покриттів, статико-геометричний метод, величина зовнішнього формоутворюючого навантаження, геометричний апарат суперпозицій, коефіцієнти суперпозиції.*

Introduction. Geometric designing is important for designing modern building structures, architectural forms of coatings, when the main geometric shapes are determined at the sketch stage with their advantages and disadvantages.

An expedient way to obtain the medial coating surfaces is inefficient. A surface must satisfy the pre-set conditions and requirements that are often competing with each other.

Calculations of such structures considering material physical properties, manufacturing technology and installation features require information about the object presented in the discrete form, therefore it is advisable to form them in the discrete form in the very beginning.

Discrete geometric modeling is the most promising trend in the applied geometry development in the current period, which can be conventionally divided into studies on the discretization of continuous geometric images and the shaping based on discrete source data [1, 2].

The static-geometric method of discrete geometric modeling of curved lines and surfaces [3] allows obtaining discrete frames of curvilinear surfaces under the influence of external shaping loads and, moreover, it is simple and descriptive. Using the static-geometric method allows obtaining discrete frames of curvilinear surfaces on an arbitrary reference contour.

Analysis of recent research sources and publications. The basis of the static-geometric method mathematical apparatus is solution of cumbersome linear equations systems, which complicates the process of computer-implemented calculations. Publication [4] is devoted to the issue of expanding the shape-forming possibilities of the static-geometric method by means of the numerical sequences mathematical apparatus, which permits, in particular, to avoid composing of linear equations systems in the discrete images formation.

In publication [5], the problems of the discrete surfaces frames construction by functional addition based on two pre-calculated frames by means of the static-geometric method were studied. However, grids verification obtained as a result of the initial grids superimposition with different braced stretching coefficients showed that the resulting grid is not balanced with a set of external load on the nodes, thus the results of such superimpositions are not accurate but approximate.

In study [6] the notion of the sets superimposition apparatus in applied geometry is defined. A number of properties have been proved permitted to draw conclusions about the deep comprehensive studies prospect concerning the superimposition apparatus.

The articles reports [7-11] show the approaches to determination of certain functional dependences discrete analogues based on the superimpositions geometric apparatus of one-dimensional point sets permitting the formation of discrete images without composing and solving cumbersome equation systems. The shaping control of discretely submitted curves is performed by varying the superimposition coefficients values.

Identification of previously unsettled parts of the general problem. The classical finite difference method, static-geometric method, mathematical apparatus of numerical sequences have their advantages and disadvantages concerning the solution of specific practical tasks. Therefore, their studies, enrichment with new efficient algorithms, studying the possibility of their compilation, and on this basis, expansion of the output data set, are topical. The above methods further development and improvement in general are topical as well. Using the geometrical superimpositions apparatus in combination with the above-mentioned methods permits to improve significantly the efficiency and to extend the continuous geometric images discrete modeling possibilities.

Setting objectives. The purpose of the present article is to study the method of two-dimensional geometric image modeling in the form of discretely presented curved surface by means of the superimpositions geometric apparatus of one-dimensional point sets based on a single curved surface formed using the static-geometric method.

Main material and results. It is supposed the surface differential equation specified as $z = x^2 + y^2$ (fig. 1) : $\frac{\delta^2 z}{\delta x^2} + \frac{\delta^2 z}{\delta y^2} = 4$ for a limited area $-2 \le x \le 2$; $-2 \le y \le 2$, and the marginal conditions are set as four lines:

$$\begin{array}{ll} 1.\ y = 2\ ; & z = x^2 + 4\ ; \\ 2.\ y = -2\ ; & z = -x^2 + 4\ ; \\ 3.\ x = 2\ ; & z = y^2 + 4\ ; \\ 4.\ x = -2\ ; & z = -y^2 + 4\ . \end{array} \quad (1)$$

It is determined discrete point surface frame in the specified area with step $h = 1$.

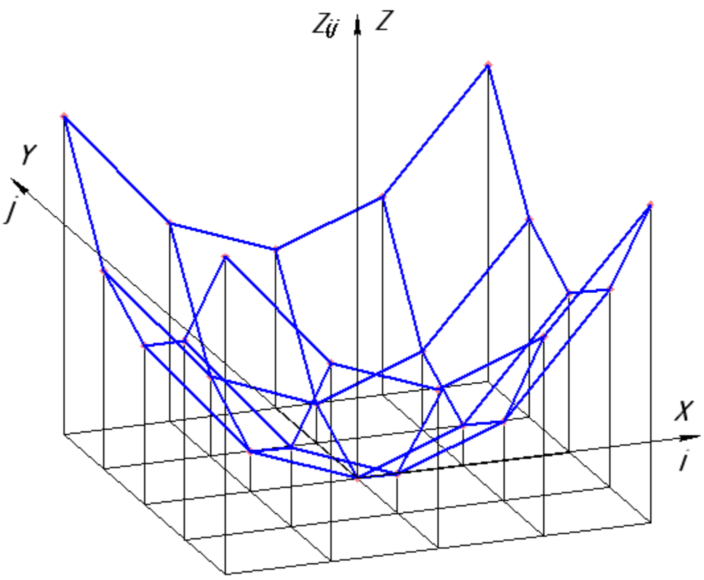

Figure 1 – Discretely presented surface $z = x^2 + y^2$.

According to dependences (1) the applicative points values are determined on the surface boundary lines with step h = 1. In the Table 1 they are located in the boxes surrounded with a heavy line.

Table 1 – Values of surface applicate points

	$i = -2$	$i = -1$	$i = 0$	$i = 1$	$i = 2$
$j = 2$	8	5	4	5	8
$j = 1$	5	2	1	2	5
$j = 0$	4	1	0	1	4
$j = -1$	5	2	1	2	5
$j = -2$	8	5	4	5	8

Let us replace the differential equation with the finite differences expression

$$z_{i-1,j} + z_{i+1,j} - 4z_{i,j} + z_{i-1,j} + z_{i+1,j} - 4 = 0 \quad (2)$$

and compose the system (3) of finite differences equations for all unidentified nodes of the area.

$$\begin{cases} z_{10} + z_{-10} - 4z_{00} + z_{01} + z_{0-1} - 4 = 0 \\ z_{20} + z_{00} - 4z_{10} + z_{11} + z_{1-1} - 4 = 0 \\ z_{21} + z_{01} - 4z_{11} + z_{12} + z_{10} - 4 = 0 \\ z_{11} + z_{-11} - 4z_{01} + z_{02} + z_{00} - 4 = 0 \\ z_{01} + z_{-21} - 4z_{-11} + z_{-12} + z_{-10} - 4 = 0 \\ z_{00} + z_{-20} - 4z_{-10} + z_{-11} + z_{-1-1} - 4 = 0 \\ z_{0-1} + z_{-2-1} - 4z_{-1-1} + z_{-10} + z_{-1-2} - 4 = 0 \\ z_{1-1} + z_{-1-1} - 4z_{0-1} + z_{00} + z_{0-2} - 4 = 0 \\ z_{2-1} + z_{0-1} - 4z_{1-1} + z_{10} + z_{1-2} - 4 = 0 \end{cases} . \quad (3)$$

Considering the initial data symmetry, the system of equations can be significantly reduced by writing the symmetry conditions

$$z_{01} = z_{10} = z_{-10} = z_{0-1} ; \quad z_{11} = z_{-11} = z_{-1-1} = z_{1-1} .$$

It is substituted the marginal contour applicative-nodes values and the symmetry conditions into the system (3):

$$\begin{cases} 4z_{10} - 4z_{00} - 4 = 0 \\ 4 + z_{00} - 4z_{10} + 2z_{11} - 4 = 0 \\ 8 + 2z_{10} - 4z_{11} - 4 = 0 \end{cases} . \quad (4)$$

The results of solving the system (4) are presented in the Table 1.

To form the surface discrete frame by the static-geometric method, the model of the given surface in the form of a discrete grid with a uniform step $h = 1$ along the axis Ox: $x_{i+1} = x_i + h$; and the axis Oy: $y_{i+1} = y_i + h$; can be imagined as being balanced by certain external efforts in nodes and forces in the braces so that these efforts are directly proportional to these braces lengths (Fig. 2).

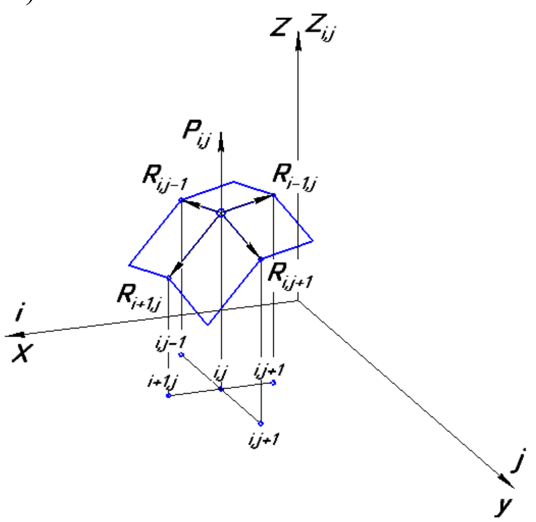

Figure 2 – Diagram of the balanced discrete grid nodes

Then it can be determined the external effort KP_{ij} that balances the effort $R_{i-1,j}$, $R_{i+1,j}$, $R_{i,j-1}$, $R_{i,j+1}$ of the respective braces

$$\overline{P}_{i,j} = \overline{R}_{i-1,j} + \overline{R}_{i+1,j} + \overline{R}_{i,j-1} + \overline{R}_{i,j+1} . \quad (5)$$

For the type II grid [12] with the reference system of nodes, as it is shown in Fig. 2, the equation (5) in the coordinate form is:

$$u_{i-1,j} + u_{i+1,j} - 4u_{i,j} + u_{i,j-1} + u_{i,j+1} + KP_{i,j} = 0 \, , \tag{5}$$

where u is a generalized designation of the respective coordinate.

Thus, to form the discrete frame of the surface shown in Fig. 1 by the static-geometric method, it is necessary to compose and solve a system of equations on the internal nodes equilibrium:

$$\begin{cases} 4z_{10} - 4z_{00} - KP_{ij} = 0 \\ 4 + z_{00} - 4z_{10} + 2z_{11} - KP_{ij} = 0 \\ 8 + 2z_{10} - 4\,z_{11} - KP_{ij} = 0 \end{cases} , \tag{7}$$

where KP_{ij} is the value of the external shaping load.

It is considered an example of discrete surface model formation based on two discrete surface frames superimpositions formed by the static-geometric method in a single reference contour (Fig. 3).

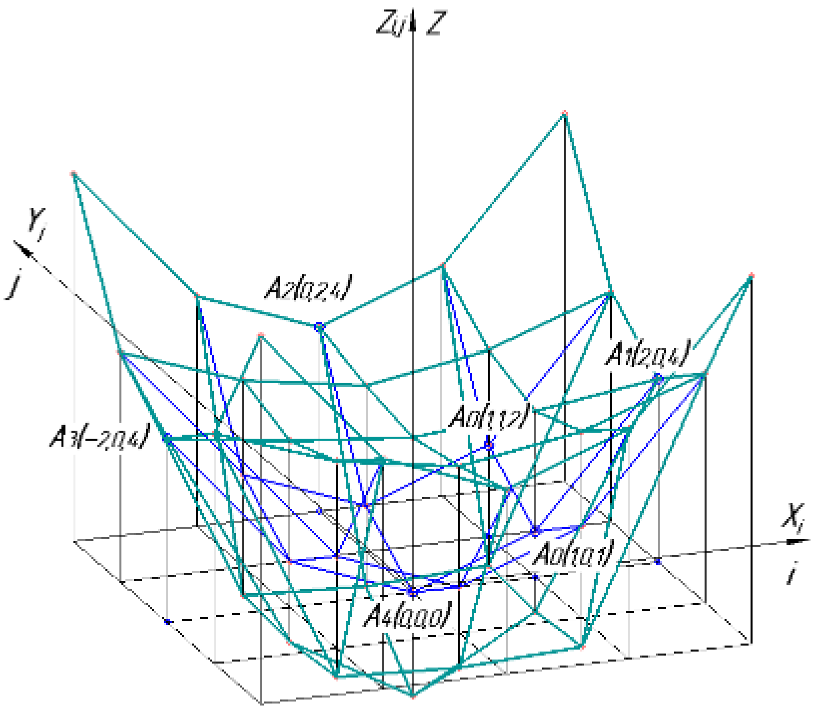

Figure 3 – Formation of discrete surface models based on superimpositions of two discretely determined surfaces.

Discrete values of the first curved surface are formed basing on the initial data $z_{A1} = 4$, $z_{A2} = 4$, $z_{A3} = 4$, $z_{A4} = 4$, $P_{ij} = -1$.

The equation system (7) is:

$$\begin{cases} 4z_{10} - 4z_{00} - 1 = 0 \\ 4 + z_{00} - 4z_{10} + 2z_{11} - 1 = 0 \\ 8 + 2z_{10} - 4\,z_{11} - 1 = 0 \end{cases} .$$

This system solution results in: $z_{00}=27/8$, $z_{10}=29/8$, $z_{11}=65/16$;

The discrete values of the first surface application formed by the static-geometric method in the specified reference contour and by the value of the external shape-forming load $P_{ij} = -1$, specified and uniformly distributed between the discrete frame nodes, are presented in Table 2.

Table 2 – Application points values of the first surface with the load value of P_{ij} = –1

	i = –2	i = –1	i = 0	i = 1	i = 2
j = 2	8	5	4	5	8
j = 1	5	65/16	29/8	65/16	5
j = 0	4	29/8	27/8	29/8	4
j = –1	5	65/16	29/8	65/16	5
j = –2	8	5	4	5	8

Statement. Any point coordinates of two-dimensional set of points are the coordinate superimposition (8) of this set four arbitrary points

$$\begin{cases} x_0 - x_4 = k_1\,(x_1 - x_4) + k_2\,(x_2 - x_4) + k_3\,(x_3 - x_4) \\ y_0 - y_4 = k_1\,(y_1 - y_4) + k_2\,(y_2 - y_4) + k_3\,(y_3 - y_4) \\ z_0 - z_4 = k_1\,(z_1 - z_4) + k_2\,(z_2 - z_4) + k_3\,(z_3 - z_4) \end{cases}. \tag{8}$$

It can be computed the superimposition coefficients values for three specified points of the reference contour $A_1(2;\,0;\,4)$, $A_2(0;\,2;\,4)$, $A_3(-2;\,0;\,4)$ and the central node $A_4(0;\,0;\,3{,}375)$ to determine the coordinates of point $A_0(1;\,0;\,3{,}625)$.

Then the system of equations (8) is

$$\begin{cases} 1 - 0 = k_1\,(2 - 0) + k_2\,(0 - 0) + k_3\,(-2 - 0) \\ 0 - 0 = k_1\,(0 - 0) + k_2\,(2 - 0) + k_3\,(0 - 0) \\ 3{,}625 - 3{,}375 = k_1\,(4 - 3{,}375) + k_2\,(4 - 3{,}375) + k_3\,(4 - 3{,}375) \end{cases}.$$

The solution of this system gives the value of the superimposition coefficients: $k_1 = 9/20$, $k_2 = 0$, $k_3 = -1/20$.

To determine the coordinates of point $A_0(1;\,1;\,4{,}0625)$, the superimposition coefficients has the values: $k_1 = 11/20$, $k_2 = 1/2$, $k_3 = 1/20$.

Discrete values of the second curved surface are formed according to the initial data $z_{A1} = 4$, $z_{A2} = 4$, $z_{A3} = 4$, $z_{A4} = 4$, $P_{ij} = -6$.

The system of equations (7) is

$$\begin{cases} 4z_{10} - 4z_{00} - 6 = 0 \\ 4 + z_{00} - 4z_{10} + 2z_{11} - 6 = 0 \\ 8 + 2z_{10} - 4\,z_{11} - 6 = 0 \end{cases}.$$

Solution of this system gives the result: $z_{00} = -9/4$, $z_{10} = -3/4$, $z_{11} = 5/8$.

The discrete values of the second surface application formed by the static-geometric method and by the specified value of the external shape-forming load P_{ij} = –6 distributed uniformly among the nodes of the discrete frame are presented in Table 3.

Table 3 – Applicative points values of the second surface with the load value of P_{ij} = –6

	i = –2	i = –1	i = 0	i = 1	i = 2
j = 2	8	5	4	5	8
j = 1	5	5/8	-3/4	5/8	5
j = 0	4	-3/4	-9/4	-3/4	4
j = –1	5	5/8	-3/4	5/8	5
j = –2	8	5	4	5	8

It can be computed the superimposition coefficients the values of the same three specified points of the reference contour $A_1(2; 0; 4)$, $A_2(0; 2; 4)$, $A_3(-2; 0; 4)$, and also the central node $A_4(0; 0; -2{,}25)$ to determine the coordinates of point $A_0(1; 0; -0{,}75)$.

According to the above initial data, the solution of the equations system (8) gives the values of superimposition coefficients: $k_1 = 37/100$, $k_2 = 0$, $k_3 = -13/100$.

To determine the coordinates of point $A_0(1; 1; 0{,}625)$, the superimposition coefficients is the values: $k_1 = 23/100$, $k_2 = 0{,}5$, $k_3 = -27/100$.

The value of the external shaping load and the nodal points application of the sought curved surfaces discrete frames as superimpositions of two discrete curved surfaces frames pre-formed by the static-geometric method (Fig. 3) is determined by the formulas:

$$P_{ij} = k_1 P_{ij}^1 + k_2 P_{ij}^2 \ ,$$

$$z_{ij} = k_1 z_{ij}^1 + k_2 z_{ij}^2 \ ,$$

where P_{ij}^1 – magnitude of the uniformly distributed external shape-forming load applied to the nodes of the modeled first surface, and P_{ij}^2 – that of the second one;

z_{ij}^1 – applicate of the ij-node of the first surface, and z_{ij}^2 – that of the second surface.

Considering the uniform step of the load size changing from 1 to 6: $6 - 1 = 5$; $1/5 = 0{,}2$ obtains $k_1 = k_2 = 0{,}2; 0{,}4; 0{,}6; 0{,}8$.

For example, the load value $P_{ij}^{A_4}$ and application of nodal points $z_{ij}^{A_4}$ of the discrete surface frames is determined by the formulas:

$$P_{ij}^{A_4=2,25} = 0{,}2 \cdot (-6) + 0{,}8 \cdot (-1) = -1{,}2 + (-0{,}8) = -2 \ ,$$

$$P_{ij}^{A_4=1,25} = 0{,}6 \cdot (-6) + 0{,}4 \cdot (-1) = -3{,}6 + (-0{,}4) = -4 \ ,$$

$$P_{ij}^{A_4=0} = 0{,}6 \cdot (-6) + 0{,}4 \cdot (-1) = -3{,}6 + (-0{,}4) = -4 \ ,$$

$$P_{ij}^{A_4=-1,25} = 0{,}8 \cdot (-6) + 0{,}2 \cdot (-1) = -4{,}8 + (-0{,}2) = -5 \ ,$$

$$z_{Pij=-2}^{A_4} = 0{,}2 \cdot (-2{,}25) + 0{,}8 \cdot 3{,}375 = -0{,}45 + 2{,}7 = 2{,}25 \ ,$$

$$z_{Pij=-3}^{A_4} = 0{,}4 \cdot (-2{,}25) + 0{,}6 \cdot 3{,}375 = -0{,}90 + 2{,}025 = 1{,}125 \ ,$$

$$z_{Pij=-4}^{A_4} = 0{,}6 \cdot (-2{,}25) + 0{,}4 \cdot 3{,}375 = -1{,}35 + 1{,}35 = 0 \ ,$$

$$z_{Pij=-5}^{A_4} = 0{,}8 \cdot (-2{,}25) + 0{,}2 \cdot 3{,}375 = -1{,}80 + 0{,}675 = -1{,}125 \ .$$

The recurrent dependence and nodal points application value of the surface curves discrete models is presented in Table 4.

Table 4 – The value of the external shaping load and the nodal points ordinates of the sought surface curves discrete models

	$k_1 = 0{,}8$ $k_2 = 0{,}2$	$k_1 = 0{,}6$ $k_2 = 0{,}4$	$k_1 = 0{,}4$ $k_2 = 0{,}6$	$k_1 = 0{,}2$ $k_2 = 0{,}8$	
$P_{ij}^{A_{00}=3,375} = -1$	$P_{ij}^{A_{00}=2,25} = -2$	$P_{ij}^{A_{00}=1,125} = -3$	$P_{ij}^{A_{00}=0} = -4$	$P_{ij}^{A_{00}=-1,125} = -5$	$P_{ij}^{A_{00}=-2,25} = -6$
$P_{Pij=-1}^{A_{00}} = 3{,}375$	$P_{Pij=-2}^{A_{00}} = 2{,}25$	$P_{Pij=-3}^{A_{00}} = 1{,}125$	$P_{Pij=-4}^{A_{00}} = 0$	$P_{Pij=-5}^{A_{00}} = -1{,}125$	$P_{Pij=-6}^{A_{00}} = -2{,}25$
$P_{Pij=-1}^{A_{10}} = 3{,}625$	$P_{Pij=-2}^{A_{10}} = 2{,}75$	$P_{Pij=-3}^{A_{10}} = 1{,}875$	$P_{Pij=-4}^{A_{10}} = 1$	$P_{Pij=-5}^{A_{10}} = 0{,}125$	$P_{Pij=-6}^{A_{10}} = -0{,}75$
$P_{Pij=-1}^{A_{11}} = 4{,}0625$	$P_{Pij=-2}^{A_{11}} = 3{,}375$	$P_{Pij=-3}^{A_{11}} = 2{,}6875$	$P_{Pij=-4}^{A_{11}} = 2$	$P_{Pij=-5}^{A_{11}} = 1{,}3425$	$P_{Pij=-6}^{A_{11}} = 0{,}625$

The results of the computed superimposition coefficients of four specified points A_{20}, A_{02}, A_{-20}, A_{00} to determine the coordinates of unknown points A_{10} and A_{11} (considering the symmetry conditions) of the formed curved surfaces discrete models for various values of P_{ij} are presented in Table 5.

Table 5 – Values of superimposition coefficients

	A_{20}	A_{02}	A_{-20}	A_{00}	A_{10}	A_{11}
			$P_{ij}=-1$			
z_{ij}	4	4	4	27/8	29/8	65/16
k_1					9/20	11/20
k_2					0	½
k_3					-1/20	1/20
$k_4=1-k_1-k_2-k_3$					12/20	-2/20
			$P_{ij}=-2$			
z_{ij}	4	4	4	9/4	11/4	27/8
k_1					11/28	9/28
k_2					0	½
k_3					-3/28	-5/28
$k_4=1-k_1-k_2-k_3$					20/28	10/28
			$P_{ij}=-3$			
z_{ij}	4	4	4	9/8	15/8	43/16
k_1					35/92	25/92
k_2					0	½
k_3					-11/92	-21/92
$k_4=1-k_1-k_2-k_3$					68/92	42/92
			$P_{ij}=-4$			
z_{ij}	4	4	4	0	1	2
k_1					3/8	2/8
k_2					0	½
k_3					-1/8	-2/8
$k_4=1-k_1-k_2-k_3$					6/8	4/8
			$P_{ij}=-5$			
z_{ij}	4	4	4	-9/8	1/8	21/16
k_1					61/164	39/164
k_2					0	½
k_3					-21/164	-43/164
$k_4=1-k_1-k_2-k_3$					124/164	86/164
			$P_{ij}=-6$			
z_{ij}	4	4	4	-9/4	-3/4	5/8
k_1					37/100	23/100
k_2					0	½
k_3					-13/100	-27/100
$k_4=1-k_1-k_2-k_3$					76/100	54/100

According to the results presented in Table 5, a certain regularity can be observed relating to the superimposition coefficient values of the specified three boundary and central nodes to determine the sought nodes coordinates that lies in the following.

The difference in the superimposition coefficient values of two specified nodes in the boundary contour k_1 and k_3 to determine the sought nodal points coordinates is a constant value and equals to 1/2 ($k_1 - k_3 = 1/2$) .

It can be proved this conclusion by writing the system of equations 7 for the above-determined curved surfaces discrete models in general terms:

$$\begin{cases} 4z_{10} - 4z_{00} = P_{ij} \\ 4 + z_{00} - 4z_{10} + 2z_{11} = P_{ij} \\ 8 + 2z_{10} - 4z_{11} = P_{ij} \end{cases} .$$

Solution of this system gives the result:

$$z_{00} = \frac{36 - 9P_{ij}}{8}, \quad z_{10} = \frac{36 - 7P_{ij}}{8}, \quad z_{11} = \frac{76 - 11P_{ij}}{16} .$$

Hence, the nodal point coordinates of the specified and modeled discrete curved surfaces is:

$$A_{00}(0;0;\frac{36}{8} - \frac{9}{8}P_{ij}), \quad A_{10}(1;0;\frac{36}{8} - \frac{7}{8}P_{ij}), \quad A_{11}(1;1;\frac{76}{16} - \frac{11}{16}P_{ij}) .$$

$$A_1(2;0;4), \quad A_2(0;2;4), \quad A_3(-2;0;4) .$$

Substituting coordinates of point A_{10} into the equation system (8), it is obtained:

$$\begin{cases} 1 - 0 = k_1(2 - 0) + k_2(0 - 0) + k_3(-2 - 0) \\ 0 - 0 = k_1(0 - 0) + k_2(2 - 0) + k_3(0 - 0) \\ \left(\frac{36}{8} - \frac{7}{8}P_{ij}\right) - \left(\frac{36}{8} - \frac{9}{8}P_{ij}\right) = k_1\left(4 - \left(\frac{36}{8} - \frac{9}{8}P_{ij}\right)\right) + k_2\left(4 - \left(\frac{36}{8} - \frac{9}{8}P_{ij}\right)\right) + \\ + k_3\left(4 - \left(\frac{36}{8} - \frac{9}{8}P_{ij}\right)\right) \end{cases}$$

Solution of this system gives the result:

$$k_1 = \frac{13P_{ij} - 4}{36P_{ij} - 16} \; ; \; k_2 = 0 \; ; \; k_3 = \frac{4 - 5P_{ij}}{36P_{ij} - 16}$$

Including: $k_1 - k_3 = 1/2$.

Substituting coordinates of point A_{11} into the equation system (8), it is obtained:

$$\begin{cases} 1 - 0 = k_1(2 - 0) + k_2(0 - 0) + k_3(-2 - 0) \\ 0 - 0 = k_1(0 - 0) + k_2(2 - 0) + k_3(0 - 0) \\ \left(\frac{76}{16} - \frac{11}{16}P_{ij}\right) - \left(\frac{36}{8} - \frac{9}{8}P_{ij}\right) = k_1\left(4 - \left(\frac{36}{8} - \frac{9}{8}P_{ij}\right)\right) + k_2\left(4 - \left(\frac{36}{8} - \frac{9}{8}P_{ij}\right)\right) + \\ + k_3\left(4 - \left(\frac{36}{8} - \frac{9}{8}P_{ij}\right)\right) \end{cases}$$

Solution of this system gives the result:

$$k_1 = \frac{7P_{ij} + 4}{36P_{ij} - 16} \; ; \; k_2 = \frac{1}{2} \; ; \; k_3 = \frac{12 - 11P_{ij}}{36P_{ij} - 16} .$$

Including: $k_1 - k_3 = 1/2$.

Conclusions. The method of constructing a grid based on superimpositions of pre-calculated two or more grids with the similar topology permits to easily determine the coordinates of an arbitrary node in a new grid according to the coordinates of the respective nodes in the known grids.

Four specified nodal point coordinates coefficient superimpositions values of discretely presented two-dimensional geometric images for determining the sought nodal point coordinates can be determined bas on known initial geometric images superimposition coefficients.

According to a single discrete model of the curved surface formed by the static-geometric method, it is possible to form, by means of the superimpositions method, any number of balanced discrete surface models with an arbitrary number of nodal points under the same boundary conditions and different values of the external shaping load without composing and solving cumbersome systems of linear equations, that permits to discretely modeling of various shapes surfaces and to solve the tasks of discrete spatial interpolation. Coordinates of any node in the modeled surfaces discrete frame is determined by the formulas (8).

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SUPPORTED CONSTRUCTION FOR QUARRY EXCAVATOR TRAVERS

It has been established that repair of quarry excavators should be carried out directly on site. In this case, the use of jacks for weighing the rotary platform of excavators Bucyrus RH340 and Terex RH200 models for repair of running equipment is impossible because of their design features. This type of repair is suggested to be performed as an excavator based on the design developed by the authors. The installation of the excavator on the support structure is carried out by its working equipment. The supporting structure is suggested to be made in the form of two identical fragments for its unification regarding the base of excavators and the convenience of transporting to the site of work. The suugested method of fixing the excavator to the support structure prevents it from being displaced from the impact of seismic loads during blasting operations. The investigation of the stress-strain state of the supporting structure was carried out using the complex of finite-element analysis.

Keywords: *hydraulic excavator, strength analysis, method of ultimate elements, strain state.*

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ОПОРНА КОНСТРУКЦІЯ ПІД ТРАВЕРСУ КАР'ЄРНОГО ЕКСКАВАТОРА

Установлено, що при ремонті кар'єрних екскаваторів використання домкратів для виважування поворотної платформи для моделей Bucyrus RH340 і Terex RH200 неможливе через їх конструктивні особливості. Тому ремонт запропоновано виконувати на розробленій авторами опорній конструкції. Установлення екскаватора на опорну конструкцію виконано його робочим обладнанням. Запропоновано спосіб фіксації екскаватора на опорній конструкції, що запобігає його зміщенню від впливу сейсмічних навантажень при вибухових роботах. Дослідження напружено-деформованого стану опорної конструкції проведено з використанням комплексу скінченноелементного аналізу.

Ключові слова: *гідравлічний екскаватор, аналіз міцності, метод скінченних елементів, деформований стан.*

Introduction. Crawler moving equipment for quarry excavators is subject to heavy loads during the process of soil development and moving the machine. Also, metal surfaces of the propeller parts of are exposed to moisture, abrasive, and sometimes aggressive media. As a result of these factors, crawler tapes, sprockets, steering wheels, rollers and other components of the running gear are triggered and deformed, which leads to the repair work need.

Unlike pneumatic equipment (such as dump trucks, loaders, etc.) delivery of a crawler excavator in general or its separate element to a service center, even on a specially equipped platform for repair, is a complex task. Therefore, the maintenance and repair of the excavator directly on the site of work is actual. In turn, these types of technological influences are often associated with the need for unloading the running gear, that is, with a partial or complete weighting of the machine [1 – 3].

Analysis of recent sources of research and publications. The practice of performing such a type of work is tested, which is based on the disconnection of the rotary part of the excavator by lifting it over irreversible with the help of powerful hydraulic jacks [4, 5]. However, such a lift can only be made if the overall dimensions of the swing platform in terms of at least one of its positions exceed the overall dimensions of the crawler. This makes it possible to bring jacks or beams to the carving elements of the swing platform. In excavators of a number of models, for example, the Bucyrus RH340 and Terex RH200 [6, 7] (see Figure 1), the rotary platform does not go beyond the running gear and it is impossible to put it under the jack. Therefore, in this case, other ways to unload the running equipment, in particular, considering the excavator as a whole, should be applied.

Figure 1 – Service object – quarry excavator Bucyrus

The complexity of this work type is due to the large mass of excavators (so the mass of Bucyrus RH340 is 542 tons) and the possibility of the impact of seismic loads during the performance of explosive work in a quarry on a machine that is in a weighted condition.

Identification of general problem parts unsolved before. The height of the excavator weighing is determined by the possibility of access to the main components of the running equipment and is relatively small. So for excavators Bucyrus and Terex models (see Figure 1), the height above the surface of the platform, which is 300 mm, is quite sufficient for maintenance.

In the process of digging, when immersing a bucket in a rock and turning it for scraping, the working equipment of these excavators develops considerable effort both in the horizontal and in the vertical directions. These efforts are quite sufficient for partial weighting of the machine over the support surface [8]. Characteristics of the working equipment strength meet these requirements too. However, the full unloading of all running gear cannot be achieved. Maintenance of the machine in a weighed condition during a performance of repairs is extremely dangerous. Consequently, it is necessary to create a supporting structure of the required carrying capacity, which is supplied to the excavator not simultaneously, but consistently, as a partial weighing of the machine with the help of its own working equipment.

Consudering this fact, it was decided to lift the excavator to the required height by installing the supports for the part of the excavator weighed out using its own working equipment. Thus, the **goal** is to reduce the design of the supporting structure of the required payload under the traverse of the excavator.

Basic material and results. In the analysis of the excavators traverse structure of the indicated models and consuidering the wishes of the customer, it was found that the optimum place for establishing the support structure under the traverse is its supporting platforms (see Figure 2).

Figure 2 – The location of the supporting platforms on the traverse of the excavator

It was decided to locate the support structure under the excavator between the crawler belt (see Figure 3). This is due to the need for access to the main components of the chassis, the location of the traversing support platforms, the dimensions of the chassis, namely the distance in the transverse direction between the crawler belts and the base, requirements for pressure on the support platform.

The permissible pressure on the soil was taken in the same way as the pressure created by the excavator itself. In this case, the required area of the supporting structure is approaching the area of the intergenerational space. The same basic structure is proposed to be made of two identical fragments in order to unify it with respect to the base of excavators and the convenience of transportation to the place of work (see Figure 4).

It was decided to carry out the positioning of the machine before weighing it by coming onto pre-exposed fragments of the supporting structure. This is due to the large mass of individual fragments of the supporting structure, and, consequently, the complexity of their placing under the excavator. However, the constructive feature of the traverse of the excavator is that in the middle part it has the highest height above the level of the reference surface. At crawler tapes, in areas where the main platforms are located, the altitude is significantly reduced. Therefore, constructively, each piece of the supporting structure is executed with two mobile-bearing stands. The curbs on the excavator are shifted to the middle, which ensures unhindered movement of the excavator over the support structure. In the future, when weighing the excavator with its own working equipment, the main pillars are split into the sides, under the support of the traverse of the excavator. Such a constructive solution provides the necessary height of fixing a weighted excavator for repair of running gear.

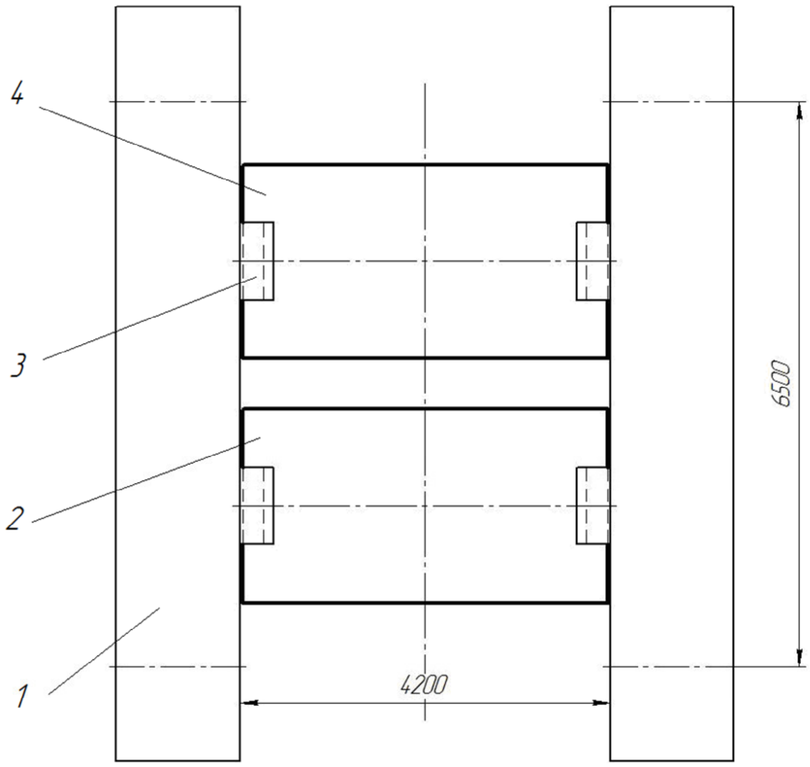

Figure 3 – The location of the supporting structure under the excavator (indicated by a thickened line): 1 – crawler tape; 2, 4 – fragments of the supporting structure; 3 – the main platforms of the traverse of the excavator

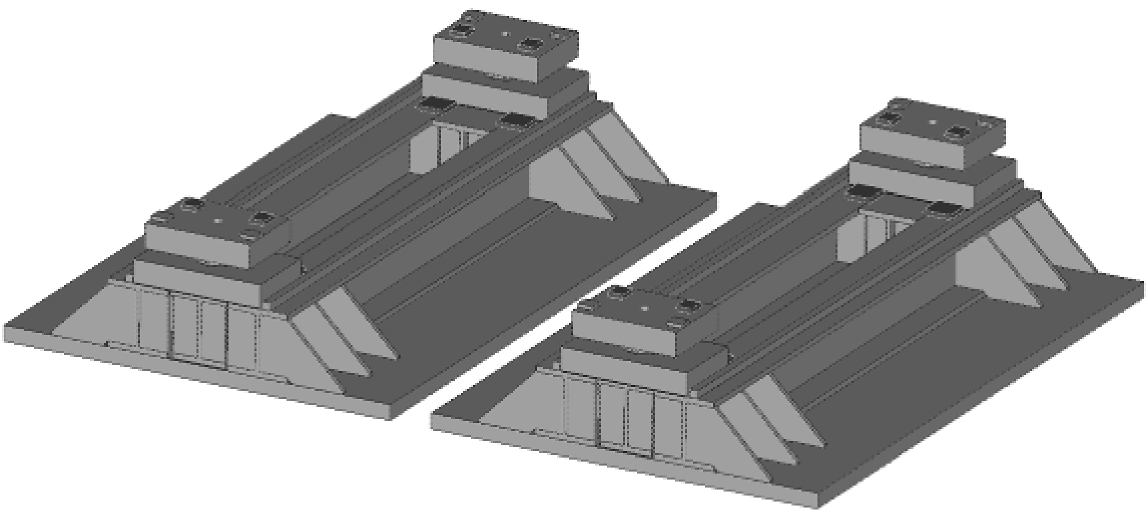

Figure 4 – The structure of the supporting structure under the traverse of the excavator

The significant weight of mobile stands does not allow them to be moved at the expense of the worker's efforts along the metalwork. Therefore, it was decided to use a screw mechanism, with the possibility of its reinstallation as the pedestal progressed to the working position (see Fig. 5). A characteristic feature of the mechanism design is in order to compensate for distortions when pushing the pedestal, the nut of the screw mechanism is made such that it has two degrees of freedom relative to the fixing position. Due to this fact, the distortions are compensated in the horizontal and vertical planes. Moving the pedestal to the working position is carried out in two resetting of the mechanism.

Installing the excavator on the support structure takes place in several techniques. The first step is to arrange the towers in the plan at the site clearly, which is selected and pre-prepared in accordance with the requirements for repair work. Further, the excavator moves in its turn on the support structure to ensure the clear location of the supporting platforms of the traverses over the reference stands.

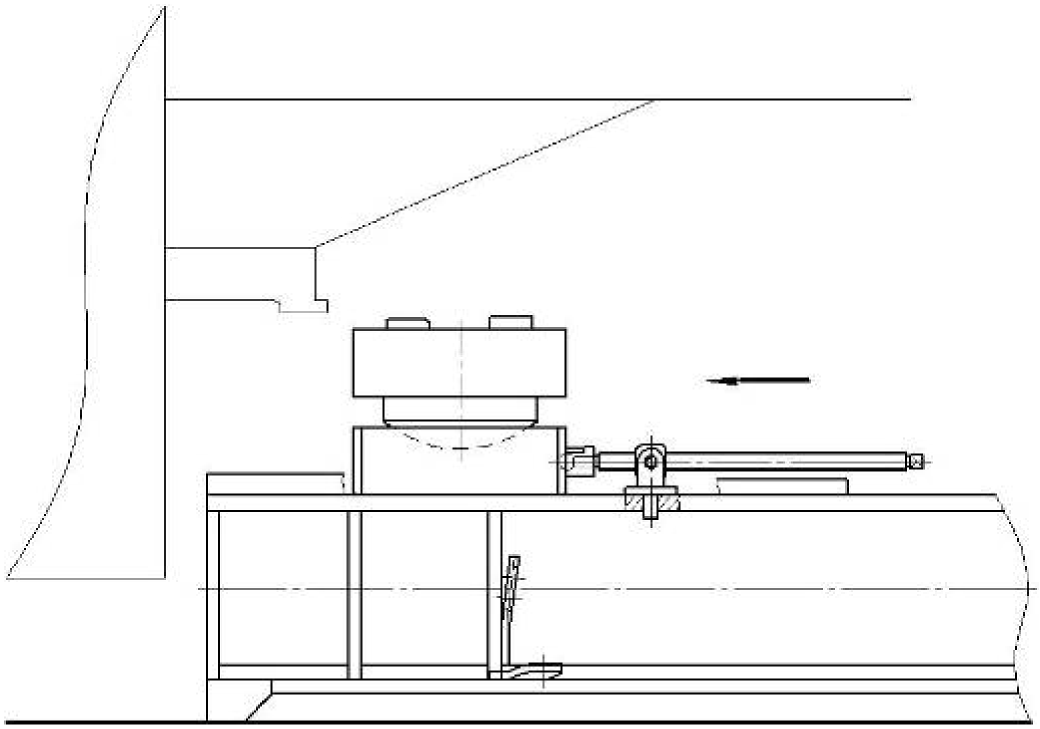

Figure 5 – Fitting the pedestal under the support of the traverse with the help of a screw mechanism

At the expense of its own working equipment, one side of the excavator is weighed to the height necessary for the installation of support posts under the supporting platforms of the traverse. When weighing the position of the excavator, with the help of screw mechanism, the digging of the stands in the working position is carried out, after which the excavator falls to the curbstones.

After turning the rotary part by 180° the steps are repeated - the opposite part of the excavator is weighed and fixed on the stands of the second piece of the supporting structure. In order to avoid self-propelled pivots with respect to the support plate during blasting, their fixation is foreseen. The removal of the excavator from the support structure after the repair work is carried out in reverse order.

Another characteristic feature of the support structure is the following: each of the stands has two assembly units that contact each other through a spherical surface (see Fig. 5). Such a constructive decision was made considering the support areas of the weights of the weighted excavator are at a certain angle to the horizon (see Fig. 6), and, with further lowering of the excavator, the support surfaces of the platform traverses and pedestals have a small touch area. It causes significant specific loads in the point of contact, which can lead to deformation of these elements.

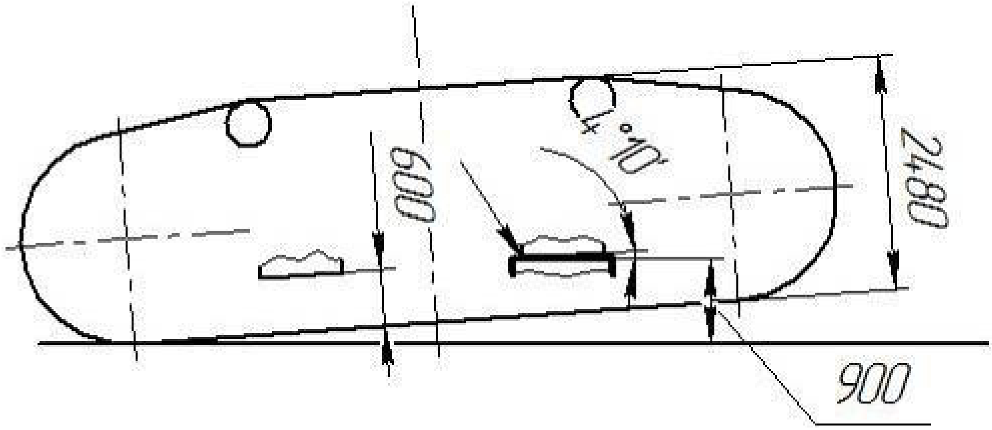

Figure 6 – Scheme for calculating the deviation angle of the excavator traverse site from the horizontal at the moment of contact with the bearing surface of the curb

Damage to the support can lead to loss of durability and drop of the excavator causing significant material damage and is hazardous to the health and life of the maintenance staff. This explains the high requirements for calculations.

The use of traditional analytical techniques for theoretical mechanics and material resistance to perform a precise analysis of the stress-strain state (NDS) of the supporting structure is not possible due to the practical impossibility of determining the exact values of stresses in the zones of their possible concentrations. The research of the NDs of the supporting structure, most expediently to carry out using finite-element analysis, which perspective in solving various tasks of the mechanics of a deformable solid, has recently been increasingly confirmed in the works of both foreign and domestic authors [9 – 12].

The idea of the Finite Element Method (FEM) consists in representing the geometry of the supporting structure as a set of individual elements of rather simple geometric forms, which are described by the well-known theoretical and experimental dependencies of the deformable solid mechanics. The combination of elements is carried out by satisfying the conditions of continuity of movements and equilibrium conditions. In this case, there are used three groups of equations: static, describing the system equilibrium state; geometric, linking deformation and displacement; and physical, which bind forces and deformations among themselves.

Mathematical modeling was performed using well-tested modeling complex and a finite-element analysis of MSC / NASTRAN designed for implementation in Windows environment at the PC. The package by which the model data is created and analyzed on the basis of the finite-element procedure determines the movement of each node of the finite element by three coordinate axes, the normal and tangent stresses, as well as the equivalent stresses that are calculated according to the well-known equation of the energy of the Von Mises modification energy the formula

$$\sigma_{equ} = \sqrt{0{,}5[(\sigma_x - \sigma_y)^2 + (\sigma_y - \sigma_z)^2 + (\sigma_z - \sigma_x)^2 + 6(\tau_{xy}^2 + \tau_{yz}^2 + \tau_{zx}^2)]} \ ,$$

where σ_x, σ_y, σ_z – normal stress acting on the axes respectively *x, y* and *z;*

τ_{xy}, τ_{yz}, τ_{zx} – tangential stresses arranged in planes accordingly *xy, yz, zx*.

The MSC / NASTRAN for Windows Simulation and Finite Analysis package has advanced geometric modeling and data exchange tools with well-known CAD programs: AutoCAD, SolidWorks, Solid Edge, Pro Engineer, etc.

For any machine-building design, the typical calculation involves the following stages of constructing finite-element models: the development of a geometric model; description of materials properties; creation of a finite-element grid; formation of loads and boundary conditions; execution of various types of settlements; analysis of the calculations.

At the first stage in the modeling package and finite-element analysis of MSC / NASTRAN, designed for implementation on a personal computer results in the Windows environment, a full-fledged elastic three-dimensional model of support structure was created. The developed three-dimensional model is divided into a fairly small finite-element grid of tetrahedral elements in the sizes from 10 to 100 mm (for the entire construction used 97184 volumetric elements with 193954 nodal points).

The physical and mechanical characteristics used in the subsequent calculations of the structural components of the finite-element model of supporting structure are given in Tab. 1.

Table 1 – Physical and mechanical characteristics of supporting structure finite element model structural components

Material	Modulus of elasticity, E, МПа	Poisson's coefficient
Steel Ст3сп3	$2{,}1 \cdot 10^5$	0,30
Base under the support	50	0,30

Accordingly [13], the yield strength for structural elements made of steel Ст3сп3 thickness from 20 to 40 mm $R_{yn} = 235$ MPa and elements with a thickness of more than 100 mm $R_{yn} = 205$ MPa, then according to [14] the design resistance: $R_y = R_{yn} / \gamma_m = 235 / 1{,}025 = 230$ MPa – for steel elements in the thickness from 20 to 40 mm; $R_y = R_{yn} / \gamma_m = 205 / 1{,}025 = 200$ MPa – for steel elements in the thickness exceeding 100 mm.

Considering the presence of contact surfaces both in the hinges of mobile stands, and between mobile slides and longitudinal beams, the coefficients of friction for dry contact steel surfaces are taken to be equal to 0.15, and for steel surfaces with a lubricant – 0.05, respectively.

As the main evaluation criterion, the maximum values of equivalent stresses in the reference structure, which arise under the action of the calculated values of loads, are accepted.

The basis for the proposed supporting structure is a pre-prepared site on the rubble gravel with a slope that does not exceed 3°. As the calculated load values, four different load variations that occurred at different stages of the weighting of the excavator were considered.

The first boot option occurs when one-sided lift and installation of the excavator on the support structure. Given the lack of exact coordinates of the center of gravity of the excavator, as the most unfavorable load, the calculated value of the vertical load from the gravity of the excavator, divided into two hinged supports, is assumed. Considering the possibility of an asymmetrical position of the excavator gravity center between the two hinged supports during rotation of the rotary platform is carried out by distributing 60% of the calculated vertical load per resistance and 40% of the load to the other.

The second variant of the support structure loading occurs when the one side of the excavator is tied to the support structure and weighing the other side with the boom arrow. In this case, in addition to the vertical component of the excavator own weight load, the horizontal component of the load, directed along the line of the excavator, may appear. For the most adverse combination of these two components, a combination of a vertical load equal to 90% of the calculated net weight and the maximum horizontal load, corresponding to the value of the friction force between the traverse of the excavator and the support structure (the beginning of the excavator traverse slipping on the support structure, the coefficient of friction between the steel surfaces $k_f = 0.15$). The possibility of an asymmetrical position of the excavator gravity center between the two hinged supports is considered by allocating 60% of the calculated load per resistance and 40% of the load to the other.

The third version of the supporting structure loading considers the possibility of seismic loads occurence during blasting during the excavator stay on the support structure. According to the results of the instrumental measurements of the seismic fluctuations and shock airwaves intensity during the mass explosion in the Yaristovsky GOK, LLC, the maximum value of horizontal seismic influences is 0.109g, and the vertical seismic influences 0.159g (g – free acceleration) on distance 450-550 m from the epicenter of the explosion. The direction of a horizontal load from seismic influences is taken along the excavator axis, considering the previously most unfavorable location of the excavator gravity centre.

The fourth variant of the loading of the supporting structure differs from the third variant only in the direction of the horizontal load from seismic influences, which is taken perpendicular to the excavator axis.

Influences of wind and snow loads in calculations for the strength of the supporting structure were not considered due to the insignificance of their estimated values in comparison with the estimated values of seismic loads.

Calculations of the strength of the proposed supporting structure are made for the four above variants of its loading. In all four cases, the loading of the maximum equivalent stress occurring in the lower part of the hinge support is 191 MPa, 156 MPa, 181 MPa and 196 MPa, respectively, in the field of equivalent stresses distribution in the lower part of the hinge support for the most unfavorable fourth loading variant, are shown in Figure 7. As can be seen from this figure, the maximum values of equivalent stress occur on the side of the hinge on the inner side of the hinge support lower part.

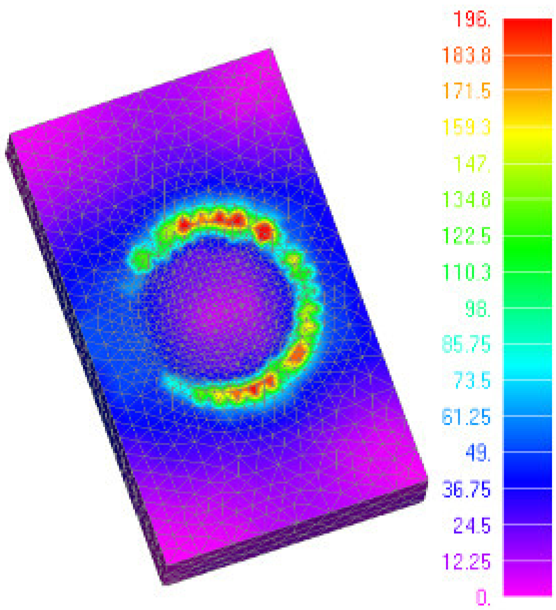

Figure 7 – Fields of distribution of equivalent stresses in the lower part of the hinge support in the case of the fourth boot option

The maximum values of the equivalent stresses occurring in the support plate and longitudinal beam for all four cases of loading are 186, 188, 202 and 157 MPa, respectively.

The fields of the equivalent stresses distribution in the base plate and longitudinal beam for the most unfavorable third loading variant are shown in Figure 8. As shown in this figure, the maximum values of equivalent stresses are observed in stress concentration zones both under the reference stands and in the middle parts of the longitudinal beam.

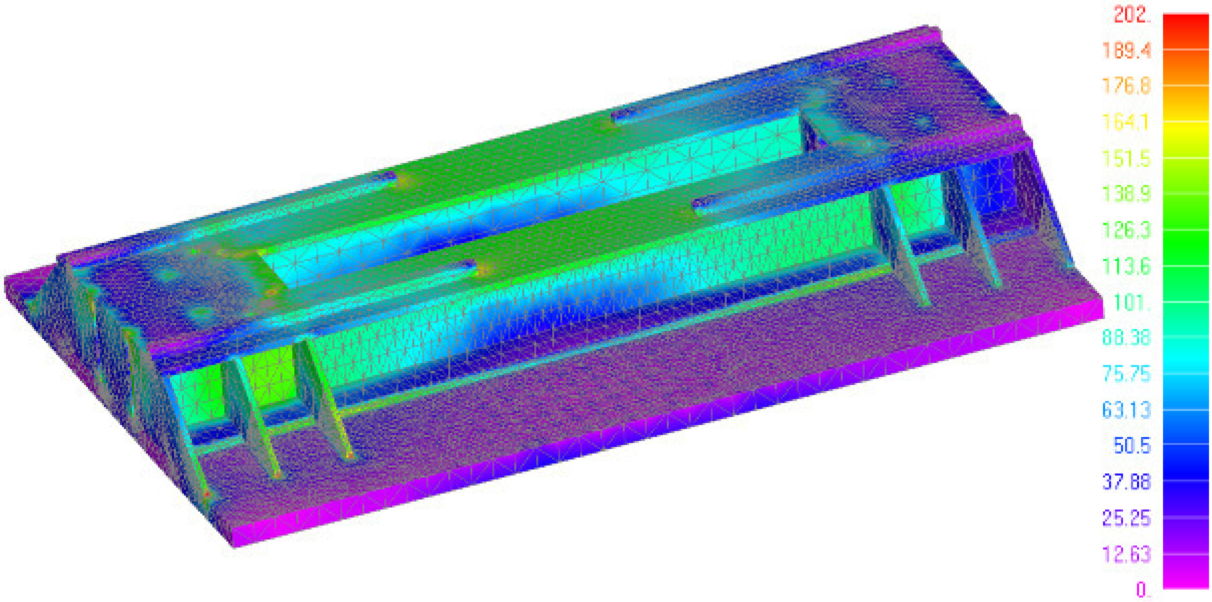

Figure 8 – Distribution fields for equivalent stresses in the case of the third boot option

Considering that the maximum values of equivalent stresses occurring in the reference plate and the longitudinal beam for all cases of loading do not exceed the rated resistance Ry = 230 MPa, and the maximum values of equivalent stresses in the reference pedestal do not exceed Ry = 200 MPa the strength of the supporting structure is ensured.

As a result of checking the possibility of stability loss of the supporting structure, the magnitudes of the critical load factors exceeding 3 are obtained, which means that the stability of the supporting structure is ensured.

Conclusions. Thus, the repair and maintenance of the running gear equipment of the Bucyrus RH340 and Terex RH200 career excavators are proposed to be carried out directly on the open mine site, by installing the machine on the supporting structure using its own working equipment.

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THE AUTOMOBILE TIRES VULCANIZATION METHOD REPAIR PROCESS EXPERIMENTAL INVESTIGATION

In the article local repair automobile tires expediency by vulcanization means in mechanical damage carcass case is shown. It has been established that analytical and experimental data on determining the required temperature of the heating element have a significant discrepancy. It was found that on the vulcanization degree among all factors temperature and pressure on the welding surfaces are the most influential. After processing the experimental data and using the static methods, mathematical dependence of the temperature on the welding surface from the heating element temperature and on the welding surfaces as a second-degree polynomial is obtained. In order to verify the research results reliability, the control welding of the automobile tires with cord lateral rupture was conducted, which gave positive results.

***Keywords:** thermal conductivity, vulcanization, automobile tires.*

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ЕКСПЕРИМЕНТАЛЬНІ ДОСЛІДЖЕННЯ ПРОЦЕСУ РЕМОНТУ АВТОМОБІЛЬНИХ ШИН СПОСОБОМ ВУЛКАНІЗАЦІЇ

Обґрунтовано доцільність проведення місцевого ремонту автомобільної шини способом вулканізації при механічних ушкодженнях каркаса. Установлено, що аналітичні та експериментальні дані з визначення необхідної температури нагрівального елемента мають значну розбіжність. З'ясовано, що на ступінь вулканізації серед усіх факторів найбільший вплив мають температура та тиск на поверхнях зварювання. За допомогою статистичних методів обробки експериментальних даних отримано математичну залежність температури на поверхні вулканізації від температури нагрівального елемента й тиску на поверхнях зварювання у вигляді полінома другого ступеня. З метою перевірки достовірності результатів досліджень виконано контрольне зварювання автомобільної шини з боковим розривом корду, яке дало позитивні результати.

***Ключові слова:** теплопровідність, вулканізація, автомобільна шина.*

Introduction. Nowadays tires repair is one of the most demanded services in the automobile service market. The statistics show that in the course of operation from 25 to 75% of tires prematurely fail due to mechanical damage to the carcass (punctures and cuts) that require local repair. In most cases, the tread does not exceed 30 – 50%, and timely and qualified repairs allow to continue using this tire. In this case local repairs are more effective than restorative. Its cost is 2 – 7% from a new product price, so even a slight increase in after-repair runs can significantly increase the economic effect from such tires use [1].

One of the most effective methods to automobile tires repairing with local injuries is vulcanization, which is the converting process general purpose caoutchouc (natural and synthetic) into rubber under pressure and at temperature of 140 – 160 ° C. Compliance with the necessary and stable temperature regime in the vulcanization zone has a decisive influence on the quality of the repaired product. The numerous studies results show, that the temperature variation in the tire vulcanization zone at 5 °C leads to a change in the vulcanization degree by more than 30% [2]. However, in a number by the industry manufactured vulcanizers, there is a significant difference in temperature across the entire heating element surface (more than 5 °C), and it is difficult or sometimes impossible to control the temperature in the vulcanization zone. It is also noticed that the temperature regime in the vulcanization zone is influenced by the pressure force between tire and the repair material and the heating element [3 – 5].

Analysis of the last research sources and publications. Analytical calculations the heating element required temperature, based on the thermal conductivity of multilayer walls theory [6], do not give objective results. There is considerable discrepancy between calculated and experimental data. Is is due to the fact that, in the analytical calculation method, it is assumed that the contact is perfect at the point of contact of the individual layers [7] and the heat transfer is carried out from the layer to the layer without considering the spaces between them. However, in a real object research, the touch of individual layers is not dense and it gives an error in the calculations. It is difficult to consider the lack of tight contact by analytical method is difficult [8]. In this connection, it was decided to conduct a physical experiment using methods of statistical data processing [9 – 11] to determine the required heating element temperature when vulcanizing the various sizes tires and with different pressures on the vulcanized surfaces. This method also gives an opportunity to obtain a mathematical investigated process model, which in the future will allow determining required temperature of heating element by the calculation method.

Objectives setting. The article purpose is to highlight the experimental studies results of the vulcanization car tires process with new improved design vulcanizer.

The mains and researches. Studies have shown that the vulcanization process with local tire repair takes place under constantly changing conditions. These changes are caused by the thermophysical vulcanizer characteristics, the uneven distribution the layer inside temperature, which is vulcanized and fluctuations in the environment temperature. Also, the clamping force is significantly influenced by the vulcanization process.

Known mathematical models and automatic control systems for the vulcanization process [8] do not consider the uncertainty factors. This leads to significant temperature deviations from the given mode, resulting in the required complex of physical and mechanical properties of the repaired product is violated. And there is also an irrational electricity use present here.

The car tyres vulcanization process was investigated on the advanced vulcanizer «Asogis EVU-3MP» [12]. In order to ensure uniform heating elements pressure and vulcanizing materials to the tyres, the cavity frame was reinforced (Fig. 1) and a bag with sand or aluminum balls was applied (Fig. 2). Experimental studies were carried out using a baking form for 215/60/R16 type tyres (Fig. 3), which was made independently. The surface temperature for baking is determined by «Laserliner» laser pyrometer (Fig. 4).

Figure 1 – The new reinforced vulcanizer frame

Figure 2 – Use the bag with sand

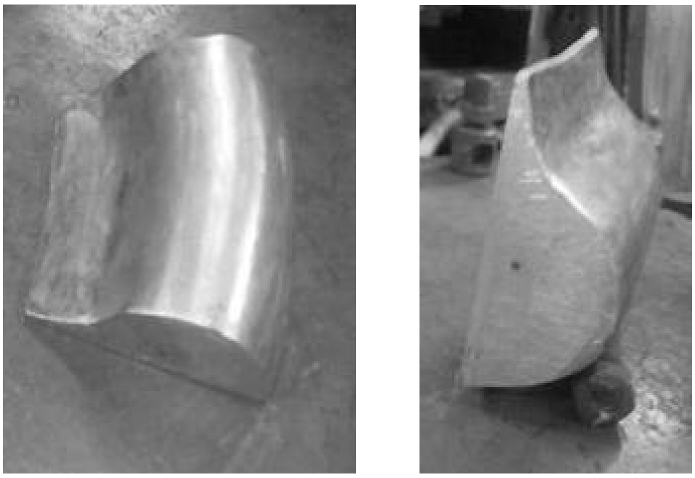

Figure 3 – A new baked form

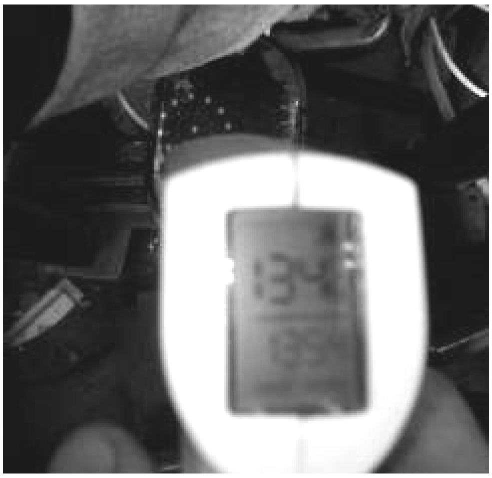

Figure 4 – The temperature definition with «Laserliner» laser pyrometer

In order to determine the optimum vulcanization and mathematical dependence regimes describing this process, the experiment planning was applied. Since the main factors influencing the vulcanization process are the heating element temperature and the pressure on the vulcanization surface, and the form of the received dependence is unknown, it is decided to accept the two-factor three-level plan of the experiment [8, 9].

During the experiment, the factors varied on three levels – the average (main), the lower and upper, the distances from the main to the same magnitude. The intervals values variables and the investigated range variables are given in Table 1.

Table 1 – The variables value intervals

Factors	Variable intervals
The heating element temperature, ° C	155 – 195
Pressure force on the welding surface, kgf/cm^2	0 – 16

The temperature welding surfaces at different heating element temperatures and the clamping force is determined by the plan are given in Table 2.

Table 2 – The three-tier experiments plan with the factors number k=2 … ($N=N_1+N_\alpha+n_0$)

Experiment number		The planning matrix, (x_i)		Variable squares, (x^2_i)		Interaction, ($x_i\, x_j$)	The surface welding temperature
		x_1	x_2	x_1^2	x_2^2	$x_1\, x_2$	y
N_1	1	+	+	+	+	+	167,3
	2	+	–	+	+	–	132,3
	3	–	+	+	+	–	158,4
	4	–	–	+	+	+	110,5
N_α	5	+	0	+	0	0	145,8
	6	–	0	+	0	0	125,5
	7	0	+	0	+	0	149,3
	8	0	–	0	+	0	122,2
n_0	9	0	0	0	0	0	141,5
	10	0	0	0	0	0	142,5
	11	0	0	0	0	0	142,1

In an investigation on vulcanization, experiments are divided into groups so that experiments at the zero point are evenly distributed among others. In particular, it is taken the following procedure for the implementation of the plan: Experiments 1, 2, 9, 3, 7, 4, 5, 10, 6, 7, 8, 11.

The experimental results are processed using from the mathematical statistics [8, 9]. As an implementation result of this experimental plan, it is obtained an algebraic equation in the form

$$\bar{y}_i = b_0 + b_1 x_1 + b_2 x_2 + b_{11} x_1^2 + b_{22} x_2^2 + b_{12} x_1 x_2 \ , \qquad (1)$$

where $\bar{y}_i$ – the investigated surface welding temperature ;

x_1, x_2 – outgoing factors;

b_0 , b_1 , b_2 , b_{11} , b_{22} , b_{12} – equation coefficients.

Having determined by the method [6, 7] the coefficients and substituting them in to (1) it is obtained the regression equation in code form

$$\bar{y}_i = 139{,}4 + 8{,}5x_1 + 18{,}34x_2 + 0{,}26x_1^2 + 0{,}36x_2^2 - 3{,}23x_1x_2 \,, \qquad (2)$$

Further equation (2) refine by checking the difference coefficients b_i zero by Student's test. Check the equations for the Fisher criterion confirms its suitability to describe the process under study.

For the convenience of further calculations, the equation (2) is reduced to the natural form (3)

$$t_л = 0,0041 \cdot P^2 + 0,0009 \cdot t_{н.е.}^2 + 4,52 \cdot P + 0,76 \cdot t_{н.е.} - 0,02 \cdot P \cdot t_{н.е.} - 30,02. \quad (3)$$

Using equation (3) building graphical dependencies from surface temperature welding depend on the heating element temperature and values pressure (fig. 5).

Based on the graphic depending determine the necessary heating element temperature and values pressure which provide temperature at bottom 150 ° C, which is the best value.

According to experimental studies, control of car tyre vulcanization with the side gap rupture of the rubber-coated layer was carried out (Fig. 6). The quality of the repaired tyre is tested pumped to pressure that exceeds the operating in twice. The repair defects is not found.

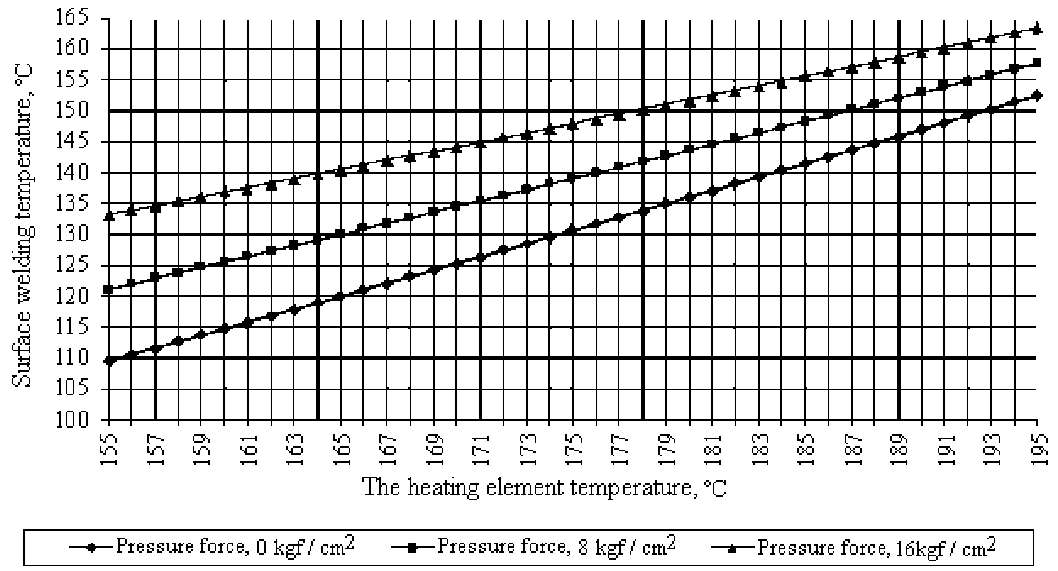

Figure 5 – The graphical dependencies from surface temperature welding depending on the heating element temperature and values pressure

Figure 6 – The control automobile tire vulcanization

Conclusions. In experimental investigation result of the automobile tires vulcanization process with the vulcanizer «Asogis EVU-3MP» help, the mathematical relationship between the temperature welding surfaces at heating element temperatures and the clamping force in a regression equation was received which is suitable for vulcanization modes optimization.

Practical recommendations for improving the automobile tires vulcanization process are developed, depending on the pressing force and heating element temperature.

During the work, the frame «Asogis EVU-3MP» structure was developed, and additional equipment for local tire repair was developed too.

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PROBABILISTIC NUMERICAL CHARACTERISTICS OF OVERHEAD CRANES LOADS ON INDUSTRIAL BUILDINGS FRAMEWORKS

The article is dedicated to the development of analytical model of loads four-wheels travelling cranes loads. The design values of mathematical expectation, variance and standard for vertical and horizontal component of crane load were received. Numerical example has shown that the values of analytical numerical characteristics are very close to the experimental values of loads, and can be applied in the reliability estimation. Analytical numerical characteristics are used in the of steel framework one-storey industrial building columns calculation reliability. The time factor and the stochastic nature of the loads and the strength of steel were considered. The effect of different parameters (cranes capacity and the mode of travelling cranes, columns step, type of connection the column and girder, the type of roofing and the values of wind and snow loads) on the reliability of steel frameworks was considered.

***Keywords:** probabilistic model, crane loads, four-wheel overhead travelling cranes, numerical characteristics.*

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ІМОВІРНІСНІ ЧИСЛОВІ ХАРАКТЕРИСТИКИ НАВАНТАЖЕНЬ МОСТОВИХ КРАНІВ НА КАРКАСИ ВИРОБНИЧИХ БУДІВЕЛЬ

Побудовано аналітичну модель навантажень чотириколісних мостових кранів. Отримано розрахункові значення математичного сподівання, дисперсії та стандарту для вертикальної й горизонтальної складової кранового навантаження. Числовим прикладом показано, що значення числових характеристик, отриманих аналітично, досить близькі до експериментальних значень навантажень, вони можуть застосовуватися при оцінюванні надійності. Аналітичні числові характеристики використано при розрахунку надійності колон сталевого каркаса ОВБ (одноповерхової виробничої будівлі) з урахуванням фактора часу та стохастичного характеру навантажень й міцності сталі. Досліджено вплив на надійність каркасів вантажопідйомності та режиму роботи мостових кранів, кроку колон, типу вузла з'єднання ригеля з колоною, типу покриття та значень вітрового й снігового навантажень.

***Ключові слова:** ймовірнісна модель, кранові навантаження, чотириколісні крани, числові характеристики, надійність колон виробничих будівель.*

Introduction. Assessment of reliability of industrial buildings with overhead traveling cranes requires the use of probabilistic characteristics of crane loads. The process of obtaining these characteristics experimentally is lengthy, time consuming and technically difficult. Therefore, the crane loads mathematical model problem is relevant and justified.

Analysis of last research sources and publications. The results of extensive experimental investigations of crane loads nature are presented in the work [1]. Experimental data were processed in the technique of random variables and random processes and the probabilistic nature of crane loads were confirmed. Standardization and development of crane loads analytical models problems are considered in works [2]. Comparative analysis of the crane loads values defined according to national and international codes [3 – 5] is presented in [6]. The refined values of crane loads allow to get more accurate reliability of industrial buildings assessment [7 – 9]. The question of structures reliability is considered in [10 – 14]. The approaches to the reliability assessment were developed using probabilistic methods describing the structures conduct under external loads. Reliability assessment of one-storey industrial buildings with overhead cranes steel frames is described in [15 – 17], where the spatial nature of the frames was refined. Furthermore, the detailed analysis of loads and review of Codes [18, 19] which classify parameters of overhead traveling cranes were done.

Unsolved aspects of the problem. Notwithstanding, crane loads probabilistic model problem describing actual nature of the crane loads on industrial building structures is not solved.

Statement of the problem. Creating the probabilistic models of vertical and horizontal loads from influences of overhead travelling cranes on industrial buildings structures were done. Crane loads numerical characteristics values of crane loads were calculated.

Basic material and results. *Numerical characteristics of the vertical crane loads*. Vertical load (Fig. 1) on the structures of different rows (columns, crane girders) was defined as:

$$\tilde{F}_{max} = \left[\frac{G_B}{2} + (\tilde{Q} + G_{crab})\frac{L_{cr} - \tilde{a}}{L_{cr}}\right]\frac{\tilde{y}}{n_0}, \quad \tilde{F}_{min} = \left[\frac{G_B}{2} + (\tilde{Q} + G_{crab})\frac{\tilde{a}}{L_{cr}}\right]\frac{\tilde{y}}{n_0}, \tag{1}$$

where G_B, G_{crab} − weight of the bridge and the crab of crane;

$\tilde{Q}$ − hoisting load;

L_{cr} − crane span;

$\tilde{a}$ − approach of the crane hook;

$\tilde{y}$ − sum of influence line ordinates;

n_0 − the number of wheels on one side of crane.

To the non-linear function (1) with three random arguments the procedure of statistical linearization was applied. In this case the mathematical expectations $\overline{F}_1$ and $\overline{F}_2$ were determined by substitution instead of random arguments the mathematical expectations of $\overline{Q}$, $\overline{a}$, $\overline{y}$. So accurate result was got because each second derivatives that define the mathematical expectation are zeros.

To calculate the dispersion of maximum crane load, the next coefficients were determined:

$$A_{1,max} = \frac{dF_{max}}{dQ} = \frac{L_{cr} - \overline{a}}{L_{cr}}\frac{\overline{y}}{n_0}; \quad A_{2,\max} = \frac{dF_{\max}}{da} = -\frac{G_{crab} + \overline{Q}}{L_{cr}}\frac{\overline{y}}{n_0};$$

$$A_{3,\max} = \frac{dF_{\max}}{dy} = \frac{1}{n_0}\left[\frac{G_B}{2} + (G_{crab} + \overline{Q})\frac{L_{cr} - \overline{a}}{L_{cr}}\right]. \tag{2}$$

Using the obtained coefficients, the dispersion of vertical crane load was difined as follows:

$$\widehat{F}_{\max}=\left(\frac{L_{cr}-\bar{a}}{L_{cr}}\frac{y}{n_0}\right)^2\hat{Q}^2+\left(\frac{G_{crab}+\overline{Q}}{L_{cr}}\frac{\bar{y}}{n_0}\right)^2\hat{a}^2+\frac{1}{n_0^2}\left[\frac{G_B}{2}+(G_{crab}+\overline{Q})\frac{L_{cr}-\bar{a}}{L_{cr}}\right]^2\hat{y}^2. \qquad (3)$$

For the estimation the precision of dispersion the mixed derivatives are calculated:

$$\frac{d^2F_{\max}}{dQda}=-\frac{y}{L_{cr}n_0};\quad \frac{dF_{\max}}{dQdy}=\frac{L_{cr}-a}{L_{cr}}\frac{1}{n_0};\quad \frac{d^2F_{\max}}{dady}=-\frac{G_{crab}+Q}{L_{cr}n_0}. \qquad (4)$$

The dispersion precision of maximum crane load was determined using the linearization procedure:

$$\Delta\widehat{F}_{\max}=\frac{1}{L_{cr}^2n_0^2}\left\{\left[(L_{cr}-\bar{a})\hat{Q}\hat{y}\right]^2+(\bar{y}\hat{Q}\hat{a})^2+\left[(G_{crab}+\overline{Q})\hat{a}\hat{y}\right]^2\right\}. \qquad (5)$$

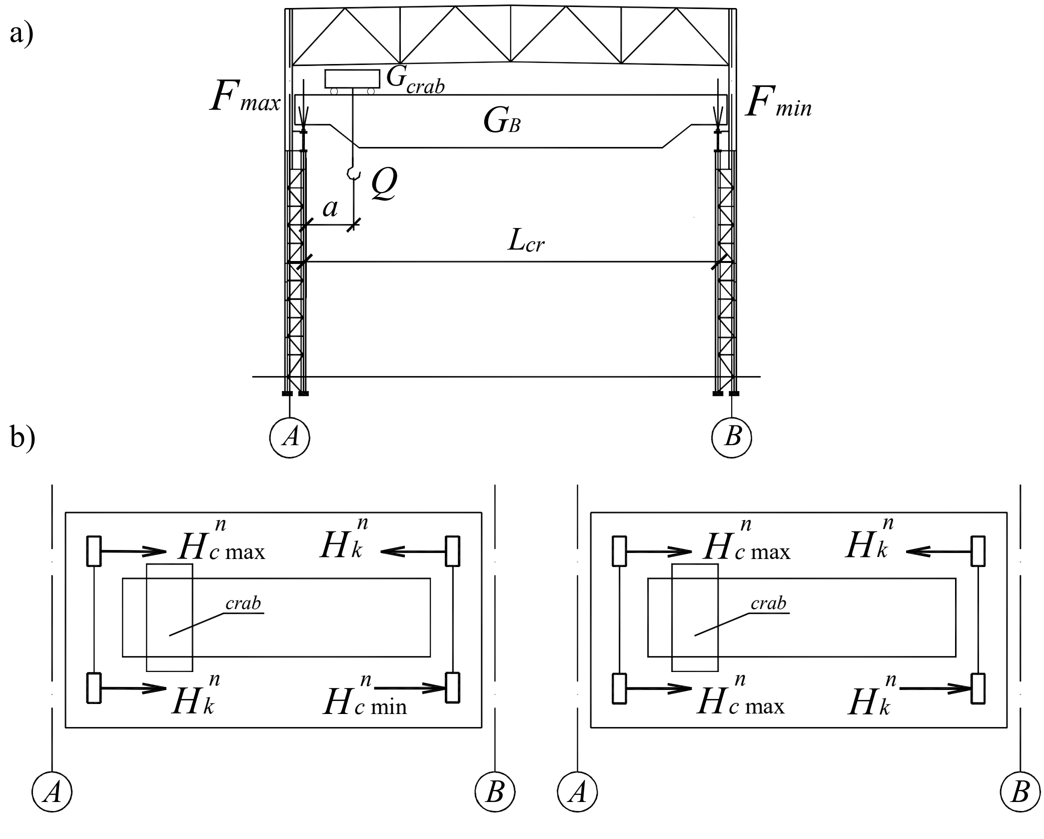


Figure 1 – Schemes of crane loads:
a – application of vertical load to the transverse frames;
b – application of horizontal loads to the crane wheels

The precision of minimum crane loads dispersion can be defined similarly.

For the numerical evaluation the crane with lifting capacity Q=50/10 t was taken. The weight distribution was taken as normal with variation coefficient V_Q=1/3 $V_Q=1/3$, distribution y − uniform. The obtained precision of dispersion was very low (2,2% $\widehat{F}_{\max}$). The obtained numerical characteristics were used to construct a graph of load normal distribution on the column. This graph well corresponds to experimental polygons of loads.

Obtained formulas allow to use simple random arguments $\tilde{Q}$, $\tilde{a}$ i $\tilde{y}$ instead of complicated experimental study of vertical crane loads. Furthermore, the available experimental data and priori reasons followed by analytical determination of the crane loads characteristics can be used.

Numerical characteristics of horizontal crane loads. To calculate the dispersion of minimum crane load coefficients are determined:

$$A_{1,min} = \frac{dF_{min}}{dQ} = \frac{\bar{a}}{L_{cr}}\frac{\bar{y}}{n_0}; \ A_{2,min} = \frac{dF_{min}}{da} = -\frac{G_{crab} + \bar{Q}}{L_{cr}}\frac{\bar{y}}{n_0};$$

$$A_{3,min} = \frac{dF_{min}}{dy} = \frac{1}{n_0}\left[\frac{G_B}{2} + (G_{crab} + \bar{Q})\frac{\bar{a}}{L_{cr}}\right]. \tag{6}$$

Then the dispersion of minimum crane load can be defined as:

$$\hat{F}_{\min} = \left(\frac{\bar{a}}{L_{cr}}\frac{y}{n_0}\right)^2 \hat{Q}^2 + \left(\frac{G_{crab} + \bar{Q}}{L_{cr}}\frac{\bar{y}}{n_0}\right)^2 \hat{a}^2 + \frac{1}{n_0^2}\left[\frac{G_B}{2} + (G_{crab} + \bar{Q})\frac{\bar{a}}{L_{cr}}\right]^2 \hat{y}^2. \tag{7}$$

The mathematical expectation of lateral forces on the wheels of four-wheel crane (Fig. 2) can be found using formula (8). These forces are limiting skewing of the bridge:

$$\bar{H}_k^n = 0{,}1\bar{F}_{\max} + \frac{\alpha\left(\bar{F}_{\max} - \bar{F}_{\min}\right)L_{cr}}{B}. \tag{8}$$

To determine the dispersion of lateral forces also linearization process can be applied and the necessary coefficients can be defined. Then the dispersion of maximum lateral forces will be:

$$\hat{H}_k^n = \left[\left(0{,}1 + \frac{\alpha L_{cr}}{B}\right)\hat{F}_{\max}\right]^2 + \left(\frac{\alpha L_{cr}}{B}\hat{F}_{\min}\right)^2. \tag{9}$$

On the other crane side lateral forces with the following numerical characteristics appear:

$$\bar{H}_c^n = 0{,}1\bar{F}_{\max} \text{ або } \bar{H}_c^n = 0{,}1\bar{F}_{\min}; \text{ or } \hat{H}_c^n = 0{,}1\hat{F}_{\min}. \tag{10}$$

The obtained formulas allow to use the numerical characteristics of horizontal crane loads in calculations and to use these characteristics for estimation reliability of industrial buildings structures.

Calculation of crane loads numerical characteristics. For the crane loads numeric characteristics definition the industrial building (with a span of 24 *m* and a columns step 6 *m*) with four-wheels traveling cranes was chosen. The cranes with operating mode 6K and the separate drive base were considered. Crane span $L_{cr} = 23{,}0\,m$ and a crane base is $B = 4{,}4$ *m*. The mathematical expectations of maximum and minimum loads on crane wheels $\bar{F}_{max} = 124{,}63$ *kN*, $\bar{F}_{min} = 67{,}87$ *kN* were calculated by substituting in (1) the numerical characteristics of all parameters. The mathematical expectations of lateral forces on the wheels of the crane were calculated using formulas (8) and (10): $\bar{H}_k^n = 15{,}43$ *kN*; $\bar{H}_c^n = 12{,}46$ *kN*.

The mathematical expectations of horizontal load on a column from lateral forces are: $\bar{H} = \bar{H}_k^n \cdot y_1 + \bar{H}_c^n \cdot y_2 = 23{,}32$ kN. Expectation and standard of lateral forces with $0{,}1F_C^{WW}$ are expressed:

$$\bar{X} = \frac{\bar{H}}{0{,}1F_C^{WW}} = 1{,}843, \ \hat{X} = \frac{\hat{H}}{0{,}1F_C^{WW}} = \frac{2{,}94}{0{,}1 \cdot 126{,}583} = 0{,}232,$$

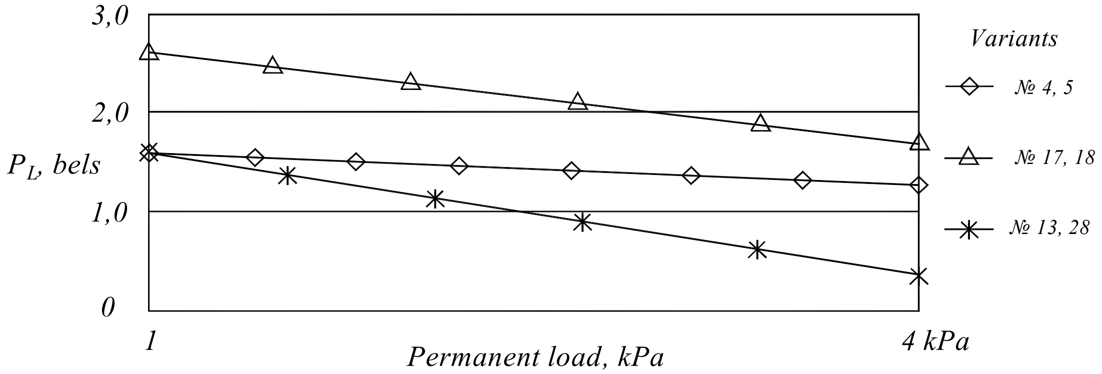
where F_C^{WW} is load on the crane column without weight.

The obtained numerical characteristics of horizontal crane load correspond to the experimental values. For the further calculations of the industrial buildings columns reliability the numerical characteristics of vertical and horizontal crane loads were worked out.

The numerical reliability evaluation of industrial buildings columns. The analytical model of crane loads was used in the calculation of columns reliability on the example of multispan industrial building to design the three-span industrial building with columns of constant cross section. The spans of building are 24 *m*, the top elevation mark of the column is +14,000.

The building columns were designed on the resistance of structures in the plane and out of the plane of dead and variable compatible effect loads action defined by DBN V.1.2-2: 2006 [7]. The structures were uploaded by random vertical loads: dead and snow loads were applied with eccentricity, the vertical crane loads and horizontal loads were distributed with wind loads. The results of probabilistic reliability calculation are shown in Fig. 2 – 4 as the probability of structures no-failure during 50 years. It was expressed in bells $P_L = -\lg[1 - P(t)]$.

The main objective of the reliability estimation was to identify the various parameters which effect on no-failure probability of structures. In particular, two types of coverings for buildings: «heavy» – prefabricated concrete panels and «light» – profiled steel (Fig. 2) were considered.



1 kPa - «light» (profiled steel); 4 kPa - «heavy» (prefabricated concrete panels)

Figure 2 – The probability of columns no-failure dependence on the coverage type

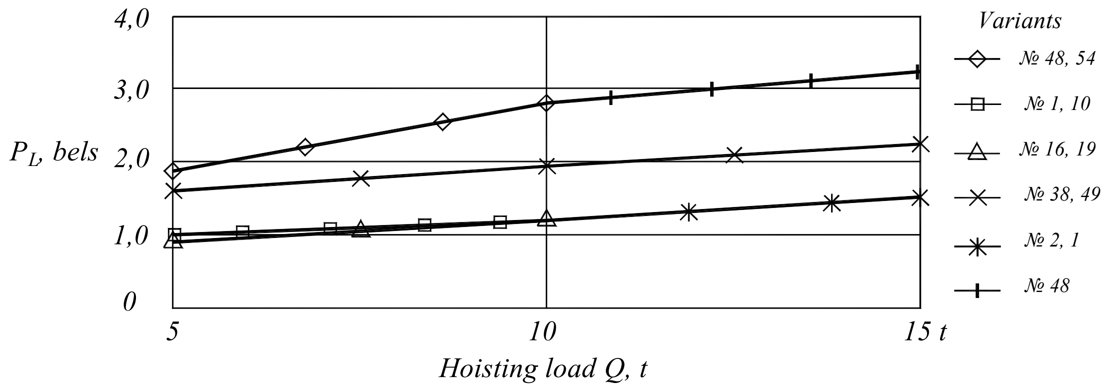


Figure 3 – The probability of columns no-failure of duty overhead cranes dependence

For each variant of covering the various ranges of building columns (Fig. 3) and types of column and girder connection (Fig. 4) were considered. In addition, the varied climatic loads were calculated (by considering the building, located in the II, III, V, and VI snow area and II, III and V wind area). Since the extreme wind load effect on the outer columns, the parameters of middle and outer columns were analyzed separately (for such columns different loading surfaces were considered). In total 56 variants of columns were worked out.

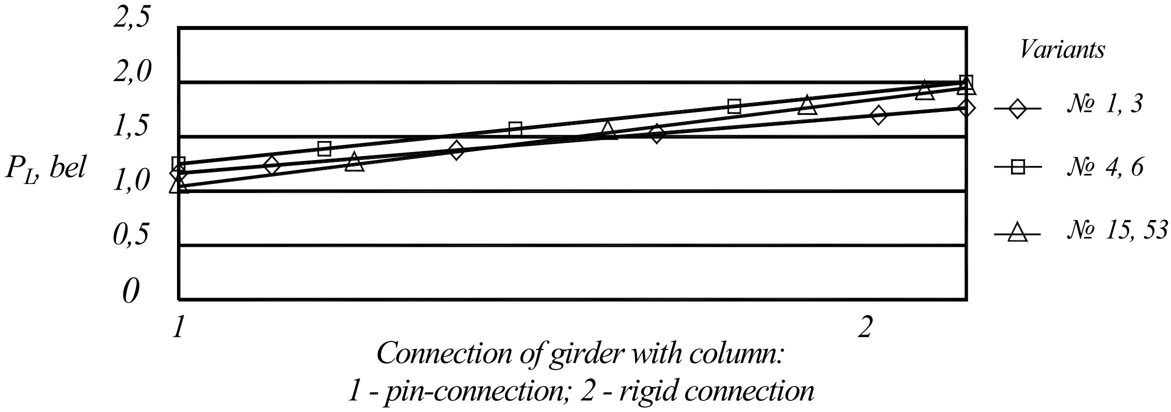


Figure 4 - The probability of columns no-failure dependence on the type of girder and column connection

Conclusions. The probabilistic model of vertical and horizontal crane loads was constructed. This model can be used for obtaining analytical stochastic loads characteristics of overhead traveling cranes and the following use of this characteristics in assessing the reliability of industrial buildings structures with overhead cranes. The obtained numerical characteristics of vertical and horizontal crane loads correspond to the experimental values, therefore characteristics can be used for reliability estimation of industrial buildings structures with crane equipment.

The numerical industrial buildings columns reliability evaluation was done considering the time factor and the stochastic loads nature and steel strength. For the investigation of the different factors as for frameworks reliability influence, the next parameters were considered: capacity and mode of overhead cranes, columns spacing, the type of column and girder connection, the coverage type and the wind and snow loads values. The tendency to increase the no-failure probability of structures with overhead cranes capacity increasing was detected. Such character of probability change can be explained by reliability stress ratio decrease of one and two overhead cranes and the growth of weight characteristic (ratio of hoist load and own crane weight). The growing of columns no-failure probability with permanent cross section of the industrial building was detected after increasing the coverage weight (due to the part of permanent load growing and reducing the part of high-frequency component of strength reserve). The redistribution of internal forces and components of columns reserve was detected by changing pin-connection to the rigid connection of column with girder. Herewith the increasing of permanent and snow loads parts and the reduction of wind and crane loads parts provide the reliability columns enhancement.

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THIN-WALLED BARRELL SHELL DEFLECTIVE MODE ANALYSIS

The paper deals with the deflective mode of steel rotary shell with different form of outer surface that are loaded with axially symmetric load. The results show solution of shell voltage and strain equation under the load that is described by exponential law and based on efforts from temperature differentials. Besides, the paper represents design formulas for deflection analysis, running bending moments and running transverse forces in shells with different abutment to the basis. It was shown the basic function of deflection and maximum value of relevant parameters of the reaction. The analysis of aspects about efficiency of using corrugated wall for steel barrel shell is done. The results in analytical and graphical form are shown. According to resulting formulas, it was made comparative calculations of shells with constant and variable wall thickness.

***Keywords:** barrel shell, exponential law, deflection functions, state of stress, zone of end effect.*

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АНАЛІЗ НАПРУЖЕНО-ДЕФОРМОВАНОГО СТАНУ ТОНКОСТІННИХ ЦИЛІНДРИЧНИХ ОБОЛОНОК

Стаття присвячена дослідженню напружено-деформованого стану сталевих оболонок обертання, з різною формою зовнішньої поверхні, звантажених осесиметричним навантаженням. Отримане рішення рівняння напружень і деформацій оболонки при навантаженні, що описується експоненціальним законом та з урахуванням зусиль від перепаду температури. Зокрема, представлені розрахункові формули для визначення прогинів, погонних згинальних моментів та погонних поперечних сил в оболонках з різним примиканням до основи. Приведені основні функції прогинів, а також максимальні значення відповідних параметрів реакції. Виконаний аналіз аспектів, щодо ефективності застосування гофрованої стінки для сталевих циліндричних оболонок. Результати представлені в аналітичному та графічному вигляді. Відповідно до отриманих формул, виконані порівняльні розрахунки оболонок з постійною та змінною товщиною стінки.

***Ключові слова:** циліндрична оболонка, експоненціальний закон, функції прогинів, напружений стан, зона крайового ефекту.*

Introduction. Steel rotary shell is a prototype of many real constructions. In particular it is the prototype of cylindrical capacities for keeping bulk material which exploration was in the last authors' works. In this paper barrel shells are examined from this point, and it is the vector to be researched. But the resulting aspects can be used in other branches of material durability and construction elements.

The analysis of the last researches and publications. A lot of national and foreign scientists explore the deflective mode of barrel shells [1 – 10]. Though the named sources are purely theoretical and the given formulas are not appropriate enough for practical solving of calculation and designing capacities for keeping bulk materials.

Parts of the general problem that were not researched earlier. The determination of the internal efforts and displacement of the thin-walled barrel shell on different boundary conditions is shown in many scientific and reference sources. However, there must be pointed out particular concepts that were not examined yet. First of all, it is to find a solution under the load. It is described by the exponential law that is typical for bulk material pressure. Secondly, there is no analysis of aspects about efficiency of using corrugated plate for the shell surface. The achieved results are useful for further examination of shells that are supported by upright stiffening rib.

Problem statement. The general aim of research is the solution of shell voltage and strain equation under the load that is described by exponential law, and derivation of practical formulas for deflection analysis, running bending moments and running transverse forces in shells with different abutment to the basis.

Main materials and results. The shell with the diameter D_w and the height H_w is loaded with axially symmetric load *p(x)*. Independent variable x indicates that radial load *p(x)* in general case can be changeable in length or function on the bounded length of shell. To determine voltage and strain of such shell, there should be solved the differential equation [1, 2, 9, 10]

$$\frac{d^4 w(x)}{dx^4} + 4k_w^4(x) = \frac{p(x)}{D_r} , \tag{1}$$

where D_r is the cylindrical stiffness of the shell on flexion in circular direction; $w(x)$ is the function of the shell body displacement.

Coefficient k_w can be defined using the equation

$$k_w^4 = Et_{ef,r} \Big/ \left(D_w^2 D_r \right), \qquad k_w^4 = Et_w \Big/ \left(D_w^2 D_r \right), \tag{2}$$

where $t_{ef,r} = t_w \, \ell_{w,1} \big/ \ell_w$ is the thickness of plates (stiffness of its equivalent to the stiffness of the corrugated profile with the thickness t_{ef}) that receive circular efforts; $l_{w,1}$ is the length of corrugation sweep on the corrugation plate of shell l_w; E is the elastic modulus of the material.

Equation (1) is used for the shell with the constant thickness t_w regardless of the surface form. It can be both smooth and corrugate. The used simplification about constancy of thickness substantiates that numerical results solving of the differentiate equation (1) with the changeable thickness nearly do not differ from solving when t_w = const if to assume the thickness of wall as the end of the shell.

By using trigonometric function, the solution for the differentiate equation (1) is the next

$$w(x) = e^{-k_w x} \left[C_1 \sin(k_w x) + C_2 \cos(k_w x) \right] + e^{k_w x} \left[C_3 \sin(k_w x) + C_4 \cos(k_w x) \right] + w_* , \tag{5}$$

where C_1, C_2, C_3 and C_4 are constant integrations, that are defined by boundary conditions, and $w_*(x)$ is the partial solution of the differentiate equation, that is defined by analytical form of putting down the function *p(x)*.

As it is know, deflection of shell that is defined by equation (5) is the two pairs of rapidly decayed periodical functions. Each of them decays depending on the distance from the top or lower edge. For the shells long enough that are met the demands of membrane theory of shells, both parts of equation for deflection have an independent meaning. The first part describes image of strained condition near the shell surface, and the second – on the top edge. Considering this feature and considering further shells pinched near the surface, it can be considered that C_3 and C_4 are equal to zero.

Considering two variants of solution, when $p(x) = p_0$ and $p(x) = p_0(1-\alpha e^{\beta x})$, it can be conformed the famous Yansen-Kenen's formula. In the first case partial solution $w_d(x)$ has an easy solve such as

$$w_* = w_0 = p_0 / (4k_w^4 D_r), \qquad (6)$$

where w_0 is an introduced designation for this equation.

Constant integrations on condition are defined, when $x = 0$ the deflection and rotation angle are equal to zero. After a number of calculations there is

$$w(x) = w_0 \left\{1 - e^{-k_w x}\left[\sin(k_w x) + \cos(k_w x)\right]\right\}, \qquad (7)$$

It must be pointed out that deflection of shells from the shaped sheet will be less only on ΔA_w quantity than the shells from the flat sheet (area ratio of cross-sections of shaped and flat sheets). Since the given quantity is not very different from the unit, shaping of sheets during the axially symmetric load has a quite indirect value. During the load $p(x) = p_0(1-\alpha e^{\beta x}) = p_0 K_p(x)$ partial solution of differential equation (1) appropriate to solve in the form of

$$w_*(x) = w_0\left(1 - \frac{\alpha\, e^{\beta x}}{1 + 0.25\zeta^4}\right), \qquad (8)$$

where $\zeta = \beta / k_w$ is the ratio of the congruent coefficient.

General solution of the differentiate equation is quite cumbersome

$$w(x) = w_0 \left\{1 - e^{-k_w x}\left[\sin(k_w x) + \cos(k_w x)\right]\right\} - \\ - w_0 \frac{\alpha e^{\beta x}}{(1 + 0.25\zeta^4)}\left\{1 - e^{-k_w x(1+\zeta)}\left[(1+\zeta)\sin(k_w x) + \cos(k_w x)\right]\right\}. \qquad (9)$$

Since for every sort of agricultural production the coefficient ζ is quite little, therefore ζ^4 acquire lesser value. It allows to equal this coefficient to zero and to put down the formula (9) in easier form

$$w(x) = w_0 K_p(x)\left\{1 - e^{-k_w x}\left[\sin(k_w x) + \cos(k_w x)\right]\right\}, \qquad (10)$$

For levels x that are detached from the shell surface, the quantity indicated in braces can be neglected and rate deflections using the equation

$$w(x) = w_0 K_p(x). \qquad (11)$$

Parameters p_0, α and β have the next value in problems of barrel shells calculation

$$p_0 = \gamma_g D_w / 4 f_g, \qquad \beta = 4\lambda_0 f_g / D_w, \qquad \alpha = e^{-\beta H_w}, \qquad (12)$$

where γ_g, f_g and λ_0 are the calculated value of specific gravity, frictional coefficient and coefficient of lateral pressure for food storage.

It should be defined how far the levels x have to be from the shell surface and to be analyzed the effect from the shaping sheets on the given quantity. There should be performed the rate of decay speed of functions (7) and (9) with the help of crest value ratio of deflections in the range of two adjacent half-waves. Let the length of half-wave be line $\lambda = \pi / k_w$. For the cases of constant and exponent load the decay speed is characterized by the quality $e^{\lambda k_w}$

(for exponential load $e^{\lambda k_w(1-\zeta)} \approx e^{\lambda k_w}$), i. e. top levels for which $x > \pi / k_w$ can be considered as distant enough from the surface. Considering that shells, consisting of shaped sheets, have the coefficient value k_w it is lesser than the shells with flat wall. That is why it has longer zone of end effect in k_λ times

$$k_\lambda = \sqrt[4]{\frac{D_r}{D_0} \frac{t_w}{t_{ef}}} \,, \tag{13}$$

where value D_0 is defined as $D_0 = Et_w^3 \Big/ \left[12(1-\mu^2)\right]$.

Perform the rate of internal efforts using the classic ratios of materials strength:

- circular normal strain

$$\sigma_h(x) = 2w(x)\frac{E}{D_w}; \tag{14}$$

- running bending moments

$$M_x(x) = -D_r \frac{d^2 w(x)}{dx^2}; \tag{15}$$

- running transverse forces

$$Q_x(x) = \frac{dM_x(x)}{dx} = -D_r \frac{d^3 w(x)}{dx^3}. \tag{16}$$

The received analytic dependences have been summarized to the Table 1, where apart from the equations for bending moments and transverse forces, the main functions of deflection and maximum value of the corresponding parameters of reaction are given.

The procedure of internal force factor receiving has to be studied additionally when the shell has a swing joint to the surface. Results in accurate mathematical statements for $M_x(x)$ and $Q_x(x)$, when the exponent load operates, are quite cumbersome. But the used simplification about lesser coefficient ζ allows significantly simplify final equations. Attention to important feature of received dependences should be paid to. If the quantity of shell deflections from flat and shaped sheets is practically not different, this assertion is not true for value of bending moments and transverse forces.

For shells performed from shaped sheets, bending moments will be bigger in k_λ^2 times, and transverse forces – in k_λ^4 times. It is explicated by enlarged length of end effect zone that in k_λ times lesser in shells with the flat wall than with the shaped. More clearly the given feature is illustrated on the Drawing 1.

I should be concentrated on one more aspect of using the differentiate equation (1). Respectively to norms of design, such as NBS (National Building Standards) [12] and Eurocode [13], spectrum of efforts from axially symmetric pressure of bulk material is always added by efforts from temperature differential Δt. Considering that this differential does not depend on height x, the additional deflections are equal to $w_t = 0.5\alpha_t D_w \Delta t$, where α_t is the coefficient of temperature expansion.

But the given deflection can be induced by some uniformly distributed load p_t over girth and height of the shell that is connected with w_t ratio

$$w_t = p_t D_w^2 / (4Et_w). \tag{17}$$

Probably the density of load

$$p_t = 2\alpha_t \Delta t E \frac{t_w}{D_w}. \tag{18}$$

Table 1 – Design equations for determination of deflections and internal efforts in shells with different abutment to the basis

№	Equation *R(x)*	x_{max}	R_{max}
The shell stiffened near the surface (uniform load)			
1	$w(x) = w_0\left\{1 - e^{-k_w x}\left[\sin(k_w x) + \cos(k_w x)\right]\right\}$	λ	$w_{\max} = 1.043 w_0$
2	$M_x(x) = \dfrac{p_0}{2k_w^2} e^{-k_w x}\left[\cos(k_w x) - \sin(k_w x)\right]$	0	$M_{x,\max} = 0.5\dfrac{p_0}{k_w^2}$
3	$Q_x(x) = \dfrac{p_0}{k_w} e^{-k_w x}\left[1 - 2\sin^2\left(\dfrac{k_w x}{2}\right)\right]$	0	$Q_{x,\max} = \dfrac{p_0}{k_w}$
The shell stiffened near the surface (exponential load)			
4	$w(x) = w_0 K_p(x)\left\{1 - e^{-k_w x}\left[\sin(k_w x) + \cos(k_w x)\right]\right\}$	λ	$w_{\max} = 1.043 w_0 K_p(\lambda)$
5	$M_x(x) = \dfrac{p_0}{2k_w^2} K_p(x) e^{-k_w x}\left[\cos(k_w x) - \sin(k_w x)\right]$	0	$M_{x,\max} = 0.5\dfrac{p_0}{k_w^2}(1-\alpha)$
6	$Q_x(x) = \dfrac{p_0}{k_w} K_p(x) e^{-k_w x}\left[1 - 2\sin^2\left(\dfrac{k_w x}{2}\right)\right]$	0	$Q_{x,\max} = \dfrac{p_0}{k_w}(1-\alpha)$
The shell with a swing joint to the surface (uniform load)			
7	$w(x) = w_0\left[1 - e^{-k_w x}\cos(k_w x)\right]$	$\dfrac{3}{4}\lambda$	$w_{\max} = 1.067 w_0$
8	$M_x(x) = \dfrac{p_0}{2k_w^2} e^{-k_w x}\sin(k_w x)$	$\dfrac{1}{4}\lambda$	$M_{x,\max} \approx 0.161\dfrac{p_0}{k_w^2}$
9	$Q_x(x) = \dfrac{p_0}{2k_w} e^{-k_w x}\left[\sin(k_w x) - \cos(k_w x)\right]$	0	$Q_{x,\max} = \dfrac{p_0}{2k_w}$
The shell with a swing joint to the surface (exponential load)			
10	$w(x) = w_0 K_p(x)\left[1 - e^{-k_w x}\cos(k_w x)\right]$	$\dfrac{3}{4}\lambda$	$w_{\max} = 1.067 w_0 K_p\left(\dfrac{3}{4}\lambda\right)$
11	$M_x(x) = \dfrac{p_0}{2k_w^2} K_p(x) e^{-k_w x}\sin(k_w x)$	$\dfrac{1}{4}\lambda$	$M_{x,\max} \approx 0.161\dfrac{p_0}{k_w^2}(1-\alpha)$
12	$Q_x(x) = \dfrac{p_0}{2k_w} K_p(x) e^{-k_w x}\left[\sin(k_w x) - \cos(k_w x)\right]$	0	$Q_{x,\max} = \dfrac{p_0}{2k_w}(1-\alpha)$

Thus, if to consider that $p_0 = p_t$, then we can use all the results that were obtained earlier for the load $p(x) = p_0$. Using reference data of NBS (National Building Standards) [14] we will obtain quantitative assessment of the quantity p_t for steel shells. Before this, we have to put down equation (18) in easy and convenient form

$$p_t \approx 5 t_w \frac{\Delta t}{D_w}. \qquad (19)$$

There should be considered the variant of solving differential equation (1) in the case when the thickness of the shell $t_{ef,r}$ is changing over the height x and find out how functional dependence $t_{ef}(x)$ influence the deflective mode of the shell in comparison with the case when $t_{ef,r}$ = const.

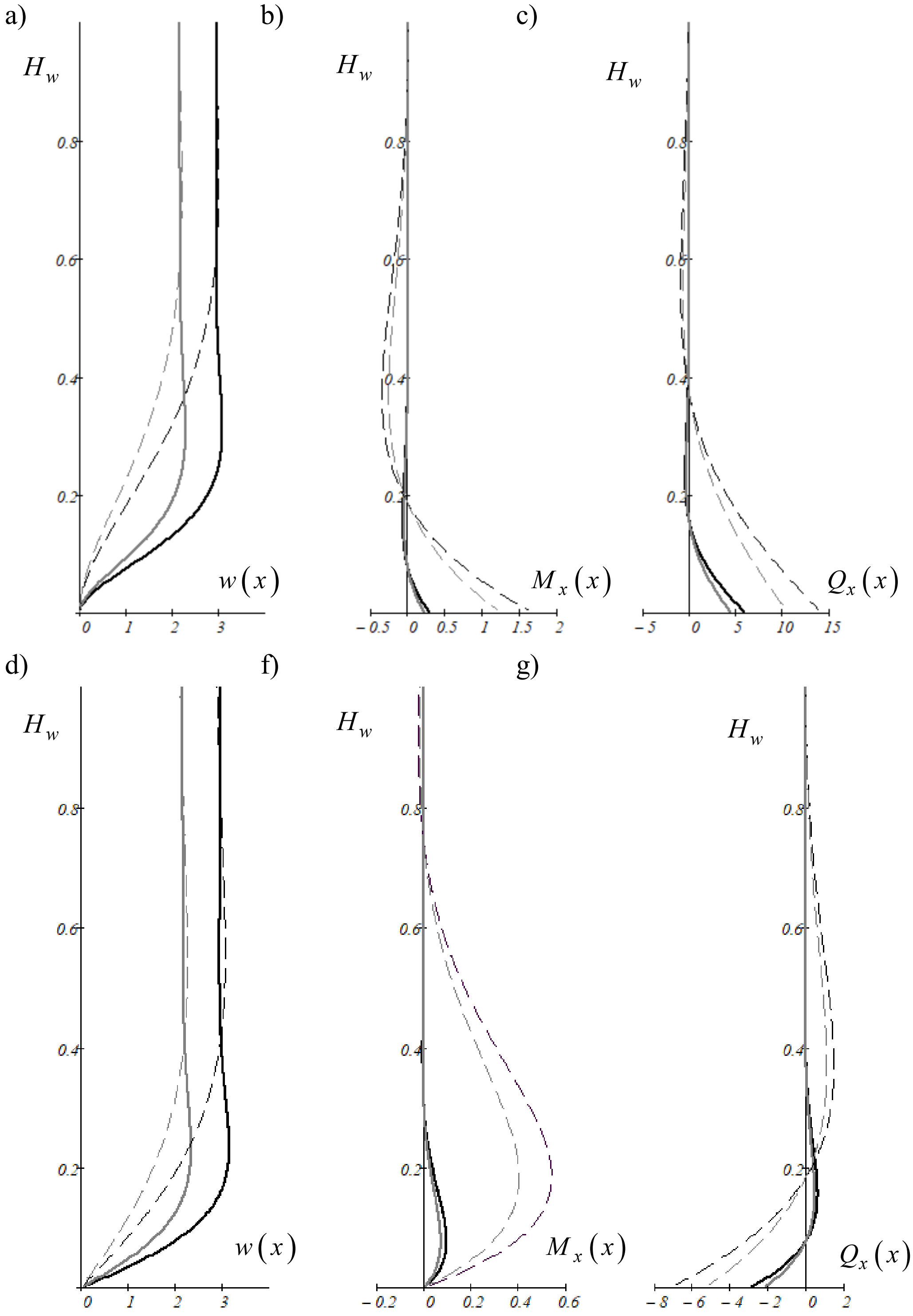

Figure 1 – Charts of deflections (a), running bending moments (b) and running transverse forces (c) for shell stiffened near the surface and with a swing joint to the surface (respectively d, f, g): solid line – flat wall; dotted line – corrugated wall; black line – uniform load; gray line - exponential load

The law of the thickness change over the height is considered in the form of power dependence

$$t_{ef}(x) = t_{ef,r} g_w(x) = t_{ef,r} \exp\left[-\varepsilon_w(\frac{x}{H_w})\right], \quad (20)$$

where $t_{ef,r}$ is the thickness of shells near the surface; ε_w is non-dimensional parameter that is responsible for the form of curve $g_w(x)$.

Since, the cylindrical stiffness D_r of the shell is the function of height x, the equation (1) must be put down in more general form [11]

$$\frac{d^2}{dx^2}\left[D_r(x)\frac{d^2 w(x)}{dx^2}\right] + \frac{4Et_{ef}(x)}{D_w^2} w(x) = p(x), \quad (21)$$

where $D_r(x)$ is the function of cylindrical stiffness of shell that accordingly to equation (20) can be represented in form of the product $D_r(x) = D_r g_w^k(x)$; for shells with flat wall $k = 3$, and with corrugated wall $k = 1$.

Considering the equation for $D_r(x)$, the differentials of equation are revealed (21)

$$\left[g_w^k(x)\frac{d^4 w(x)}{dx^4} + g_w(x)\right] + 4k_w^4 g_w(x) w(x) = p(x)/D_r \quad (22)$$

where $f_w(x)$ is additional

$$f_w(x) = 2\frac{dg_w^k(x)}{dx}\frac{d^3 w(x)}{dx^3} + \frac{d^2 g_w^k(x)}{dx^2}\frac{d^2 w(x)}{dx^2} . \quad (23)$$

Using equation (20), there is set the equation (22) more specific form. Respectively for values $k = 1$ and $k = 3$ it is

$$\frac{d^4 w(x)}{dx^4} - 2\frac{\varepsilon_w}{H_w}\frac{d^3 w(x)}{dx^3} + \frac{\varepsilon_w^2}{H_w^2}\frac{d^2 w(x)}{dx^2} + 4k_w^4 w(x) = \frac{1}{g_w(x)}\frac{p(x)}{D_r}, \quad (24)$$

$$g_w^2(x)\left[\frac{d^4 w(x)}{dx^4} - 6\frac{\varepsilon_w}{H_w}\frac{d^3 w(x)}{dx^3} + 9\frac{\varepsilon_w^2}{H_w^2}\frac{d^2 w(x)}{dx^2}\right] + 4k_w^4 w(x) = \frac{1}{g_w(x)}\frac{p(x)}{D_r} . \quad (25)$$

Differential equation (24) is simpler and its solution can be represented in the form that is similar to equation (5). Before making a general solution, take into consideration that the root of characteristic equation (24) is equal

$$\frac{\varepsilon_w}{2H_w}\left[1 \pm \sqrt{1 \pm 8i\left(\frac{k_w H_w}{\varepsilon_w}\right)^2}\right]. \quad (26)$$

Since for the barrel shells $k_w H_w / \varepsilon_w >> 1$

$$k_w\left(1 + \frac{\varepsilon_w}{2H_w k_w}\right) \pm ik_w \approx k_w \pm ik_w, \qquad -k_w\left(1 - \frac{\varepsilon_w}{2H_w k_w}\right) \pm ik_w \approx -k_w \pm ik_w . \quad (27)$$

This simplification permits not to consider completely equations (24) and (25), but to replace them by simpler analogs, which will be used next

$$\frac{d^4 w(x)}{dx^4} + 4k_w^4 w(x) = \frac{1}{g_w(x)}\frac{p(x)}{D_r} . \quad (28)$$

$$g_w^2(x)\frac{d^4 w(x)}{dx^4} + 4k_w^4 w(x) = \frac{1}{g_w(x)}\frac{p(x)}{D_r} . \quad (29)$$

Thus, roots of characteristic equation for (1) and (24) can be considered as the same, and the general solution of the differential equation (24) assume in the form of equation (5). Partial solution w_* will be researched for the load of the form $p(x) = p_0 \alpha e^{x(\beta+\varepsilon_w/H_w)}$ that takes all the analyzed variants (i. e. uniform and exponential load with different values of the parameter ε_w.

For the partial solution it is

$$w_* = \frac{p_0 \alpha e^{x(\beta+\varepsilon_w/H_w)}}{D_r\left[(\beta+\varepsilon_w/H_w)^4 + 4k_w^4\right]}. \quad (30)$$

A general solution of the differential equation (24) for the shell stiffened near the surface is function

$$w(x) = \frac{w_0 \alpha e^{xs_w}}{1+0.25s_w^4/k_w^4}\left\{1 - e^{-k_w x}\left[\left(1+\frac{s_w}{k_w}\right)\sin(k_w x) + \cos(k_w x)\right]\right\}, \quad (31)$$

where $s_w = \beta + \varepsilon_w / H_w$ is the auxiliary constant inserted for cancellation.

Considering that s_w/k_w<<1 and s^4_w/k^4_w<<1, the equation (31) can be represented in the form

$$w(x) = w_0 \alpha e^{xs_w}\left\{1 - e^{-k_w x}\left[\sin(k_w x) + \cos(k_w x)\right]\right\}. \quad (32)$$

The similar method can be used to determine the deflections of shell, the load on which is circumscribed by Yansen-Kenen exponential law or on other conditions by the abutment of the shell to the surface. Thus, when the wall thickness is changed, obtained formulas of the Table 1 can be successfully used with only one difference – the result must be multiplied by coefficient $K_\varepsilon(x) = e^{x\varepsilon_w/H_w}$.

Comparative calculations of shells with constant and variable thickness of wall indicate that more accurate calculation practically does not influence on quantity of bearing internal efforts (moments and transverse forces), but displays only on diagram of deflections. Nature of the given influence is illustrated in the Figure 2, it can be noticed that disregard of functional dependence $t_{ef,r}(x)$ can lead to underestimation of shell deflections. Expansion of the given error is influenced by increase of thickness differences of shell near its surface and near the top. Il corresponds to relatively large parameter value ε_w>1.0.

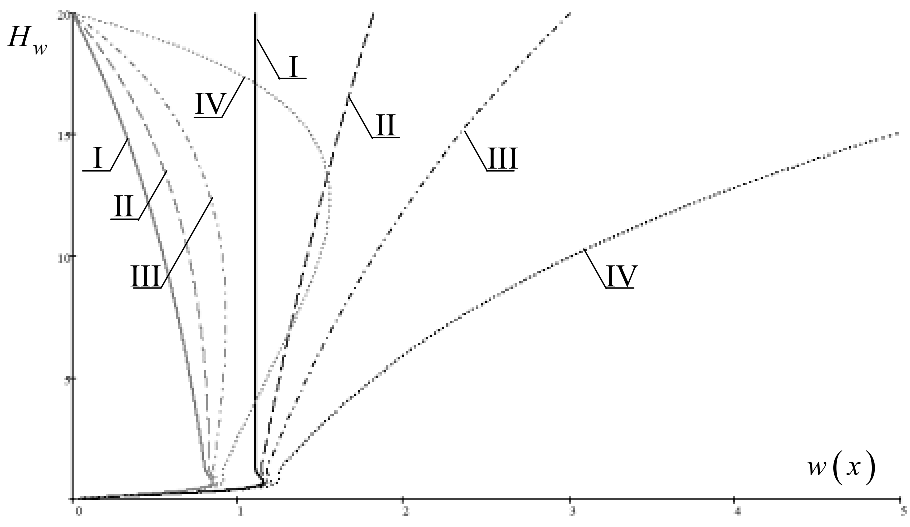

Figure 2 – Charts of deflections for shell with a stiff (swing joint) abutment to the surface:
I – ε_w= 1.0; II – ε_w= 0.5; III – ε_w= 1.0; IV – ε_w= 2.0;
black line – uniform load; gray line – exponential load

It should be mentioned that for Yansen-Kenen exponential load ordinate of maximum deflection of the shell $x_{w,max}$ is displaced to its top, when ε_w is increased. So, gripe conditions l have less influence on numeric evaluation. To obtain formula $x_{w,max}$, the Table 1 is used (p. 4 or 10). Let's differentiate any of these equations and equal the result to zero. Neglecting all the terms that are responsible for the behavior of the deflection function near the surface and liken the similar, we can obtain the next concise result; taking into account that parameter of the load α is equal to $\exp(-\beta H_w)$

$$x_{w,\max} = \frac{1}{\beta}\ln\left[\frac{\varepsilon_w \exp(\beta H_w)}{\varepsilon_w + \beta H_w}\right], \tag{33}$$

where the parameter β is determined by other equation of formulas (12) and is the function of the friction coefficient f_g and the lateral friction λ_0.

The formula indicates that $x_{w,max}$ depends on shell height H_w and diameter D_w, characteristics of the bulk material (parameters f_g and λ_0) and the nature of shell thikness changing in height (parameter ε_w). If to insert the obtained value $x_{w,max}$ in an equation from the Table 1 (p. 4 or 10), it can be determined maximum deflection of the shell that corresponds to accepted distribution of shell thickness in height.

In clear mathematic formulation the solution of differential equation (29) of shell with flat wall has considerable analytical difficulties. If to admit that function *w(x)* is not very different from its analog for shell with corrugated wall (in adjustment of cylindrical stiffness and parameter k_w), then to find a solution is easy enough and it corresponds to the circumscribed assumption.

So, formulas in the Table 1 stay relevant, considering multiple by coefficient $K_\varepsilon(x) = e^{x\varepsilon_w/H_w}$ considering distribution of thicknesses in height.

Conclusion.

1. Analytical dependences (Table 1) for deflection analysis, running bending moments and running transverse forces in shells with different abutment to the basis are obtained. the basic function of deflection and maximum value of relevant parameters of reaction have been shown.

2. The deflections of shells are relatively little influenced by shaping of sheets under the axially symmetric load and has an indirect meaning have been confirmed theoretically. But the quantities of bending moments for shells with profiling sheets are bigger in k_λ^2 times. And the quantities of transverse forces are bigger in k_λ^4 times.

3. Considering efforts from the temperature differential Δt, formula for defining the load p_t on steel shells has been obtained.

4. Dependence of internal efforts and displacements for shells with changeable wall thickness represented in analytical and graphical forms have been established.

5. Comparative calculations of shells with constant and variable thickness of the wall that correspond to formulas from the Table 1 have been made, considering multiple coefficient $K_\varepsilon(x) = e^{x\varepsilon_w/H_w}$. Distribution of thicknesses in height has been considered.

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CAPACITY FLEXIBLE COMPRESSED REINFORCED CONCRETE ELEMENTS REINFORCED WITH STEEL SHEETS

The experimental studies results of reinforced concrete elements with sheet reinforcement load bearing capacity are presented. The drawing of experimental designs is shown. The bearing capacity dependence graphs of the tested steel-reinforced concrete samples with sheet reinforcement on the height of the element and dependence graphs of tested steel-reinforced concrete samples with sheet reinforcement carrying capacity on the applied eccentricity are constructed. The photo shows the destruction character of experimental steel-concrete samples with sheet reinforcement depending on their height. The general schedule of bearing capacity dependence on the height of the element and the eccentricity of the applied load is constructed.

Keywords: *composite reinforced concrete, bearing capacity, steel sheets, amount of eccentricity.*

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НЕСУЧА ЗДАТНІСТЬ ГНУЧКИХ СТИСНУТИХ СТАЛЕЗАЛІЗОБЕТОННИХ ЕЛЕМЕНТІВ, АРМОВАНИХ СТАЛЕВИМИ ЛИСТАМИ

Наведено результати експериментальних досліджень несучої здатності залізобетонних елементів із листовим армуванням. Зображено креслення конструкцій експериментальних зразків. Побудовано графіки залежності несучої здатності випробуваних сталезалізобетонних зразків з листовим армуванням від висоти елемента; графіки залежності несучої здатності випробуваних сталезалізобетонних зразків з листовим армуванням від прикладеного ексцентриситету. Наведено фото, на яких показано характер руйнування дослідних сталебетонних зразків з листовим армуванням залежно від їх висоти. Побудовано загальний графік залежності несучої здатності від висоти елемента та ексцентриситету прикладеного навантаження.

Ключові слова: *сталезалізобетон, несуча здатність, листове армування, ексцентриситет.*

Introduction. Nowadays, reinforced concrete structures reinforced with rod reinforcement have become very popular. In these designs, concrete and reinforcement work along with the reinforcement fully absorb tensile stresses, although its work in the compressed zone is also effective. The construction of reinforced concrete structures is relevant. [4]. Use in the construction of prefabricated and monolithic structures with external sheet reinforcement requires a detailed study of their work, depending on their geometric dimensions and eccentricity of the acting load.

Review of the latest research sources and publications. Previously, experimental studies of bent elements with sheet reinforcement with different parameters were carried out [3]. To accomplish this work, compressed elements with reinforcing sheets 4 mm in thickness and a height of up to 1 m were investigated [2].

Definition of unsolved aspects of the problem. On the basis of the inspection performed during the experiments program development, the following items were planned:

– to produce prototypes of steel reinforced concrete pillars with sheet reinforcement 1000 mm high, 1700 mm, 2400 mm;

– determine the load-bearing capacity of samples as a result of centrally and centrally compressed elements with sheet reinforcement experimental studies;

– to reveal in the course of the study the peculiarities of deformations and displacements development, as well as the nature of steel-reinforced concrete elements destruction.

Problem statement. The purpose of the article is to obtain new data on the bearing capacity of central and non-centered flexural reinforced concrete reinforced concrete elements, depending on the flexibility of the samples and the eccentricity of the operating load.

Basic material and results. When compiling the experiment program, it was considered that the bearing capacity and the stress-strain state of the steel-reinforced concrete element depend on the constructive solution, the eccentricity of the load application and the physical and mechanical properties of the raw materials. The task was to experimentally determine experimentally the bearing capacity and features of work under compressed elements load with sheet reinforcement depending on their flexibility and eccentricity of the applied load.

For the production of experimental samples steel sheet t = 4 mm was used, cross-reinforcement class A-I ∅6 mm. The height of the samples was 1000, 1700, 2400 mm, the section 100x100 mm. To find out the work effectiveness of steel-concrete elements, a sample of steel without concrete with a height of 1000 mm was tested. Standard concrete cubes 150x150x150 mm and prisms 150x150x600 mm, made from the same concrete as the prototype samples were tested for definition of physical and mechanical properties of concrete filler.

Proceeding from the task, experimental designs were done for experimental ones which were divided into two groups: the first group of samples with reinforcing sheets in the plane of bending moment action (Fig. 1), the second group of samples with reinforcing sheets perpendicular to the bending moment (Fig. 2).

According to the existing normative documents on the design of bearing building constructions, it is necessary to perform calculations for both the first and the second group of boundary states. Considering this fact, experimental studies were conducted to obtain data on the bearing capacity and deformability of reinforced concrete steel racks. The developed method of conducting experimental researches and the design of prototypes conformed to these requirements.

As a result of the experiments it was established that the element height, type of reinforcement and other factors influence the process of destruction of steel concrete elements with sheet reinforcement in axial compression. In samples with sheet reinforcement in the middle section (for short samples), when the longitudinal deformations were equal to the limits of the metal fluidity, a grid in the form of Chernov lines was formed on the paint and varnish coating. On the concrete surface, free of sheet reinforcement, microcracks were combined into a macro crunch. The direction of these cracks coincided with the longitudinal axis of the prototype. Further, the sheet reinforcement releases due to the pressure of the concrete in the transverse direction with the formation of corrugations perpendicular to the longitudinal axis, on the area between adjacent rows of transverse clamps. The load resulted in destruction increases by breaking the clamps and breaking the concrete monolith. Concrete was rolled up and fell to the side, free of sheet reinforcement.

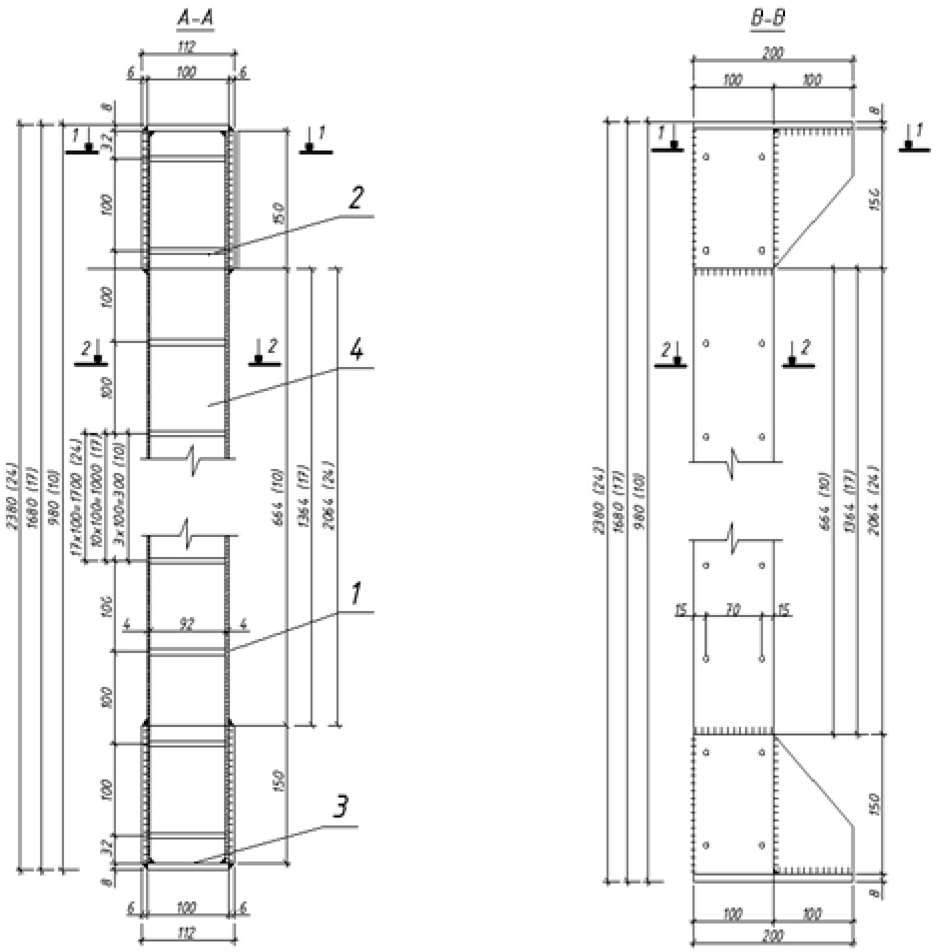

Figure 1 – Constructions of experimental samples with reinforcing sheets in the plane of bending moment action:
1 – sheet-iron plate; 2 – steel reinforcing rod; 3 – full metal; 4 – concrete

The aforementioned destruction mechanism is inherent in short central compressed samples (l / b = 4). The longitudinal axis of the destroyed samples remained straight. Samples with a height of l / b = 8..10 were destroyed by another, albeit a close circuit. The general for high samples is that under the influence of the load the distortion of the longitudinal axis occurred. This leads to an uneven distribution of longitudinal deformations in sheet reinforcement, and accordingly, corrugations were formed from the side opposite to the direction of bending. But none of the centrally compressed samples collapsed from the overall stability loss.

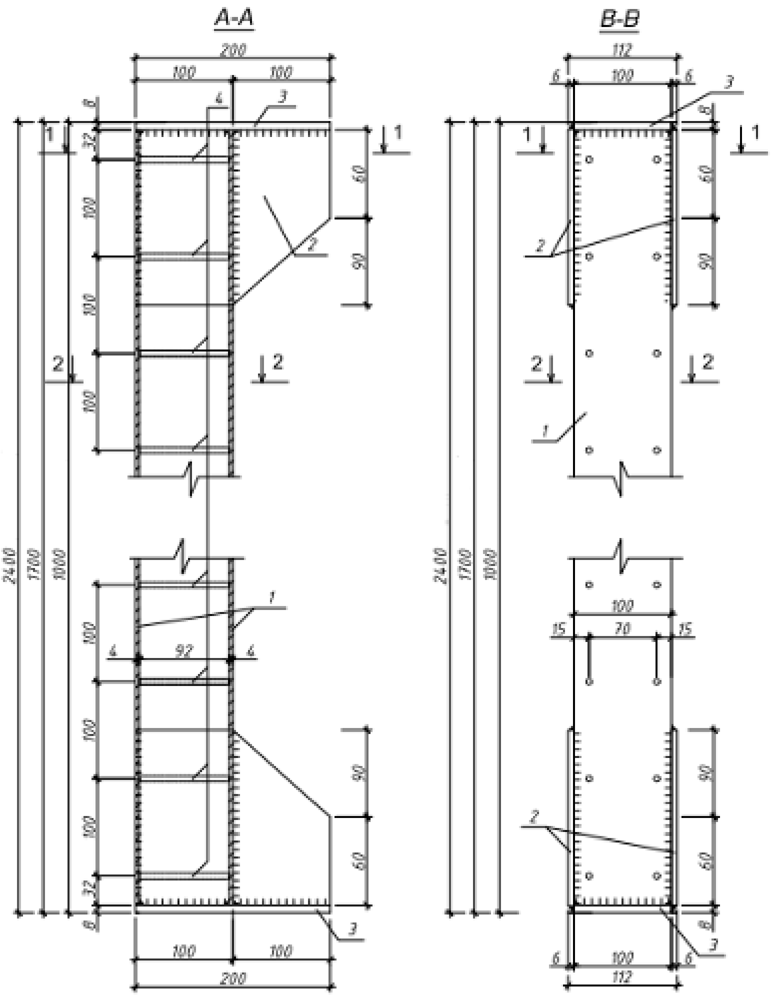

Figure 2 – Designs of experimental samples with reinforcing sheets perpendicular to the bending moment:
1 – sheet-iron plate, 2 – steel reinforcing rod, 3 – full metal, 4 – concrete

It has been established that the destruction nature of short noncentrally compressed steel-reinforced elements with sheet reinforcement depends on the magnitude of eccentricity. At an eccentricity of 1/4 of a cross-section, the mechanism of destruction is close to the destruction of centrally compressed samples, but when the the destructive force third part loading level reaches a distorted longitudinal axis. It still remains distorted until the moment of destruction.

In high noncentrally compressed steel-reinforced elements with sheet reinforcement, the curvature of the longitudinal axis was observed with the first degree of loading and increased until the time of destruction. The destruction occurred as a result of local emissions in the most compressed lane of reinforcement and simultaneous cutting of concrete. About the nature of the loss destruction samples can be judged from the photographs given in Fig. 5, 7, 9.

It should be noted that after lifting the load the longitudinal axis in all non-centered compressed steel reinforced concrete elements remained distorted and did not return to its original straight-line state. All of the above mentioned shows that the destruction of steel elements with sheet reinforcement is not brittle, as in reinforced concrete elements, and vice versa, when the load reaches a certain level, the metal elasticity limit is reached, plastic deterioration begins without reducing the loading rate.

Thus, it follows from the foregoing that for centrifugally compressed elements, as the limit, there two efforts should be taken: the first limiting effort of N_1 corresponds to the moment when the sheet of steel reinforcement is reached by the most intense fiber of the sheet strength; second N_2 is the moment of prototype destruction, where there is intense distortion of the longitudinal axis. The value of the ultimate force corresponding to the bearing capacity of the prototype is given in Table 1.

Table 1 – The bearing capacity of prototypes

Sample series	Length, L, mm	Eccentricity e_0, mm	Bearing capacity, N_1, кН	Bearing capacity, N_2, кН	N_1/N_2
SB-PD-10-1	1000	0	238	312	1,31
SB-PD -10-2	1000	25	154	198	1,29
SB-PD -10-3	1000	50	105	119	1,13
SB-PD -17-1	1700	0	234	306	1,31
SB-PD -17-2	1700	25	144	168	1,17
SB-PD -17-3	1700	50	93	105	1,13
SB-PD -24-1	2400	0	203	211	1,04
SB-PD -24-2	2400	25	138	148	1,07
SB-PD -24-3	2400	50	87	102	1,17
SB-PN-10-1	1000	0	258	319	1,24
SB-PN -10-2	1000	25	173	208	1,20
SB-PN -10-3	1000	50	108	121	1,12
SB-PN -17-1	1700	0	234	293	1,25
SB-PN -17-2	1700	25	144	168	1,17
SB-PN -17-3	1700	50	93	105	1,13
SB-PN -24-1	2400	0	184	206	1,12
SB-PN -24-2	2400	25	133	138	1,04
SB-PN -24-3	2400	50	91	94	1,03

When marking letters and figures marked: SB – samples are filled with concrete, SS – not filled with concrete; The first figure is the sample height, respectively: 10 – 1000 mm, 2 – 1700 mm, 3 – 2400 mm, the second digit is the initial eccentricity, respectively: 1–0 mm (central compression); 2–25 mm; 3–50 mm.

The test found that the experimental samples had a different bearing capacity, which depended on the constructive solution (sample height), the eccentricity of the application of the load.

The peculiarities of the work of structures with sheet reinforcement are shown by the results of testing the compressed elements shown in Fig. 3, 4, 6, 8.

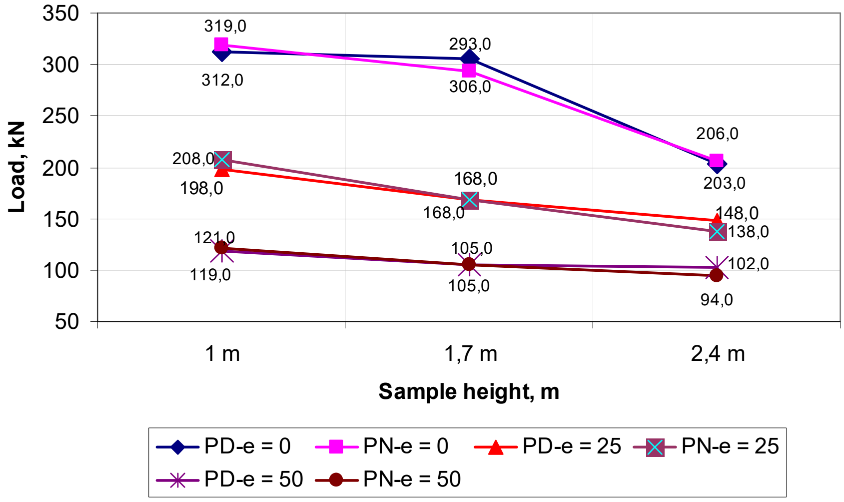

Figure 3 – Dependence of carrying capacity of tested steel-reinforced concrete samples with sheet reinforcement from element height

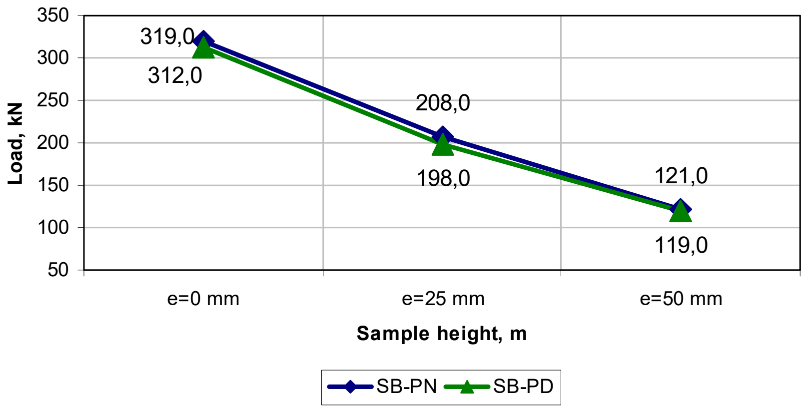

Figure 4 – Carrying capacity dependence of the tested steel-reinforced concrete samples with sheet reinforcement from the applied eccentricity at the sample height h = 1 m

Figure 5 – The destruction nature of experimental steel-concrete samples with sheet reinforcement of the series SB-PD-10-1..3 and SB-PN-10-1..3

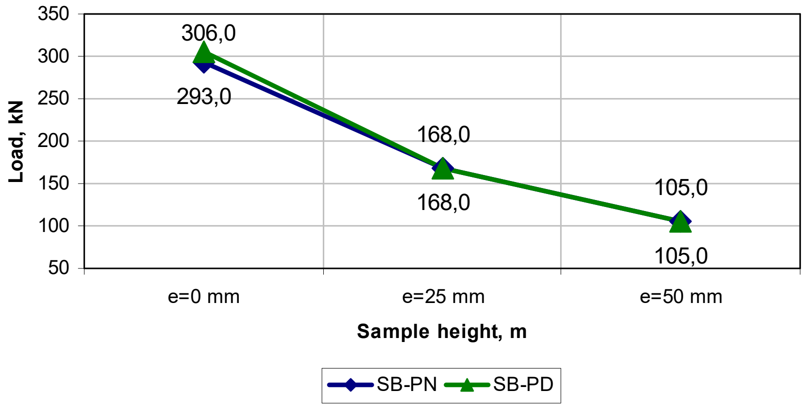

Figure 6 – Carrying capacity dependence of tested steel-concrete concrete samples with sheet reinforcement from eccentricity at samples height h = 1,7 m

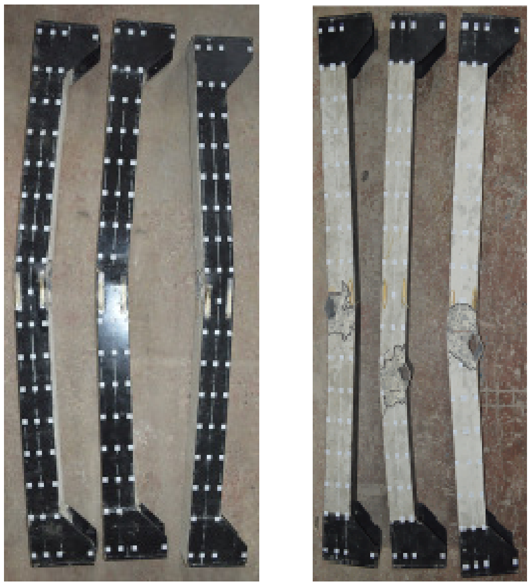

Figure 7 – The nature of the destruction of experimental steel-concrete samples with sheet reinforcement of the series SB-PD-17-1..3 and SB-PN-17-1..3

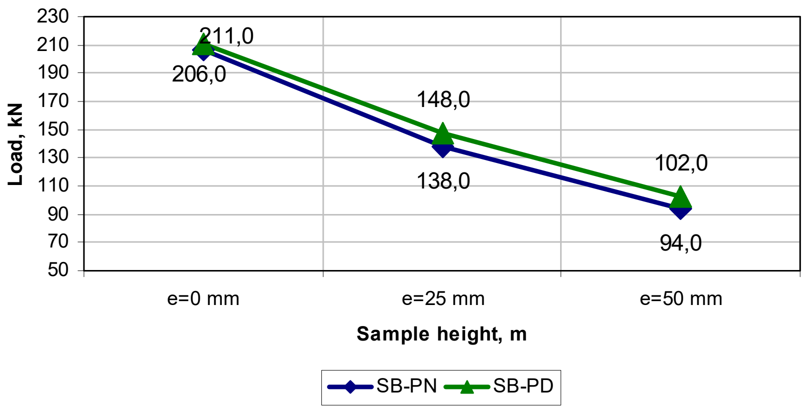

Figure 8 – Carrying capacity dependence of tested steel-reinforced concrete samples with sheet reinforcement from eccentricity at samples height h = 2,4 m

Figure 9 – The destruction of experimental steel-concrete samples with sheet reinforcement of the series SB-PD-24-1..3 and SB-PN-24-1..3 nature

The dependence of the test elements bearing capacity on their height and the applied eccentricity of the applied load is shown in Fig. 10.

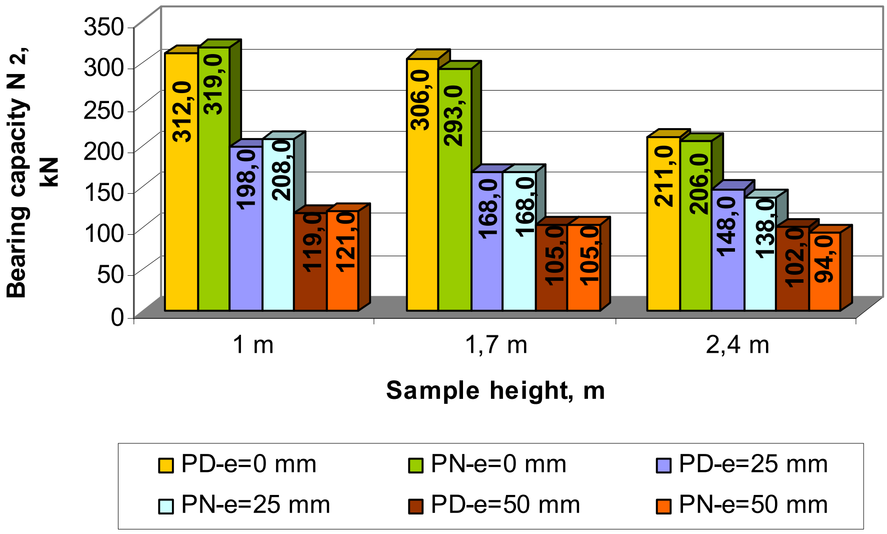

Figure 10 – Dependence of bearing capacity on the element height and applied load eccentricity

Conclusions. From the analysis of the above graphs it can be seen where extent the element height and the eccentricity of applying load are influenced by the bearing capacity of the compressed steel reinforced concrete elements with sheet reinforcement. The bearing capacity of the elements at an increase in height from 1m to 2,4 m was reduced by 35,4% with central compression, by 33,7% at center-centered compression with eccentricity of 25 mm and by 22,3% with centrifugal compression with an eccentricity of 50 mm. The bearing capacity of the tested samples practically does not depend on the location of the sheet reinforcement: in the plane or perpendicular to the plane of the operating moment. At different altitudes of the tested elements and different eccentricities, the difference between their carrying capacity was 0 – 5%.

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COMPARISON OF EXPERIMENTAL STUDIES AND NUMERICAL MODELING RESULTS OF CONCRETE FILLED TUBULAR ELEMENTS WITH DEMOUNTABLE JOINTS

The article presents the results of experimental tests of compressed tubular elements with demountable joints investigated on the central and noncentral compression (with eccentricities 0, 0,25 and 0.5 from the diameter of the sample) and numerical simulation by the finite element method. The obtained results were compared for similar samples and their models. For which using numerical simulation in the Femap software system a stress-strain state was investigated and graphical representations of principal stresses were presented. For comparison the tensions that arose when the shell's steel pipe was reached the yield strength were selected. The mean square deviation and the coefficient of variation of the data obtained varied in the range of 5 – 7%, which indicates the correspondence of the results and allows further research of partial replacement of experimental tests with numerical simulation.

Keywords: *concrete filled tubular structure, demountable joints, experimental research, numerical modeling.*

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ПОРІВНЯННЯ РЕЗУЛЬТАТІВ ЕКСПЕРИМЕНТАЛЬНИХ ДОСЛІДЖЕНЬ ТА ЧИСЕЛЬНОГО МОДЕЛЮВАННЯ ТРУБОБЕТОННИХ ЕЛЕМЕНТІВ ІЗ РОЗ'ЄМНИМИ СТИКАМИ

Наведено результати експериментальних випробувань стиснутих трубобетонних елементів із роз'ємними стиками, досліджених на центральний і позацентровий стиск (з ексцентриситетами 0, 0,25 та 0,5 від діаметра зразка) та чисельного моделювання методом скінченних елементів. Виконано порівняння отриманих результатів для аналогічних зразків і їх моделей, для яких за допомогою чисельного моделювання у програмному комплексі Femap досліджено напружено-деформований стан та наведено графічні зображення головних напружень. Для порівняння обрано напруження, що виникають при досягненні текучості сталевої труби оболонки. Середньоквадратичне відхилення та коефіцієнт варіації отриманих даних коливались у межах 5 – 7%, що свідчило про відповідність результатів, це дозволяє у подальших дослідження часткову заміну експериментальних випробувань чисельним моделюванням.

Ключові слова: *трубобетон, роз'ємні стики, експериментальні дослідження, чисельне моделювання.*

Introduction. Concrete filled tubular structures are effective combination of steel and concrete that allows to fully implement the features of such materials. For example, saving of metal and cement, reduce the size of cross section of concrete filled tube elements and as a consequence reduce the weight of the structure and transportation costs [1]. It should be noted that the joints are an important structure element of concrete filled tubular designs. One of the main features of concrete filled tube elements is thing that during design we should provide the cooperation of a steel pipe-shell and concrete core.

Over the past two decades with the significant development of computer availability and capabilities, researchers have almost unlimited prospects for calculating and modeling structures with the help of CAD (Automated Design Systems). Abroad, the terms CAD/CAE/CAM are usually used, where CAD (computer-aided design) – the use of computer technology for design. CAM (computer-aided manufacturing) – under this term is understood as the actual process of computerized production preparation and the software systems that are used in this process as well as CAE (computer-aided engineering) are the common name of software systems designed to solve various engineering tasks. For example, calculation, analysis and modeling of physical processes the most well-known among them is ABAQUS, ANSYS, ESAComp, Femap, CAE Fidesys, HyperWorks, Moldex3D, NX Nastran and many others. Calculations in these programs are carried out by solving differential equations by numerical methods (finite volume method, finite difference, finite element, etc.)

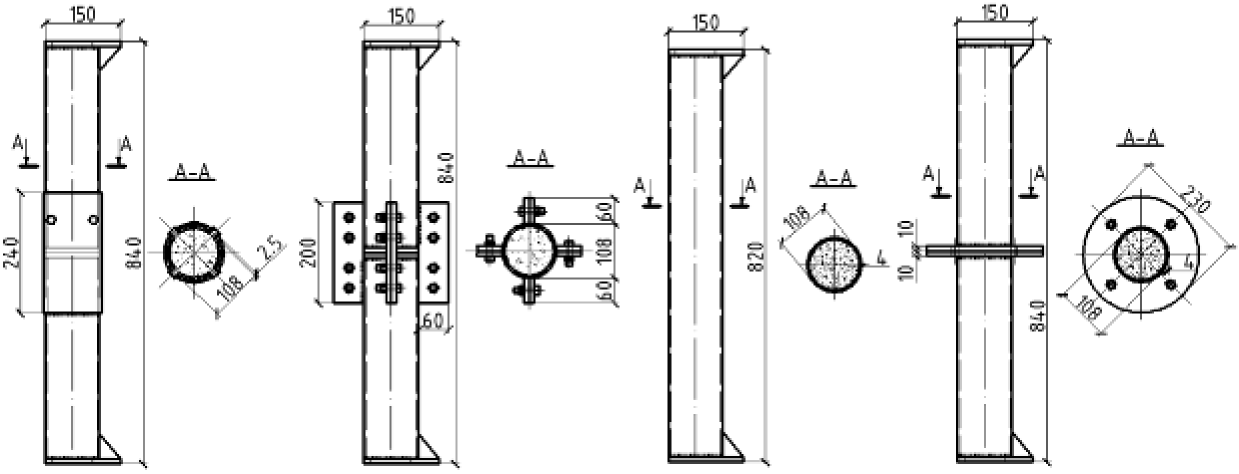
Analysis of recent research. Concrete filled tubular structures are especially actively investigated worldwide in the last 30-40 years, which was reflected in the works of: L.I. Storozhenko – research of concrete filled tubular structures [1], A.E. Lopatto – investigation of stress-strain state concrete in CFT structure [2], A. Fan – CFT elements subjected to axial compression and lateral cyclic loads [3], S. Morino – research of concrete filled tubular elements in Japan [5], Q.Q. Liang – nonlinear analysis of axially loaded CFT columns [6], S.P. Schneider – reviews of CFT frames design [7], X.L. Zhao – cold-formed tubular members and connections [8]. Specifics of numerical modeling in general and pipe structures in particular are the works of S.P. Rychkov [9].

Selection of previously unsettled parts of the general problem. Unfortunately, today the lack of data about modeling of demountable joints CFT elements. The article is devoted to comparison of experimental test results and the modeling of concrete filled tubular elements with demountable joints by numerical methods.

Problem statement. The purpose of this work was to compare the data obtained by the numerical method with the results of experimental studies of concrete filled tubular elements with demountable joints.

Main material and results. Experimental tests of concrete filled tubular elements with demountable joints were conducted. For laboratory tests joints with longitudinal ribs (TBR series) and steel couplings (TBC series) were selected as the most promising. Also for the comparison the standard flange joint (ТБF series), non-jointed concrete tubes (TB series) and 1 sample without concrete filling (T series) were investigated. Drawings and photo samples are shown in Figures 1 and 2. The common for all samples was the height of the pipe-type element – 800 mm, the diameter of the pipe (D) – 108 mm, and its thickness – 4 mm, the diameter of the bolts – 12 mm, the seam leg – 4 mm.

During the test concrete filled tubular elements with splice joints on compression, two criteria were selected for the carrying capacity of the concrete filled tubular element. The first criterion was the state of the samples, in which the deformation of the steel pipe corresponds to deformations of steel, which reached the yield strength (N1). The second is the state in which a significant deformation development occurs in a concrete filled tubular element with a constant or insignificant increase in loads, for example – in fact, this state corresponds to the destruction of the concrete filled tubular element (N2). The criterion of the bearing capacity for the joint was chosen to reach the limit of steel flux of one of its elements.



a) b) c) d)

Figure 1 – Design of prototype samples:
a) TBC series; b) TBR series; c) TB and T series; d) TBF series

Figure 2 – Photo of samples before the test

Comparing the bearing capacity of the elements with joints in the TB series, we saw that the bearing capacity of N1 was approximately equal to all experimental samples (the difference was less than 10%), analyzing the onset of destruction (N2): with central compression, samples of the TBR and TBC series had some higher bearing capacity (up to 5%); during noncentral tests difference in bearing capacity was significantly increased and amounted to 35% for samples of TBR and TBF series and up to 53% for TBC series samples.

The TBC series (from 5% to center compression up to 53% for noncentral) were the best samples of central and centripetal compression, in comparison with reference and better efficiency (from 4 to 12%), which testifies to the rationality of the construction of this type of connection at minor tensile moments in the joint (at eccentricities to 0.5 D). Table 1 shows the values of bearing capacity based on the results of experimental experiments.

A numerical simulation of 13 samples was performed for pipes without concrete, CFT samples without joints, CFT specimens with a demountable flange joint, with a joint made with the help of longitudinal ribs and a joint made using a steel sleeve. The cases of central compression and noncentral with eccentricities of applying a load equal to 0.25 and 0.5 from the diameter of the sample were considered.

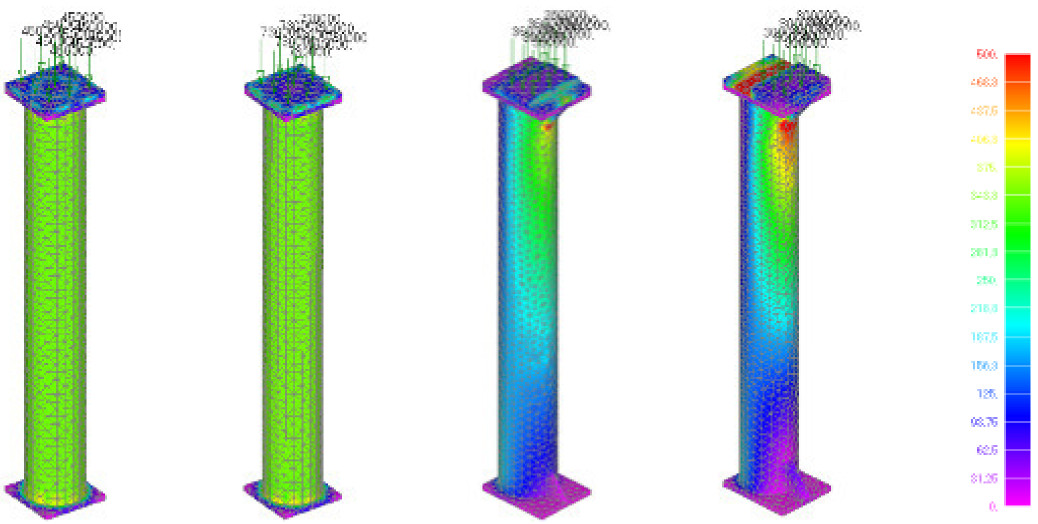
Table 1 – The value of the bearing capacity of experimental samples

№ of Sample	Eccentricity of load, mm	Bearing capacity, kN		N_2/N_1	The coefficients of efficiency of samples		
		N_1	N_2		m_1	m_2	η
T-1	0	450	580	1,29	-	-	-
TB-1	0	730	950	1,30	1,15	1,49	1,98
TB-2	27	360	465	1,29	-	-	-
TB-3	54	300	326	1,09	-	-	-
TBR-1	0	690	980	1,42	1,08	1,54	2,14
TBR-2	27	410	580	1,41	-	-	-
TBR-3	54	320	440	1,38	-	-	-
TBC-1	0	725	996	1,37	1,14	1.56	2.23
TBC-2	27	400	620	1,55	-	-	-
TBC-3	54	280	500	1,79	-	-	-
TBF-1	0	725	900	1,29	1.10	1.41	1.71
TBF-2	27	400	610	1,53	-	-	-
TBF-3	54	280	440	1,57	-	-	-

Numerical simulation of concrete filled tubular T and TB series samples.

Modeling of samples by the finite element method confirmed the typical case of destruction, as can be seen from Fig. 3 where the main stresses obtained from numerical simulation are depicted. For samples with a random (central) loading application, the values of stresses were same for almost all body of the studied element, with growth near the heads, which led to the formation of corrugations that were observed during experimental tests.

For noncentral compressed concrete filled tubular elements TB-2 and TB-3, as can be seen from Fig. 3 were characterized by uneven distribution of stresses, with a gradual increase in the approach to the upper steel head, which is confirmed by experimental data.



T-1 TB-1 TB-2 TB-3

Figure 3 – Graphical representation of main stresses in numerical simulation for the studied samples of T and TB series

Numerical simulation of concrete filled tubular TBR series samples

Piping elements were investigated for central and noncentral compression. At central compression, in fact, two rigidly connected with the longitudinal edges of concrete filled tubular elements of the analogous sample TB-1 were tested resulting in uniform distribution of stresses with an increase near the upper steel head which led to the formation in this place of corrugations. However, the test compound area was reinforced by ribs, which led to a decrease in stresses in this area. This was confirmed both by mathematical modeling and by the nature of the deformations fixed in the experiment (Fig. 4).

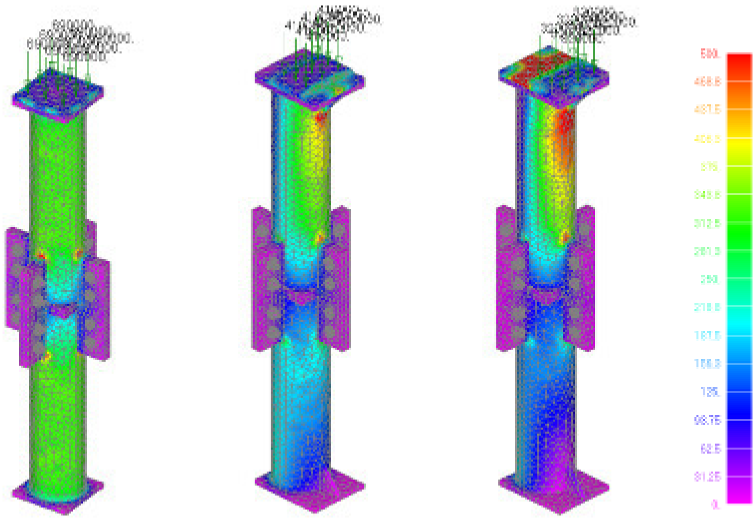

TBR-1 TBR-2 TBR-3

Figure 4 – Graphical representation of main stresses in numerical simulation for the investigated samples of the TBR series

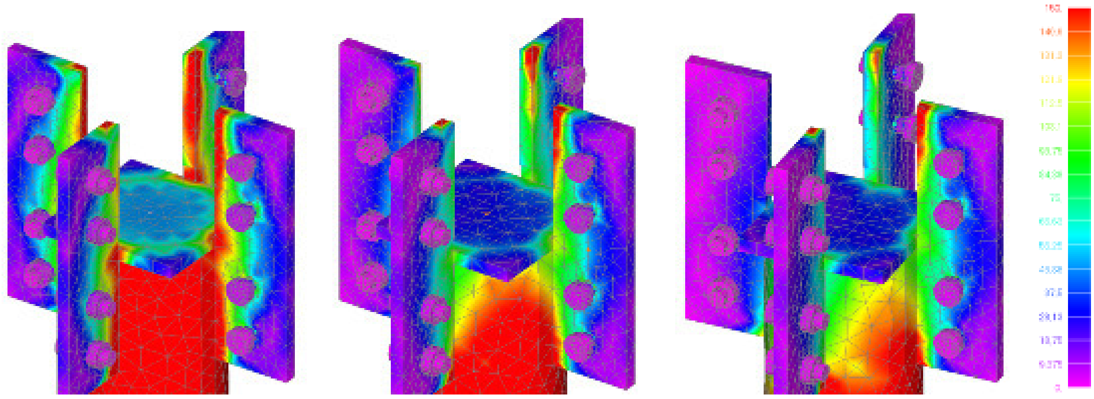
For noncentral compressed TBR-2 and TBP-3 samples, in addition to the characteristic of all concrete filled tubular elements with a similar application of the load, a significant increase in stresses near the upper steel flange attention was drawn to the reduction of stresses in the lower CFT element with magnitude of eccentricity. Also, for all samples of this series the fact that there was neither a bolt cut nor a deformation of the steel longitudinal flanges, the proof of which is both mathematical modeling and experimental data, is common.

In the places of the connection of longitudinal ribs with concrete filled tubular elements there were places of concentration of stresses.

In Fig. 5, the detail of the concrete filled tubular element is shown in more detail. It should be noted that at the central compression of the stress in the joint were larger than at the noncentral. For the given structure the impact of the size of the load which acted like a little more influence than the eccentricity of the application of the load. This corresponds to the experimental data according to which the deformations of the ribs under central compression were higher than in the noncentral.

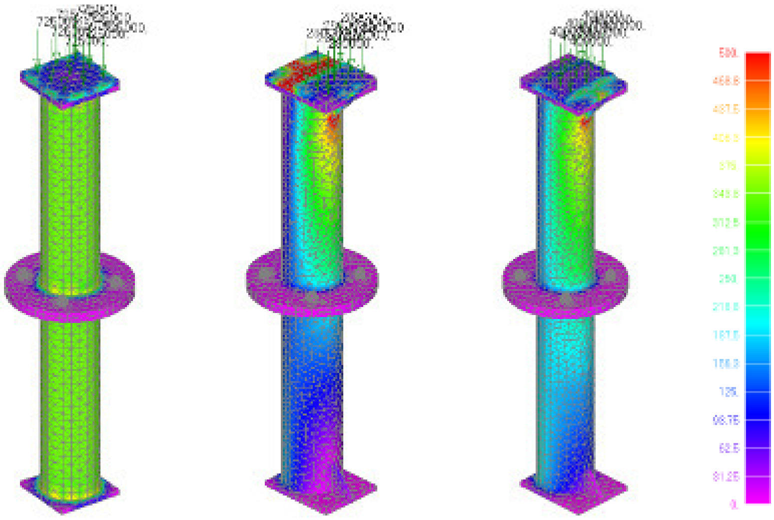
Numerical simulation of concrete filled tubular TBF series samples

The samples of the TBF series were similar to other prototype designs of the concrete filled tubular element and differed by the method of the demountable connection, this joint was carried out using steel round flanges with a milled surface welded to the upper or lower element and connected by bolts. The load was applied centrally and with eccentricities equal to 0.25 and 0.5 from the diameter of the pipe. For the boundary conditions, fixing was adopted similar to that which was during the experiment (Fig. 6).



TBR-1 TBR-2 TBR-3

Figure 5 – Graphic representation of main stresses in the lower part of the joint with longitudinal ribs



TBF-1 TBF-2 TBF-3

Figure 6 – Graphical representation of main stresses in numerical simulation for the investigated samples of the TBF series

In mathematical modeling, the fact that the presence of a joint at central compression had virtually no effect on the bearing capacity of the structure, the onset N_1 when simulations occurred at a load similar to the experimental one, which testifies to the reliability of numerical simulation.

The peculiarity of noncentral compressed samples is that the stresses in the steel flanges compared to the stresses in the concrete filled tubes are very small, even with the growth of the eccentricity of the load. It is also characteristic that the stresses and deformation, respectively in the upper concrete filled tubular elements were significantly higher than the lower ones. For the existing joint is actually an enhancement of the entire structure all specimens with joints when applying the load with the eccentricity 0,5 of diameter had a carrying capacity higher than a solid sample of more than 30%.

Numerical simulation of concrete filled tubular TBC series samples

Considering the case of central compression of the TBC-1 sample, it is noteworthy that due to somewhat more load compared with other samples of tension in the upper concrete cell were 10–15% more prominent than the lower ones, which are uncharacteristic for centrally loaded samples of other series in this experiment. and is explained by the influence of the steel coupling. Also very interesting is that the greatest stresses were in the steel coupling exactly when the central compression because in other cases the concrete element lost load bearing capacity at the upper part at significantly lower loads (Fig. 7).

Considering the stress-strain state of the steel coupling, it should be noted the emergence of small stress concentrators near the bolt holes. Similarly, the junction with the longitudinal ribs of stress was slightly larger at the central compression, especially in the lower part of the coupling (Fig. 8).

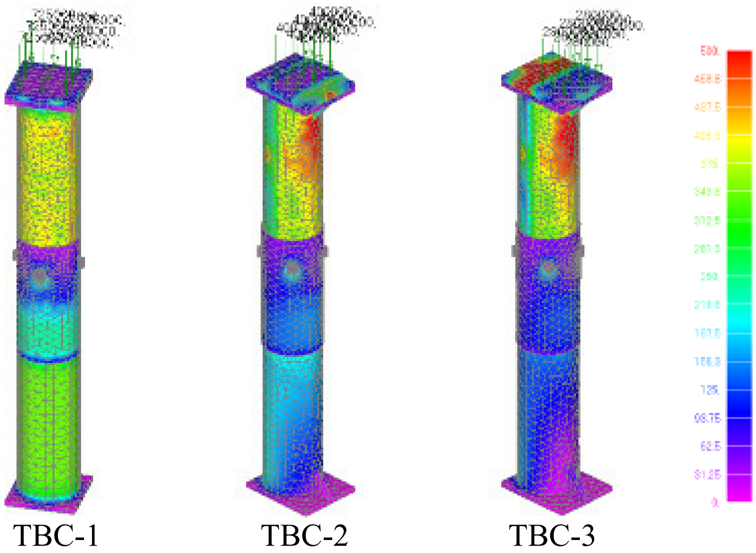


Figure 7 – Graphic representation of main stresses in numerical simulation for the studied samples of the TBC series

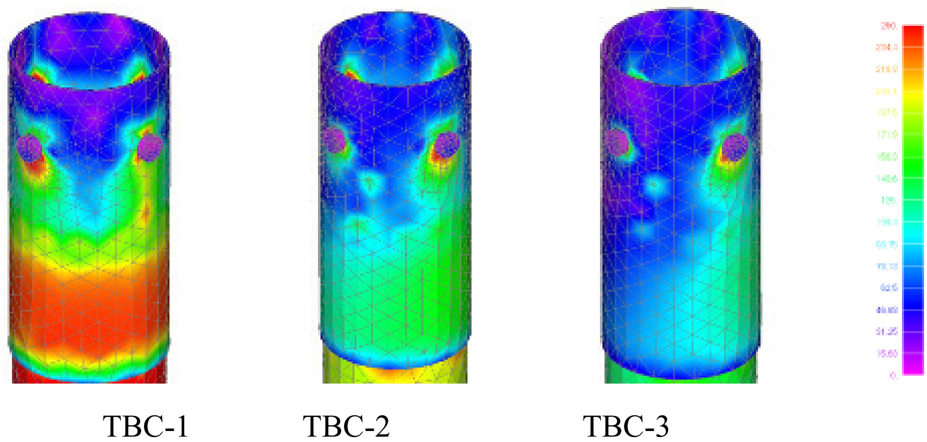


Figure 8 – Graphical representation of main stresses in the lower part of the coupling with a steel coupling

Table 2 presents a comparison of the results obtained during the experimental studies of concrete filled tubular elements and numerical simulation. The value of theoretical stresses was obtained from the software complex Femap. The mean square deviation of stresses in the compressed zone was 6.7%, the coefficient of variation was 6.8%; in a stretched mean square deviation of 4.96%, the coefficient of variation is 5.02%. Based on the above values, it can be said that the difference in the results is insignificant and varies within the variability of the materials, which confirms the objectivity and realism of the mathematical modeling of compressed concrete filled tubular samples in the software package Femap.

Table 2 – Comparison of experimental and theoretical stresses

№of Sample	e, mm	Bear. cap. N_1, kN	$\sigma_{exp.com}$ MPa	$\sigma_{mod.com}$ MPa	$\sigma_{i,\ com.}$	$\sigma_{std\ com.}$	$\sigma_{exp.ten}$ MPa	$\sigma_{mod.ten}$ MPa	$\sigma_{i,\ ten.}$	$\sigma_{std\ ten.}$
T-1	0	450	327	343	0,9534	0,00166	327	343	0,9534	0,00123
TB-1	0	730	336	345	0,9739	0,00040	336	345	0,9739	0,00021
TB-2	27	360	273	253	1,0791	0,00723	147	142	1,0352	0,00219
TB-3	54	300	332	293	1,1331	0,01934	72	78	0,9231	0,00427
TBR-1	0	690	368	334	1,1018	0,01161	368	334	1,1018	0,01285
TBR-2	27	410	340	347	0,9798	0,00020	42	40	1,0500	0,00379
TBR-3	54	320	316	341	0,9267	0,00454	120	122	0,9836	0,00002
TBC-1	0	700	310	334	0,9281	0,00434	310	334	0,9281	0,00363
TBC-2	27	400	331	340	0,9735	0,00042	48	51	0,9412	0,00223
TBC-3	54	280	326	345	0,9449	0,00241	119	117	1,0171	0,00082
TBF-1	0	725	331	334	0,9910	0,00001	331	334	0,9910	0,00001
TBF-2	27	400	349	342	1,0205	0,00070	76	79	0,9620	0,00070
TBF-3	54	280	330	360	0,9167	0,00599	183	185	0,9892	0,00000

Conclusions. After performing numerical simulation of concrete filled tubular elements with demountable joints, the following conclusions can be made:

For all investigated samples that were tested experimentally, identical real mathematical models were created. The stress-strain state was investigated using numerical simulation in the Femap software system and graphical representations of the main stresses were presented that allow estimating the stress at any point of the sample, stress concentration areas, etc.

The stresses in the compressed and stretched zone of the concrete filled tubular element were compared in accordance with the experimental data and, consequently, the results of numerical simulation. The mean square deviation and the coefficient of variation of the data obtained fluctuated within the range of 5-7%, which is admissible and suggests that the modeling results correspond to the experiment.

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DURABILITY OF HEAVY CONCRETE USING BOILER SLAGS WITH CIRCULATING FLUIDIZED BED

The results of studying the influence of boiler ash slags with a circulating fluidized bed on the freeze-thaw resistance of heavy concretes are presented. The following materials were used in the studies: Portland cement PPC 500 N, sand with the fineness modulus M_f =1.05, crushed granite fraction 5-10 mm, boiler ashes with circulating fluidized bed, hyperplasticizer «Fluid Premia-196». The study was performed using mathematical planning of the experiment. It is proved that with the replacement of sand with ashes, the freeze-thaw resistance is somewhat reduced, but the hyperplasticizer compensates the reduction of freeze-thaw resistance by reducing the W/C ratio, resulting in the formation of super-fine pore structure of concrete. Fine pores in the concrete structure compensate the ice formation stress at low ambient temperatures. The optimal cement consumption has been established in terms of freeze-thaw resistance, both at full and partial replacement of sand with ash. It was also determined that the optimum should be considered the consumption of a hyperplasticizer in the amount of 1.2-1.4% of the cement mass.

Key words: *ash and slag, fluidized bed, freeze-thaw resistance, mathematical planning.*

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ДОВГОВІЧНІСТЬ ВАЖКИХ БЕТОНІВ З ВИКОРИСТАННЯМ ЗОЛОШЛАКІВ КОТЛІВ ІЗ ЦИРКУЛЯЦІЙНИМ КИПЛЯЧИМ ШАРОМ

Наведені результати дослідження впливу золошлаків котлів із циркуляційним киплячим шаром на морозостійкість важких бетонів. В дослідженнях було використано портландцемент ПЦ 500Н, пісок з модулем крупності $М_{кр}$ = 1,05; щебінь гранітний фракції 5-10 мм; золошлаки котлів із циркуляційним киплячим шаром, гіперпластифікатор «Fluid Premia-196». Дослідження проведені з використанням математичного планування експерименту. Доведено, що із заміщенням піску золошлаками морозостійкість дещо знижується, але гіперпластифікатор сприяє компенсації зниження морозостійкості за рахунок зниження В/Ц відношення, як наслідок, утворення супердрібної порової структури бетону. Тонкі пори у структурі бетону компенсують напруження від процесу утворення льоду при низьких температурах навколишнього середовища. Встановлено оптимальна витрата цементу з точки зору морозостійкості, як при повній заміні, так і при частковій заміні піску золошлаки. Визначено також, що оптимальним слід вважати витрати гіперпластифікатора в кількості 1,2 - 1,4% від маси цементу.

Ключові слова: *золошлаки, киплячий шар, морозостійкість, математичне планування.*

Introduction. Destruction of concrete in a saturated water state influenced by the cyclic action of positive and negative temperatures, as well as varying negative temperatures, is due to a number of physical corrosion processes causing deformations and mechanical damages to products and structures.

Freeze-thaw resistance of concrete depends on its structure, particularly on the porosity nature, as the latter will determine the volume and distribution of ice that will be formed in the concrete body at low temperatures, and, therefore, the value of the stress produced during the process of the concrete structure weakening.

Literature review: In concrete micropores sizing 10^{-5} cm, there is usually bound water that does not turn into ice even at extremely low temperatures (up to –70°C), therefore micropores do not significantly affect the concrete freeze-thaw resistance [1]. The latter depends on concrete macropores and on their structure.

A number of laboratory studies have demonstrated that concrete containing fly ash and ash-slag may be less resistant to frost during freezing and thawing [2 – 4], while low-cement concretes or high-cement concretes have lower freeze-thaw resistance, but part of the cement in them is replaced with ash – fly ash or ash slag [5, 6].

Concrete with fly ash can provide satisfactory freeze-thaw resistance, provided that waterproof cement is used and W/C (water-cement ratio) does not exceed 0.45, and the fly ash content does not exceed 20-30%. In this case, of course, it is assumed that the concrete has an adequate porous structure [7].

The degree of ashes and ash slag influence on the concrete properties not only depends on their amount in the mixture, but also on other parameters, including the composition and ratio of other ingredients in the concrete mixture, the type and size of the particular component, the compaction conditions during the forming and the hardening conditions, as well as construction methods [8, 9].

Aim of the study: analyzing the influence of ash slabs of boilers with circulating fluidized bed on freeze-thaw resistance, and hence on the durability of heavy concretes designed for operation in the climatic conditions of Ukraine.

Materials and methods of the study: The following materials were used in the work: Portland cement PPC 500 N produced by «Haldeberg zement Ukraine»; sand with the fineness modulus M_f = 1.05 taken from the local deposits; ashes of boilers with circulating fluidized bed [9]; "Fluid Premia-196" hyperplasticizer based on modified polycarboxylates; crushed granite fraction of 5-10 mm taken from Kremenchuk deposit. For more complete detection of the ash slag and hyperplasticizer's influence on the concrete freeze-thaw resistance, a three-level experiment planning matrix was implemented in the study.

Freeze-thaw resistance was determined by the rapid method. According to DSTU BV 2.7-47-96 several methods for determining the freeze-thaw resistance of concrete, including two rapid methods are established. The dilatometric method of determining the freeze-thaw resistance by freezing in the kerosene medium was used in the work. According to this method, the freeze-thaw resistance is determined by the maximum difference between volumetric deformations of concrete and standard samples. The standard sample is an aluminum cube with a side length of 100 mm. Measurement of volumetric deformations was carried out using the «Beton-Frost» dilatometer (Fig. 1).

When planning the experiment the input parameters were taken:

X_1 – cement consumption;

X_2 – hyperplasticizer consumption;

X_3 – degree of sand replacement with ash slabs.

Terms of the experiment planning are presented in the Table. 1.

Figure 1 – Illustration of the dilatometer with the measuring electronic unit

Table 1 – Terms of the experiment planning

Level	Variable factors		
	X_1	X_2	X_3
maximal	1	1	1
medium	-1	-1	-1
minimal	0	0	0

Concrete samples were made according to the mathematical planning matrix of the experiment. For each set, three test cubes with an edge length of 100 mm were made. After 28 days, the samples were exposed to water saturation according to the following procedure: the first day samples were immersed in water at the depth of 1/3 height, the second day at 2/3 height, the third day the samples were completely immersed in water and the thickness of water over the sample was 5 cm. For three days of water saturation the exact volume was determined for each sample by the method of weighing in water. All the three samples of each set were tested for freeze-thaw resistance. The samples were placed into the measuring chamber of the device, the chamber was filled with kerosene and sealed hermetically, whereupon the device was fit into «Feutron» climate chamber. The chamber was connected to the electronic unit via cables. Freezing of the device with samples lasted for 4.5 hours. During this period, the electronic unit was recording changes in the sample's deformation. For all the samples, the maximum relative increase in the difference between the volume deformations Θ was determined by the formula

$$\Theta = \frac{\Delta V_i}{V_o}$$

where ΔV_i – maximum difference of deformation values for the concrete and the standard samples at freezing;

V_o – initial volume of the sample,см^3. According to this difference, using the table of DSTU B V. 2.7-47-96, the concrete freeze-thaw resistance was determined.

Basic material and results. The results of studies on concretes produced according to the experiment planning matrix are presented in Table 2.

According to the study results, the surfaces of the experiment planning input parameters' influence on the concrete freeze-thaw resistance were constructed, and are shown in Fig. 2 to 4.

Table 2 – Results of testing concretes for freeze-thaw resistance

№	Variable parameters			Freeze-thaw resistance		
	X_1	X_2	X_3	$\Theta = \frac{\Delta V_i}{V_o}$	Number of cycles	Respective grade, F
1	600	2	ash	0.304	480	400
2	400	2	ash	0.783	238	200
3	600	0.8	ash	0.548	332	300
4	400	0.8	ash	1.77	105	100
5	600	2	sand	0.147	633	600
6	400	2	sand	0.586	380	300
7	600	0.8	sand	0.184	580	500
8	400	0.8	sand	0.538	365	300
9	600	1.4	0.5+0.5	0.154	538	500
10	400	1.4	0.5+0.5	0.522	360	300
11	500	2	0.5+0.5	0.313	487	400
12	500	0.8	0.5+0.5	0.319	408	400
13	500	1.4	ash	0.362	390	300
14	500	1.4	sand	0.298	435	400
15	500	1.4	0.5+0.5	0.299	424	400
16	500	1.4	0.5+0.5	0.309	416	400
17	500	1.4	0.5+0.5	0.247	464	400

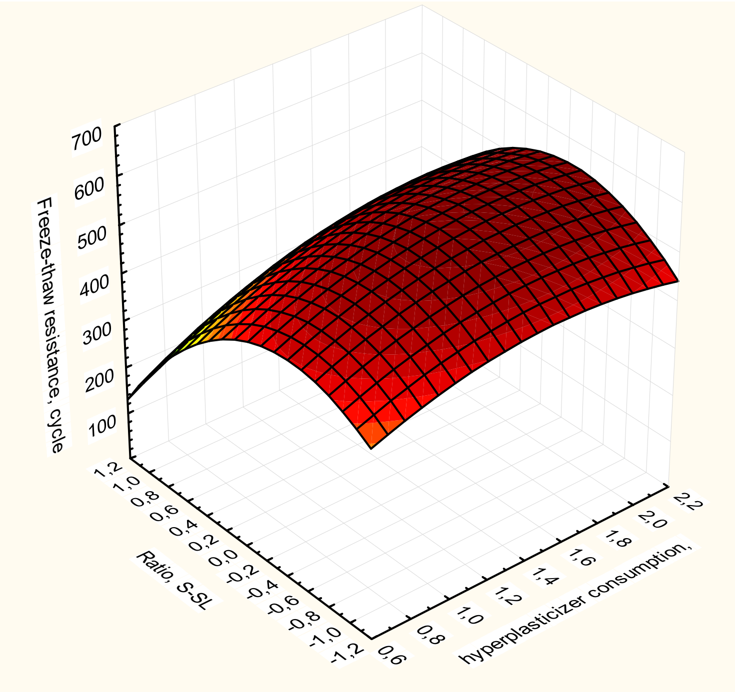

Figure 2 – Surface of hyperplasticizer consumption and the degree of sand replacement with ash slag influence on the concrete freeze-thaw resistance

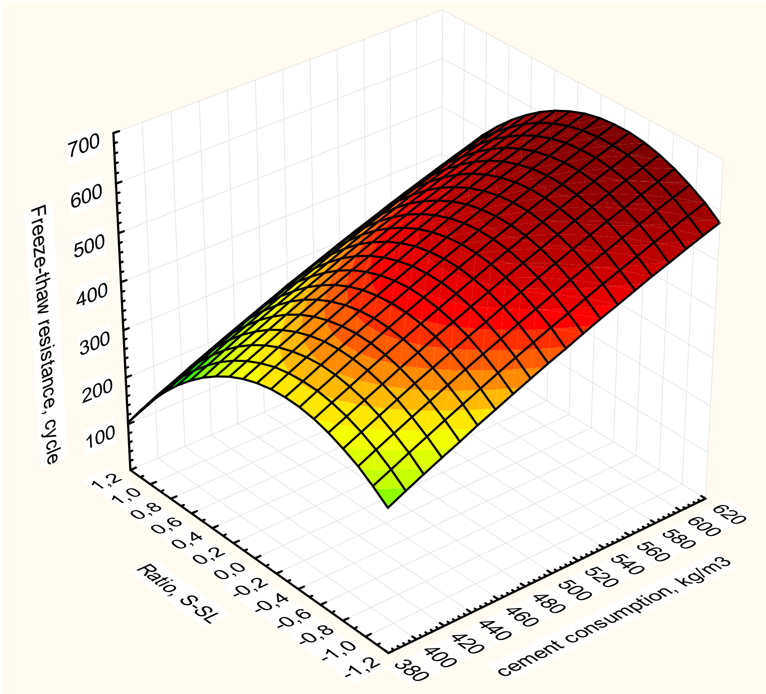

Figure 3 – Surface of cement consumption and S-AS (sand – ash slag) ratio influence on the concrete freeze-thaw resistance

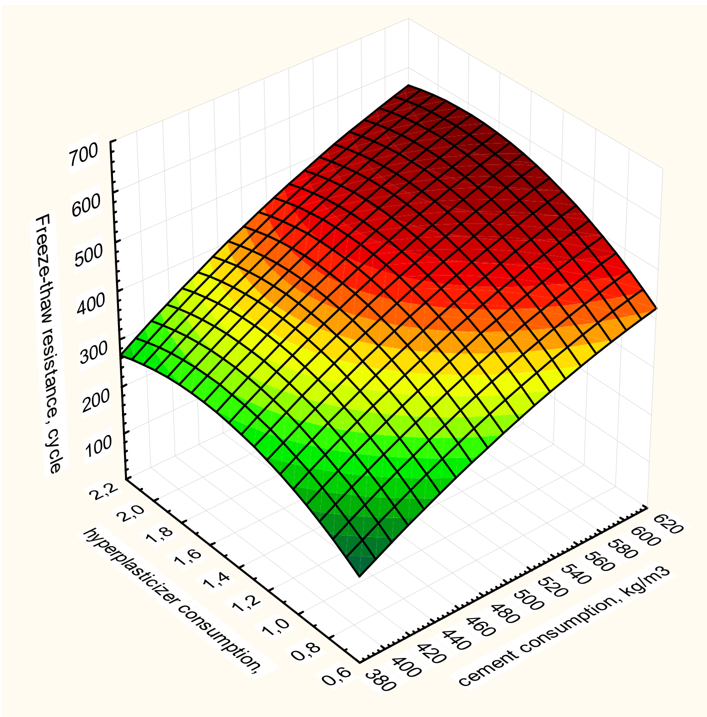

Figure 4 – Surface of cement consumption and additives on the concrete freeze-thaw resistance

Analysis of the sand replacement with ash slag degree and the hyperplasticizer consumption combined effect shows that with the amount of ash slag increasing in concrete, initially freeze-thaw resistance of concrete grows slightly, but with the further ash slag increase their freeze-thaw resistance is reduced. The maximum increase is observed at replacing half of the sand with ash slag, i.e. S-AS (sand – ash slag) = 0.5. At the maximum sand replacement with ash slag, the freeze-thaw resistance is reduced from 400 to 100 cycles.

Analysis of the surface of influence on the freeze-thaw resistance by the cement consumption and the degree of sand replacement with ash slabs shows that an increase in the concrete freeze-thaw resistance with an increase in the cement consumption is almost proportional. This fact is unmistakable, but it is known that the main component of concrete, contributing to the increased freeze-thaw resistance, is cement.

With the increase of ash slag in the composition of concrete freeze-thaw resistance tends to decrease. But on the surface of influence the maximum is observed at the value of AS-S (ash slag – sand) = 0.5, i.e. the ash slag only replaces sand by half. At the reduced sand replacement with AS in the concrete mixture, a sedimentation process was observed due to the increased mobility of the concrete mixture. Meanwhile, with the increased replacement, the freeze-thaw resistance also tends to decrease, because AS contribute to the increase of concrete porosity. As it is known, the freeze-thaw resistance of concrete depends on the porosity and the pores nature. It is obvious that concrete pores, which are formed due to the introduction of porous ash slab, are bigger than gel pores of cement stone. As it was explained above, gel pores of the cement stone contribute to the concrete freeze-thaw resistance increase, acting as compensators in water's turning into ice.

On the diagrams, we observe that with increasing cement, as expected, the freeze-thaw resistance grows, but not proportionally. The maximum value of the freeze-thaw resistance is observed with the cement consumption of 460 - 500 kg / m^3 at the plasticizer consumption of 1.2 – 1.4% of the cement mass. Thus, the optimum ratio should be considered when the cement consumption makes 460 - 500 kg / m^3, and that of plasticizer – 1.0 -1.4% of the cement mass.

Conclusions: Analyzing the data obtained as a result of the performed studies, we can state the following:

1. Studies confirm that the greatest impact on the freeze-thaw resistance is caused by the cement consumption. Although in the process of hardening, a well developed pore structure is formed in cement stone, yet it also forms gel pores where water at low temperatures – up to 30 – 40^{o}C – does not freeze, therefore, they compensate the stress that occurs during the water turning into ice and thereby contribute to the increase of the concrete freeze-thaw resistance.

2. Introduction of ashes into the concrete mixture leads to a decrease in the freeze-thaw resistance of concrete. This phenomenon is explained by the fact that the ash-slag has the water consumption three times higher than the sand they replace does, therefore they contribute to an increase in the *W/S* ratio. With the increase of *W/S* ratio grows both the total volume of open pores, and their mean size. At the same time, permeability and water absorption grow, and in such concretes less reserve pores are formed. In order to increase the concrete freeze-thaw resistance, it is customary to restrict *W/S* ratio depending on the concrete operation conditions. Reducing *W/S* is possible both due to reducing water consumption using plasticizer additives, and by increasing the cement consumption.

3. The influence of hyperplasticizer consumption on the concrete freeze-thaw resistance possesses a positive nature. It contributes to the reduction of the *W/S* ratio and thus contributes to the reserve pores formation. It should be noted that the *W/S* ratio in all the concrete mixtures within the experiment did not grow by more than 0.45. However, the mobility of the mixture fluctuated within the range of 5 to 13 cm of the cone settlement.

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ESSENTIALS AND PROBLEMS SOLVING ALGORITHMS OF GENERAL STRENGTH THEORY OF RC ELEMENTS UNDER COMPLEX STRESS-STRAIN STATES

The General strength theory of reinforced concrete elements (GSTRCE) passed the long and many-sided examination and showed the considerable advantages: well concordance with experiments, essential economic effect, solving method unity for different strength problems. Simple practical methods for calculating according to GSTRCE and available for use by designers and students are developed. The algorithms of calculating of two practically important problems are stated: strength control and selection of needed longitudinal and lateral reinforcement. Problems solving are represented by using «manual» method and well-known and easy-to-use software complex MS Excel. The results of experimental verification of GSTRCE are stated and the ones show well convergence of theoretical strength to experimental one.

Keywords: *element of structure, strength of inclined and normal sections, optimization calculation.*

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ОСНОВИ ТА АЛГОРИТМИ ВИРІШЕННЯ ЗАДАЧ ЗАГАЛЬНОЇ ТЕОРІЇ МІЦНОСТІ ЗАЛІЗОБЕТОННИХ ЕЛЕМЕНТІВ ПРИ СКЛАДНИХ НАПРУЖЕНО-ДЕФОРМОВАНИХ СТАНАХ

Загальна теорія міцності залізобетонних елементів (ЗТМЗБЕ) пройшла довгу та всебічну апробацію і показала значні переваги: добру збіжність з експериментами, істотний економічний ефект, єдиний метод розв'язання для різних проблем міцності. Розроблюються прості методи практичного розрахунку за ЗТМЗБЕ, доступні для використання проектувальниками і студентами. Викладаються алгоритми розрахунку двох практично важливих задач: перевірки міцності та підбору необхідної арматури. Розв'язання задач представлено «ручним» методом та з використанням широко відомого та простого у використанні програмного комплексу MS Excel. Наведені результати експериментальної перевірки ЗТМЗБЕ які показали добру збіжність теоретичної міцності з експериментальною.

Ключові слова: *елемент конструкції, міцність похилого та нормального перерізів, оптимізаційний розрахунок.*

Introduction. On the 4th International *fib* Congress [1] it was being observed the necessity of a general theory development for the strength calculation of reinforced concrete elements on the normal and inclined sections by unified conception basis and general relations.

Review of the latest research sources and publications. Such general theory is currently worked out [2, 3] and it has the important advantages in comparison with normative designs. The theory is based on the classification of reinforced concrete elements into groups depending on the quantity of longitudinal and transverse reinforcement (Fig. 1). This classification distinguishes the group C of elements which are not overreinforced by longitudinal and transverse reinforcement. The ones are differed by the most economical use of steel and *plastic failure on inclined section*. These elements usually have the broken off or bent up bars of longitudinal reinforcement. So *GSTRCE* is mainly limited by consideration of elements of group C. Herewith, entering some changes and additions in the design scheme of group C elements, the calculation methods of other groups can be obtained.

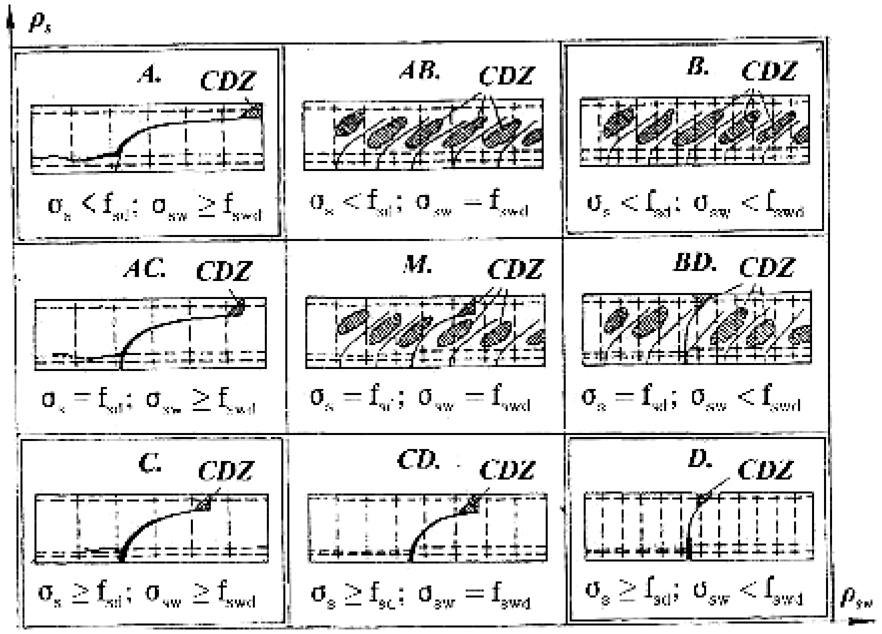

Figure 1 – RC elements classification: ρ_s, ρ_{sw} – respectively longitudinal tensile and lateral reinforcement ratio, σ_s, σ_{sw} – respective reinforcement stress, CDZ – concrete destruction zone.

Depending on the quantity of longitudinal and lateral reinforcement, respective behavior under load and type of failure, RC elements are classified [2] on the 4 main groups A, B, C, D and 5 intermediate groups AB, AC, BD, CD, M:

– group A – elements are overreinforced by longitudinal and underreinforced by lateral reinforcement; stress in longitudinal reinforcement (usually without broken off or bent up bars) does not reach and in lateral reinforcement reaches the limit value; elements are fractured brittly by dangerous inclined crack or by overreinforced normal section;

– group B – elements are overreinforced by longitudinal and lateral reinforcement; stresses in longitudinal and lateral reinforcement do not reach the limit value; brittle failure of elements by concrete compressive inclined strut between regular inclined cracks of element web or by normal overreinforced section is happened;

– group C – elements are underreinforced by longitudinal and lateral reinforcement; stresses in longitudinal reinforcement (usually with broken off or bent up bars) and in lateral reinforcement reach the limit value; plastic failure of elements by dangerous inclined crack or by normal underreinforced section takes place;

– group D – elements are underreinforced by longitudinal and overreinforced by lateral reinforcement; stress in longitudinal reinforcement (usually with broken off or bent up bars) reaches and in lateral reinforcement does not reach the limit value; plastic failure of elements by normal section, situated in action zone of shear forces that significantly influence on strength of normal sections, takes place.

Intermediate elements groups have properties which are transitional between the corresponding main groups.

Definition of unsolved aspects of the problem. The use of GSTRCE in practical designs is constrained by lack of enough simple practical calculations.

Problem statement. Thus, there is necessity of development of simple algorithms to perform calculations by this theory.

Basic material and results. Significant thesis of GSTRCE is made more exact definition of the reinforced concrete element as a part of reinforced concrete bar structure (RCS) on length l of which signs of bending moment M, shear V and longitudinal N forces are constant. This definition of element allows to unify its design scheme, which includes at one dangerous inclined (DIC) and normal (DNC) cracks, reinforcement and forces. The end points A and B of the DIC are considered as theoretical points of curtailment or bent-up of longitudinal reinforcement (Fig. 2).

Concrete strength criterion in zone of concrete failure at the end of DIC (Fig. 2) is determined as the strength condition of concrete wedge, situated over the DIC.

The interlock forces in DIC and the dowel action of reinforcement do not take into account for the group C elements because the element parts I and II (Fig. 2) as fragments of the plastic kinematic mechanism mutually rotate inducing the DIC opening without essential shear of its adjacent surfaces.

It is considered the proportional loading of RCS and its elements and the load parameter F is chosen, through which the forces in inclined section (crack) are expressed:

$$M = Ff_M\,,\ V = Ff_V\,,\ N = Ff_N\,, \tag{1}$$

where f_M, f_V, f_N – load functions, reflecting the element load character and depending on parameters x_A and c (Fig. 2).

Distribution of the normal σ_c and shear τ_c stresses along the concrete failure height x is adopted uniform.

The stresses σ_s, σ'_s σ_{sw} of the longitudinal tensile A_s, compressed A'_s and lateral A_{sw} reinforcement are restricted by the conditions of ultimate state respectively

$$\sigma_s \le f_{yd}\,,\quad \sigma'_s \le f'_{yd}\,,\quad \sigma_{sw} \le f_{ywd}\,, \tag{2}$$

where f_{yd}, f'_{yd}, f_{ywd} – design resistances of corresponding reinforcement.

Calculation of the inclined section strength in reinforced concrete elements by the GSTRCE is based on the design scheme of Figure 2 and on relationships:

– criterion of concrete strength above the DIC under shear force V_c and compressive force N_c action

$$V_c = Af_{cd}bx + a_N N_c\,, \tag{3}$$

where A, a_N – coefficients that are taken depending on the case of failure [2, 3];

f_{cd} – concrete strength under axial compression;

b – width of cross section of element;

x – height of concrete failure zone;

– equations of equilibrium of element part I (Fig. 2):

$$\sum X = 0;\; Ff_N + \sigma_s A_s - \sigma'_s A'_s - N_c = 0; \tag{4}$$

$$\sum Y = 0;\; Ff_V - \left(\sigma_{sw} A_{sw} c\right)/s - V_c = 0; \tag{5}$$

$$\sum M_0 = 0;\; Ff_M - \left(\sigma_{sw} A_{sw} c^2\right)/2s - \sigma'_s A'_s z_s - N_c\left(d - x/2\right) = 0\,, \tag{6}$$

where A_s, A'_s A_{sw} – area of cross-section of longitudinal tensile, longitudinal compressed and lateral reinforcement respectively;

σ_s, σ'_s σ_{sw} – stresses in longitudinal tensile, longitudinal compressed and lateral reinforcement;

s – step of lateral reinforcement bars;

z_s, d – are shown on Fig. 2.

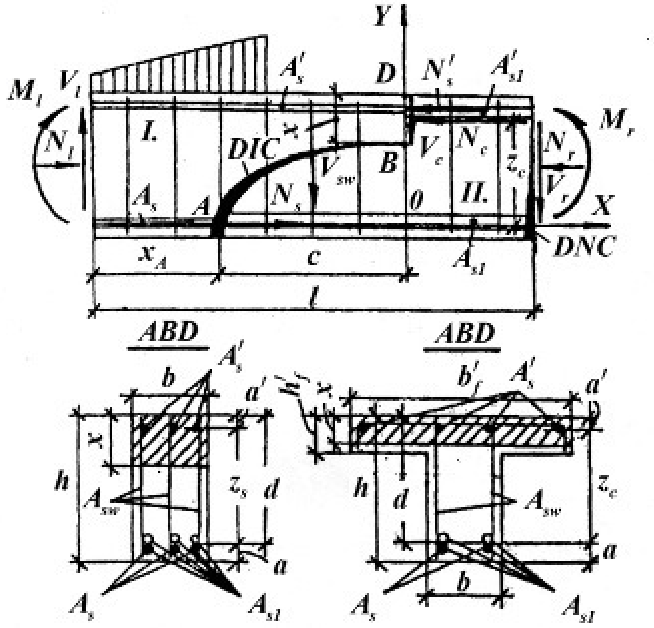

Figure 2 – Design model of reinforced concrete element with dangerous normal DNC and inclined DIC cracks, reinforcement and forces

From equations (3 – 6) the resulting relationship is obtained:

$$\begin{aligned} &P^2 f_N\left(f_V - f_N\alpha_N\right)/2A + P\left[f_M/d + \omega f_V - f_N\left(1 + m_{sw}\xi_{sw}c/2Ad - \eta\right)\right] - \\ &-\left(m_{sw}\xi_{sw}/2\right)\left(c^2/d^2 + 2\omega c/d\right) - B = 0, \end{aligned} \tag{7}$$

that is quadratic equation for the relative load parameter

$$P = F/f_{cd}bd\,. \tag{8}$$

In (7) f_M, f_V, f_N – load functions, expressing the forces M, V, N in element inclined section through the load parameter F,

$$\eta = m_s\xi_s - m'_s\xi'_s, \qquad \omega = \eta/2A, \qquad B = \eta\left(1 - \eta/2\right) + m'_s\xi'_s z_s/d\,, \tag{9}$$

$$\xi_s = f_{yd}A_s/f_{cd}bd\,, \qquad \xi'_s = f'_{yd}A'_s/f_{cd}bd\,, \qquad \xi_{sw} = f_{ywd}A_{sw}/f_{cd}bs\,, \tag{10}$$

$$m_s = \sigma_s/f_{yd}\,, \qquad m'_s = \sigma'_s/f'_{yd}\,, \qquad m_{sw} = \sigma_{sw}/f_{ywd}\,, \tag{11}$$

m_s, m'_s, m_{sw} – factors of resistance use completeness of respective reinforcement.

For determination of reinforcement cross-section area the formula of volume ratio of element reinforcement is used:

$$V_s = f_{cd}\xi_s X_A / f_{yd}l + f_{cd}\xi'_s (X_A + c) / f'_{yd}l + f_{cd}\xi_{sw} / f_{ywd} + \\ + f_{cd}\xi_{s1}(1 - X_A/l) / f_{yd} + f_{cd}\xi'_{s1}\left[1 - (X_A + c)/l\right] / f'_{yd}, \tag{12}$$

where

$$\xi_{s1} = f_{yd}A_{s1} / f_{cd}bd\,,\ \xi'_{s1} = f'_{yd}A'_{s1} / f_{cd}bd\,. \tag{13}$$

The problem of strength control is solved as optimization one with objective function of the relative load parameter P, depending on parameter c. In this problem set quantities are the next: element with all sizes, reinforcement, characters of materials (b, h, d, z_s, l, X_A, A_s, A_{s1}, A'_s, A'_{s1}, A_{sw}, s, f_{cd}, f_{ctd}, f_{yd}, f'_{yd}, f_{ywd}) , character and situation of loads. The ultimate load parameter F_u and respective value c_u are unknown.

Design algorithm of ultimate load parameter

1. The static calculation of structure is carried out.
2. The structure is divided on elements and each element design is fulfilled separately.
3. The length l of each element is determined; in element the location of DIC and DNC with input X_A, c, internal forces in the inclined section N_s, N'_s, V_{sw}, V_c, N_c are shown.
4. The load parameter F is selected and functions f_M, f_V, f_N are determined, taking into account the location and distribution of element loads.
5. The obtained functions f_M, f_V, f_N are substituted in resulting relationship (7) and the relation $P(c)$ is got.
6. The obtained function $P(c)$ is investigated on minimum by the conditions

$$m_s = m'_s = m_{sw} = 1\,, \tag{14}$$

corresponding to the group C elements, and the value c_u is determined.

The ultimate load parameter is calculated using the found value c_u.

If as result of calculation is $X_A + c > l$, the *widened element* is considered and points 2-6 are repeated.

7. The conditions of balance reinforcing are checked

$$A_s \le A_{s,opt}\,,\ A'_s \le A'_{s,opt}\,,\ A_{sw} \le A_{sw,opt}\,, \tag{15}$$

where the optimal values respective reinforcement $A_{s,opt}$, $A'_{s,opt}$, $A_{sw,opt}$ are adopted from the problem solution of the needed reinforcement determination by the load parameter that was found in the solution of previous strength control problem.

In practice of designing the problem of calculation the required longitudinal and lateral reinforcement in inclined section has the main importance.

The problem of needed reinforcement design is solved as optimization with objective function as volume ration of reinforcement in element V_s (12) together with additional conditions (7), (2) and $X_A + c \le l$. In this problem set quantities are the next: element with all sizes, reinforcement and characters of materials, the character of loads and numerical value of load parameter. From the previous calculation of normal (cross) section the A_{s1} and A'_{s1} values are known.

The analysis of this problem solving on the basis of Kuhn-Tucker conditions [4] shows that the optimal reinforcement is achieved at the highest possible stresses in reinforcement when the (14) is observed. It is appropriate to do the *incomplete optimization* of reinforcement. In this case the previously performed design of the normal (cross) section strength and the calculated A_{s1} and A'_{s1} values allow to adopt the areas $A_s < A_{s1}$ and $A'_s < A'_{s1}$ with the result that A_{sw}, X_A, c are only stayed as the unknown values.

Design algorithm of needed reinforcement

1. The static calculation of structure is carried out.

2. The structure is divided on elements and design of each element is fulfilled separately.

3. The length l of each element is determined; in element the location of DIC and DNC with input X_A, c, internal forces in the inclined section N_s, N'_s, V_{sw}, V_c, N_c are shown.

4. The load parameter is selected and functions f_M, f_V, f_N are determined, taking into account the location and distribution of element loads. The relative value of the load parameter (8) is calculated. Further the design for elements under cross bending without axial force N are given as example.

5. From function

$$\gamma_{18} = \frac{f_M}{d} + f_V \omega \tag{16}$$

the partial derivatives

$$\gamma_{14} = \frac{\partial \gamma_{18}}{\partial (c/d)}; \quad \gamma_{15} = \frac{\partial \gamma_{18}}{\partial (X_A/l)} \cdot \frac{d}{l}, \tag{17}$$

are determined.

6. The parameters (9), ξ_s, ξ'_s according to (10) and

$$D = \left[(\xi_{s1} - \xi_s) \frac{f_{ywd}}{f_{yd}} + (\xi'_{s1} - \xi'_s) \frac{f_{ywd}}{f'_{yd}} \right] \frac{d/l}{P}, \tag{18}$$

$$D_1 = \frac{(\xi'_{s1} - \xi'_s)}{P} \cdot \frac{f_{ywd}}{f'_{yd}} \cdot \frac{d}{l} \tag{19}$$

are calculated.

7. From the system of equations

$$\begin{cases} 0,5D\left(c^2/d^2 + 2\omega c/d\right) - \gamma_{15} = 0; & (20) \\ \gamma_{18} - B/P - 0,5D\left(c^2/d^2 + 2\omega c/d\right)\left[0,5D_1\left(c^2/d^2 + 2\omega c/d\right) + (\gamma_{14} - \gamma_{15})\right] \Big/ (c/d + \omega) = 0 & (21) \end{cases}$$

the values X_A/l, c/d are found.

8. The necessary intensity of lateral reinforcement is determined by the equation:

$$\xi_{sw} = P\left[0,5D_1\left(c^2/d^2 + 2\omega c/d\right) + \gamma_{14} - \gamma_{15}\right] \Big/ (c/d + \omega) \tag{22}$$

9. The A_{sw} and s values are calculated by the found ξ_{sw} (10).

10. The relative height of the concrete failure zone is determined by the formula:

$$\xi = \frac{x}{d} = \frac{Pf_V}{A} - \frac{\xi_{sw}}{A} \cdot \frac{c}{d} + \eta . \tag{23}$$

The condition $x > a'$ is checked, where a' is thickness of the protective concrete layer of compressed reinforcement A'_s.

The stated methods of problem solving can be called «manual» when the whole process of solution is considered and the unknown parameters are found in succession. This method is especially important in mastering of the GSTRCE. After GSTRCE mastering it is advisable to use «computer» method, when the optimization problems are solved by using software packages, such as processor MS Excel.

Experimental verification of the GSTRCE. The many-sided verification of the GSTRCE was conducted under V.P. Mitrofanov guidance. The strength criterion (3) was thoroughly experimentally verified by the specimens-wedges of usual heavy [5] and light-weight

[6] concretes. The significant control of (3) was fulfilled on the beams with artificial DIC, which allowed to exclude the interlock between crack sides and enabled to find reliably the experimental V_c and N_c quantities [5]. The noted all-round tests confirmed the realityof theoretical failure cases [5] of concrete wedges over the DIC and reliability of the relationship (3).

It is necessary to note that in all countries of the world the strength investigations of the reinforced concrete elements under the shear forces action were being conducted mostly on the elements of groups A and B (see Fig. 1) without curtailment of longitudinal reinforcement in zone action of shear forces. These tests led to the inexact notion that RC elements failure by the DIC has only brittle character. In our researches of the group C elements the longitudinal reinforcement was used in the form of two-bars bundles in which one bar had the break off in zone action of shear force and its end was welded to the being continued bar. The tested elements revealed the positive for practice features of the group C elements: clear-cut plastical behavior in ultimate state and the most economical expense of steel. Our investigation included the tests:

– 26 columns under joint action of M, V, N forces [7];

– 21 usual [5], 22 prestressed and 12 usual [8] simple beams;

– 22 simple and 13 cantilever beams with failure by the cross sections where the concrete failure zone was undergone the essential influence of shear force action [9];

– 4 simple beams with inclined compressive side [10];

– 6 two-span continuous beams [10];

– more 120 specimens-wedges [5, 6];

– 5 simple beams with shear span changing from 4,0 to 1,6.

All tested columns and beams were reinforced in accordance with the GSTRCE for the given load.

The test data of 131 beams and columns showed the mean ratio F^{test} / F^{calc} of experimental ultimate load parameter F^{test} to the theoretical one F^{calc} equal to 1,073 on the coefficient of variation $V = 9{,}525$ %. The comparison of the test data with design results by the Code SNiP 2.03.01-84* (Gosstroy, Moscow, 2000) led to the mean ratio $F^{test} / F^{calc} = 0{,}838$, $V = 23{,}06$ %. The like comparison with Eurocode 2-1992 produced the worst results. There are design examples according to the GSTRCE in [3].

Conclusions. The GSTRCE implementation into practice supposes perceptible economical effect at the expense of the complete use of all reinforcement strength and more exact optimization designs. The GSTRCE secures the simultaneous plastic element failure both on the cross and on the inclined sections (cracks), i.e. leads to the elements of equal resistance. The general conception basis for designs on the inclined and on the cross sections witnesses about more high development level of the GSTRCE in comparison with known designs systems. For the GSTRCE practical use it is offered two design methods: «manual» and «computering».

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MODELS OF BAY REINFORCED CONCRETE ELEMENTS RESISTANCE AT ACTION OF CYCLE PERMANENT SIGN HIGH LEVEL FORCES

The reinforced concrete span beam structures work with small, middle and large shear spans under the action of cyclic loads of high levels is investigated. It is established that researches of physical models development of bending reinforced concrete elements fatigue resistance to the cyclic action of transverse forces and calculation methods on its base are important and advisable due to following features of said load type: the nonlinearity of deformation, damage accumulation in the form of fatigue micro- and macrocracks, fatigue destruction of materials etc. The key expressions of the concrete endurance limits definition (objective strength), longitudinal reinforcement, anchoring of longitudinal reinforcement, which consists the endurance of whole construction are determined. Also the role and the features of influence of vibro-creep deformations on the change mechanics of stress-strain state of concrete and reinforcement of research elements are investigated.

***Keywords:** endurance, cyclic load, fatigue destruction.*

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МОДЕЛІ ОПОРУ ПРОГІННИХ ЗАЛІЗОБЕТОННИХ ЕЛЕМЕНТІВ ЗА ЦИКЛІЧНОЇ ЗНАКОПОСТІЙНОЇ ДІЇ ПОПЕРЕЧНИХ СИЛ ВИСОКИХ РІВНІВ

Досліджено робота залізобетонних прогінних балкових конструкцій з малим, середнім та великим прольотами зрізу в умовах дії циклічних навантажень високих рівнів. Установлено, що розроблення фізичних моделей втомного опору залізобетонних елементів, що згинаються, циклічній дії поперечних сил та методи розрахунку на їх основі є важливими та доцільними через такі особливості вказаного виду навантаження: нелінійність деформування, накопичення пошкоджень у вигляді втомних мікро- і макротріщин, втомне руйнування матеріалів тощо. Визначено ключові вирази знаходження межі витривалості (об'єктивної міцності) бетону, поздовжньої арматури, анкерування поздовжньої арматури, які й складають витривалість усієї конструкції в цілому. Досліджено роль та особливості впливу деформацій віброповзучості на механіку зміни напружено-деформованого стану бетону й арматури дослідних елементів.

***Ключові слова:** витривалість, циклічне навантаження, втомне руйнування.*

Introduction. In the current design standards, the calculation of endurance in bay reinforcement concrete structures is performing under the assumption of concrete elastic work. Calculation of sloping sections is performing under the assumption that main tension stresses, which appears on the level of transformed section centroid, should be fully carried by transverse reinforcement at stresses in it, which are equal to calculation resistance of transverse reinforcement f_{sw}, multiplied on condition load effect factor γ_{sw}, in elements without transverse reinforcement – by concrete at stresses in it, which are equal to concrete calculation tension resistance f_{ctd}, multiplied on appropriate condition load effect factor γ_c.

Such calculation approach contradicts to real work character of inelastic work of reinforced concrete elements and does not display reinforced concrete behavior features in the zone of transverse forces actions at cycling loads, does not display real stress-strain state, does not consider the ambiguity of transverse forces perception by different elements at different shear bays and character of fatigue destruction crack appearance and propagation, does not consider or consider mediated the influence of number of structural factors and factors of external action, which ultimately leads to significant differences between calculated and experimental data.

Analysis of last researches and publications sources. I.T. Mirsayapov, E.M. Babich, N.I. Karpenko [1 – 3] main attention is paid to the study of endurance and stress-strain state of normal sections of elements, which are bent, to the endurance of concrete and reinforcement and their deformability at second loads. During these researches there are accumulated a lot of experimental data, there is proposed a number of practical methods of normal section calculation in the zone of structures pure bending.

Despite the large number of experimental and theoretic researches of I.T. Mirsayapov, E.M. Babich, N.I. Karpenko, F. Aslani, W. Trapko [1 – 5] of reinforcement concrete elements resistance to the transverse forces action at static loads, the problem of reinforced concrete resistance to the second loads action remain is not studied well.

Specifying unsolved aspects of the problem. Theoretical researches of physical models development of bending reinforced concrete elements fatigue resistance to the series action of transverse forces and calculation methods on its base are almost missing. Therefore, today the development of physical models of fatigue resistance and destruction of near support parts of beams, which correctly shows their real work considering real element of concrete and reinforcement deforming at different shear spans and corresponding methods of their calculation has just been started.

Setting objectives. The main aim of the article is to study the performance of reinforced concrete structures by the means of creating these elements common models comprehensive resistance to the action of high levels cyclic loads with different shear spans.

The main material and results.

Reinforced concrete elements resistance models with small, middle and large shear spans.

Researches of I.T. Mirsayapov [1] and other investigators have shown, that at $c_0 / h_0 > 2$ fatigue destruction of near support parts of bending elements occurs with appearance of critical inclined crack, which location is connected not only with points of external force applying and support reaction, but with internal force factors, which occurs in shear span (moments and transverse forces). At $1{,}2 < c_0 / h_0 \leq 2$ destruction of near support parts of beam elements at cycle load has little similar signs of destruction of elements with small and large shear spans. In this case, conduct of appearing and extension of cracks and fatigue destruction in this zone at indicated load influences internal force factors and local concentrations of stresses in corresponding zones near points of external concentrated forces applying.

The feature of «long» bending reinforced concrete elements work at small shear spans ($a_0 < 1{,}2h_0$) is appearing of local stress strips, connected with points of concentrated forces applying within which occurs fatigue destruction. This feature of usual reinforced concrete beams with small shear spans joins them with «short» elements. In both cases this feature occurs at small values of relative distance between forces, applied to element.

B.S. Sokolov [6], T.I. Baranova [7], O.S. Zalesov [8], and others consider that for practical calculations of «short» elements the simplest solution of a problem is formation of calculating model as frame-rod system (FRS) which consists of inclined compressed strips and tensed bottom and compressed top reinforcing zones, which enclosure at points of concentrated forces and support reactions applying (fig. 1).

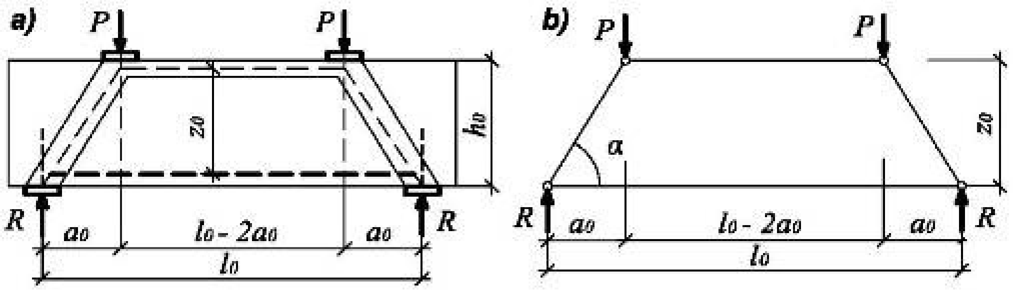

Figure 1 − Formation of force flows in usual («long») beam with small shear spans at repeated load (a) and its frame-rod analogue (b)

The principle of calculation model development is in determining of compression stresses in inclined force flows and tensile stresses in horizontal flow, intersection of which creates system, which can be conditionally called as frame-rod model of short elements. Main parameters determining calculation inclined strips are dimensions of load l_{sup}^{top} a nd support l_{loc}^{bot} areas, under which there are forming flows of compression stresses. The smaller is the sizes of areas, the higher is the trajectory density. So support and load areas forms incline strip and its width at top and at bottom. Angle of incline of main compression stresses flow approaches the angle of line incline which connects centers of support reaction and external concentrated force applying.

It is obvious that in process of near support area modeling of concrete element work at small shear spans using frame-rod analogue it sould be considered that its fatigue strength is determined by durability of every FRS element: inclined compressed strips and strength of tensed reinforcement. Fatigue destruction of tensed elements zone occurs as a result of fatigue rupture of longitudinal reinforcement at place of intersection with inclined crack or as a result of violation of reinforcement anchoring by inclined crack. Thus, occurring stresses should be limited by values of objective concrete and reinforcement strength at cycle load (durability limit) and its friction between themselves, i.e. for provision of such reinforced concrete elements durability it is necessary to follow the durability conditions:

$$\sigma_{1c}^{max}(t) \le f_{cd,rep}(t),\ \sigma_{s,\text{в}}^{max}(t) \le f_{ydq,rep}(t),\ \sigma_{s}^{max}(t) \le f_{yd,an}(t), \tag{1}$$

where $\sigma_{1c}^{max}(t)$ − compression stress on compressed force flow;

$\sigma_{s,\text{в}}^{max}(t)$ − actual tensile stresses in most loaded fibers of longitudinal reinforcement in the place of intersection with inclined crack;

$\sigma_{s}^{max}(t)$ − actual (maximum) axle tensile stresses in longitudinal reinforcement in the place of intersection with inclined crack;

$f_{cd,rep}(t)$ − limit of concrete durability at local compression;

$f_{ydp,rep}(t)$ − limit of reinforcement durability on tension;

$f_{yd,an}(t)$ − limit of longitudinal reinforcement anchoring durability.

Experimental researches [1, 6 – 8] have shown that stress-strain state inside inclined compression force flow is the same as in flat-stressed elements at local load. Thus, for evaluation of fatigue strength of inclined strip the model of fatigue destruction at compression and equations of objective (residual) strength of concrete and reinforced concrete at cycle load can be used. Wherein if axle «1» (fig. 2) is directed along longitudinal axle of inclined compression force flow and axle «2» in orthogonal direction and use the same designations as in elements with zero shear span, stress state inside inclined compressed force flow can be represented as fig. 2.

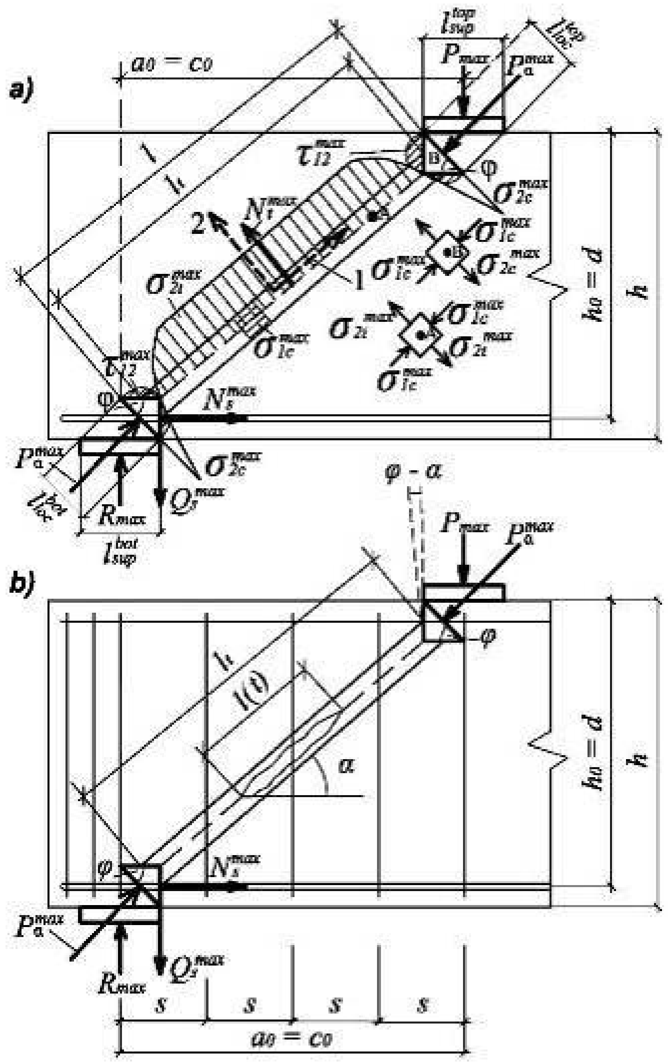

Figure 2 − Physic model (a) and calculation scheme (b) of bending reinforced concrete element resistance with small shear span at joint action of transverse force and bending moment

Since the vibro-creep deformations $\varepsilon_{1c,pl}$ extension in compressed concrete in the direction of stresses $\sigma_{1c}^{max}(t_0)$, action, as at local compression, occurs in free conditions and nothing interrupts its extension, it can be accepted that $\sigma_{1c}^{add}(t)=0$; $\sigma_{s}^{add}(t)\approx 0$; $\sigma_{1c}^{max}(t)=\sigma_{1c}^{max}(t_0)$; $\sigma_{s}^{max}(t)\approx\sigma_{s}^{max}(t_0)$, $\sigma_{1c}^{max}(t_0)$ and $\sigma_{s}^{max}(t_0)$ slightly simple determines at first load from the conditions of equilibrium on the base of fatigue resistance model.

Because stress-strain state inside inclined compressed strip and behavior of its fatigue destruction are analogical to stress-strain state and behavior of fatigue destruction of flat-stressed elements at local load action, expression for determining of objective fatigue strength (limit of durability) of inclined compressed stripe at the moment of time t by analogy and takes the form:

$$f_{cd,rep}(t)=\frac{\left(k_{scf}(t)+K_{isw}(t)cos\alpha\right)\cdot l_t ctg\varphi}{l_{sup}sin\alpha\sqrt{\pi l(t)}Y(l)}\times$$

$$\times\left\langle\frac{A_{нп}-\left\{G_c L_\varepsilon B_{нп}+\dfrac{6E_s I_s L_\varepsilon\cdot n\cdot cos(\varphi-\alpha)sin\alpha}{b\left(d_s\sqrt[4]{\dfrac{E_s}{E_c}}\left(1{,}4+1{,}25\sqrt[4]{\dfrac{a_s}{d_s}}\right)\right)^3 sin\varphi}\right\}\times}{\left\{\dfrac{1}{E_c}+C_e\prod_{k=1}^{k=n}K_k a\psi_\upsilon+\int_{t_0}^{t}\dfrac{\partial}{\partial\tau}\left[\dfrac{1}{E_c(\tau)}+C(t,\tau)\right]dt\right\}}\right\rangle^{-1}. \tag{2}$$

where $K_{isw}(t)$ − stress intensity factor which characterizes influence of transverse reinforcement on crack extension inside incline compressed flow;

$A_{нп}$ = 1, $B_{нп}$ = $1/sin^2\varphi$ − for reinforced concrete elements with load areas size l_{sup} / h < 0,2; $A_{нп} = cos^2\varphi$, $B_{нп} = ctg^2\varphi$ – for reinforced concrete elements with load areas sizes l_{sup} / $h \geq 0{,}2$; in elements without transverse reinforcement $K_{isw}(t) = 0$.

Multi-cycle fatigue of reinforcement is characterized by appear and extension of fatigue cracks in it. The formation of fatigue cracks occurs as a result of intensive plastic deformation of reinforcement steel in local volumes of stress concentration in reinforcement, main source of which is its periodic shape. It leads to significant closed hysteresis loops, which area is equal to energy, spent by one cycle of load. After plastic deformations exhausted, there microcracks appear in these local volumes, some of them can transform into large crack. At following enlargement of cycle numbers extension of main crack up to the critical size occur. Thus, for analytical description of fatigue destruction process and change of fatigue strength of steel reinforcement in reinforced concrete element at repeated loads there are used methods of destruction mechanics. The durability limit (objective strength of longitudinal reinforcement at the moment of time t at the place of its intersection with inclined crack in conditions of flat stressed state becomes:

$$f_{sd,в}(t)=\sigma_{sc}\cdot k_{scf}(t)/\sqrt{\left(Y(l)\cdot\sigma_{sc}\right)^2\cdot l_s(t)+k_{scf}^2(t)}\,, \tag{3}$$

$$\sigma_{sc}=\frac{\sigma_u}{\left\langle 1+exp\left(-2E_s\cdot\varepsilon_{pl}^{pec}/\sigma_{su}\right)\sqrt{1+3\left(\tau_{si}^{max}/\sigma_{sвв}^{max}\right)^2}\right\rangle}, \tag{4}$$

where $\sigma_{s\hat{a}i}^{max}$, τ_{si}^{max} – normal stresses in most loaded (tensioned) fibers and tangential stresses in longitudinal reinforcement in the place of its intersection with inclined crack;

$l_s(t)$ − length of fatigue crack in reinforcement at the moment of time t;

k_{scf} − critical factor of reinforcement stresses intensity at repeated loads at the moment of time t;

σ_{su} − temporary steel resistance to rupture;

ε_{pl}^{pec} − residual plastic resource of steel.

Process of multi-cycle fatigue anchoring of reinforcement is characterized by appearance and extension of fatigue cracks in contact zone between reinforcement and concrete. If the clutch stresses of reinforcement and concrete τ_g are high and these stresses are larger than limit of clutch durability, i.e. condition $\tau_g / \tau_{rep} > 1$ is true, generation and extension of through (inner) fatigue cracks occurs in contact zone between reinforcement and concrete. As it is shown in researches of B. Broms, I. Goto [9], N.I. Karpenko [3], M.M. Kholmyanskiy [10] through inner cracks cone-shaped volumes are formed. Indicated cracks permeate into concrete thickness, which crumples under these protrusions. Thus, objective fatigue strength of concrete under protrusions and forces of reinforcement protrusions clutch with concrete should be determined as function of cone-shaped crack length $l(t)$, which is permanently increasing with increasing of load cycle number. So for analytical characteristic of process of contact zone fatigue destruction and for change of longitudinal reinforcement anchoring fatigue strength at repeated loads it is also expediently to use destruction mechanic methods. Then the limit of durability (objective strength) of longitudinal reinforcement anchoring at the moment of time t is determined by:

$$
\begin{aligned}
& f_{ydan,rep}(t) = k_{scf}(t)ctg\varphi\left(\frac{1,5a}{cos\varphi_k} - \frac{c_r}{sin\varphi_k} sin\varphi cos\varphi\right) \times \\
& \times (d + 2c_r + (0,75a - 0,5c_r ctg\varphi_k sin\varphi cos\varphi)) \cdot (1,5 \cdot (1 + sin\alpha_r) - \sqrt{sin\alpha_r}) \times \\
& \times \frac{2\tau_g (d + 2c_r)(L + L_{pl})}{d^2} \cdot \left(\sqrt{\pi \cdot l(t,\tau)} \cdot Y(l) s_r (d + 2c_r) sin2\varphi_k sin\alpha_r\right)^{-1} \times \\
& \times \left(1,5 \cdot (1 + sin\alpha_r) - \sqrt{sin\alpha_r}\right) \cdot \frac{2\tau_g (d + 2c_r)(L + L_{pl})}{d^2} \times \\
& \times \left(\sqrt{\pi \cdot l(t,\tau)} \cdot Y(l) s_r (d + 2c_r) sin2\varphi_k sin\alpha_r\right)^{-1} \times \\
& \times \left\langle 1 - \frac{G_c(3atg\varphi_k - 2c_r sin\varphi cos\varphi}{c_r cos\varphi sin^2\varphi} \cdot \frac{A_{sh}}{A_c} \left\{ \frac{1}{E_c} + C_e \prod_{k=1}^{k=n} K_k a \psi_\upsilon + \int_{t_0}^{t} \frac{\partial}{\partial \tau} \left[\frac{1}{E_c(\tau)} + C(t,\tau) \right] dt \right\} \right\rangle^{-1}
\end{aligned}
\quad , \quad (5)
$$

where $\frac{A_{sh}}{A_c} = \frac{0,5cos\varphi}{(d + c_r)} \left\{ d + 2c_r + \frac{0,5c_r sin(\varphi - \varphi_k)}{sin\varphi_k cos\varphi} \right\}$;

d – rod diameter;

c_γ, s_r, a_r– accordingly height, step and angle of reinforcement protrusions incline;

a – concrete cover;

L, L_{pl} − the length of reinforcement fastening in concrete and plastic place of this fastening;

φ_k − angle of wedge under reinforcement protrusions;

$l(t,\tau)$ − length of fatigue crack in concrete under reinforcement protrusions at the moment of time t.

During cycle loading under the influence of high stresses of concrete crumpling under the reinforcement protrusions there are intensive deformations of vibro-creep. With enlargement of load cycles number N due to concrete vibro-creep under reinforcement protrusions, which surrounds them, increasing of displacement increment $g_0^{max}(t)$ on loaded end and inside fastening $g_x^{max}(t)$ occur, and it leads to redistribution of clutch forces $P_{i,r}$ from more loaded protrusions in the end of fastening to protrusions, that are situated in the depth of fastening, i.e. occurs redistribution of clutch stresses τ_g along fastening. Wherein enlargement of load cycle number leads continuous increasing of plastic area length and increasing of completeness of clutch stress diagram.

Analysis of a number of experimental data shows that fatigue strength and limit of durability of reinforced concrete bending elements in the zone of joint action of transverse forces and bending moments exceeds appropriate stresses (loads), where inclined cracks in tensed zone of element appear even at short time static load, i.e. bending reinforced concrete structures resists to repeated cycle loads at presence of normal and inclined cracks in near support areas. Concerning it at development of calculation model for evaluation of fatigue strength or such structures durability at the transverse force and bending moment action, it is necessary to consider existence of cracks in tensed zone, because appearance and extension of inclined cracks radically changes the quality of stress-strain state especially in elements with large shear spans.

The condition of crack appear in tensed zone of bending elements on appropriate trajectories is attainment by main tensile stresses the limit of concrete tensile strength at flat stress state «compression-tension» if cracks appears on first load or fatigue strength of concrete at flat stress state, if cracks appears after some number of load cycles, including its high levels.

In elements with large shear spans ($a_0 / h_0 > 2$) in the zone of joint action of transverse forces and bending moments, at first normal cracks appear, and then at optimal quantity of longitudinal work reinforcement (in not over reinforced structures) they are warped on near support areas by trajectories of main compression stresses and transform into inclined cracks. At increasing of cycle number one of such inclined cracks starts to expend more intensive and becomes critical. Trajectory of main compression stresses with appearance and extension of initial place of critical incline crack can be described by equation $y/h = m/(n + h/a)$, where m; n – are determined from boundary conditions. Appearance character analysis and expand of fatigue cracks, fatigue destruction of experimental beams, their stress-strain state in the zone of joint action of transverse forces and bending moments at repeated loads of high level and experimental thermograms [1] of near support areas of experimental elements allows to propose the following hypothesis of following expand of critical inclined crack and develop the physic model of fatigue destruction of bending reinforced concrete elements with large shear spans. Long before appearance of normal and inclined cracks in shear span especially before forming end expansion of critical inclined crack, in normal section at the end of shear span where maximum moment occurs, normal crack appears (section 1-1 on Fig. 3).

Until the remaining cracks appears in the zone of transverse force and bending moment action, the normal crack in the end of shear span extends on high height and tensed zone is practically fully off from work, the diagram $\sigma_x^{max}(t)$ is twisted, increases the completeness of thE diagram ω_σ and in top part of it starts to form plastic area; reduction of compressed part of concrete section height which has no cracks yet leads to sharp increase of completeness of diagram ω_τ of tangential stresses and to sharp increase of maximum value of tangential stresses $\tau_{xy}^{max}(t)$. Thus, inside the plastic area x_{pl} of compressed zone it is sharply increased resulting N_{R2}^{max} of normal $N_c^{max} = \int\limits_{A_{pl}} \sigma_x^{max}(t) \cdot dA$ and tangential $Q_c^{max} = V_c^{max} = \int\limits_{A_{pl}} \tau_{xy}^{max}(t) \cdot dA$ forces, where A_{pl} – area of plastic part of compressed zone in normal section with crack at the end of shear span. Influenced by force N_{R2}^{max} in compressed zone, which acts in limits of limited load area $x_{pl} / \cos\gamma$, in the direction of this force action there is incline compression force flow, inclined on angle γ to longitudinal axle of element. Pattern of stress distribution inside this inclined compressed force flow is the same as at local compression. At the cycle loading even before appearance of critical inclined crack inside inclined compressed force flow from micropores in concrete body or shrinkage microcraks on the action line of tensile stresses generates and extend fatigue separation microcracks, later they join into separation macrocrack *ed* at angle γ to longitudinal element axle.

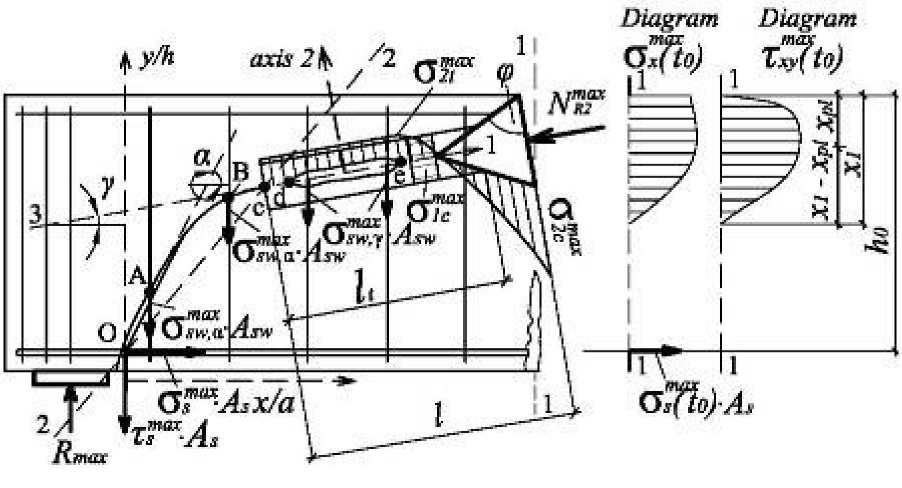

Figure 3 − Physic and calculation model of fatigue resistance of not over reinforced concrete element with large shear span

The most characteristic feature of normal separation cracks on near support parts of not over reinforced beams is tendency of any, even initially inclined to compression force action line, crack to align its trajectory in the direction of this force. Wherefrom it can accepted the hypothesis that from all inclined cracks which were formed on near support part from joint action of transverse force and bending moment in tensed zone at first load or at increasing of cycle numbers and load levels, critical becomes the inclined crack, which comes to zone of influence of inclined compressed force flow, generated by action of resulting N_{R2}^{max} of forces in compressed zone inside plastic area x_{pl}. It can confirm that the critical crack as a rule becomes extreme (closer to support) crack, which forms and expand along less loaded trajectory of main compression stresses and the following extension of critical inclined crack and more intensive its disclosure comparing to other inclined cracks and sharp increase of normal stresses in longitudinal reinforcement in the place of its intersection with critical inclined crack, i.e. alignment of longitudinal forces.

It is known that fatigue destruction of reinforced concrete element on inclined section becomes by compressed zone or as a result of fatigue rupture of most stressed rods of transverse reinforcement which intersects with initial area of critical inclined crack, or on tensioned zone as a result of fatigue rupture of longitudinal reinforcement in normal section 1-1 or because of anchoring violation of longitudinal reinforcement on and out of support.

So for assurance of operability of element at repeated load in is necessary to adhere to conditions:

$$\sigma_{1c}^{max}(t) \le f_{cd,rep}(t),\ \sigma_{sw,\alpha}^{max}(t) \le f_{ydw,rep}(t),\ \sigma_{s,в}^{max}(t) \le f_{ydq,rep}(t),\ \sigma_{s}^{max}(t) \le f_{ydan,rep}(t), \quad (6)$$

where $\sigma_{1c}^{max}(t)$ − actual main compression stresses in compressed zone over critical inclined crack in the direction of resulting of longitudinal and transverse forces action inside concrete inside plastic part of compressed zone;

$f_{cd,rep}(t)$ − durability limit (objective strength) of compressed zone over critical inclined crack at local compression in the direction of main compression stresses at the moment of time t;

$\sigma_{sw,\alpha}^{max}(t)$ − actual maximum stresses in the most loaded rods of transverse reinforcement at the moment of time t at the place of their intersection with initial part of critical inclined crack in tensed zone;

$f_{ydw,rep}(t)$ − durability limit of transverse reinforcement rods at their axle loading at the moment of time t;

$\sigma_s^{max}(t)$ − actual maximum axle stresses in longitudinal reinforcement at the moment of time t;

$\sigma_{s,в}^{max}(t)$ − actual maximum tension stresses in the most loaded fibers of longitudinal reinforcement at the place of intersection with inclined crack at the moment of time t;

$f_{ydq,rep}(t)$ − endurance limit of longitudinal reinforcement in conditions of a flat stressed state at the time t;

$f_{ydan,rep}(t)$ − durability limit of longitudinal reinforcement in conditions of flat stress state at the moment of time t.

As in elements with large shear span action of repeated load which leads to extension of vibro-creep deformations of compressed concrete in the direction of stresses σ_{x1}^{max}, σ_{1c}^{max}, action is accompanied with appearance and extension of additional (residual) stress-strain state on near support part of bended reinforced concrete element. With the aim of simplification of stress-strain state evaluation, action of repeated load and reinforced concrete element work it is expediently to divide into two stages. First stage shows stressed state of structure at first cycle (N = 1) of load to maximum cycle load P_{max}. Second stage is characterized by stressed state of element in the process of its repeated load (N > 1), which is directly continuous changed through intensive extension of vibro-creep deformations $\varepsilon_{1c,\ pl}$ of compressed concrete.

In general flow stresses in concrete and reinforcement and factors of cycle asymmetry becomes in form:

$$\sigma_i^{max}(t) = \sigma_i^{max}(t_0) \pm \sigma_i^{add}(t) \;; \tag{7}$$

$$\rho_i(t) = \left\langle \rho \cdot \sigma_i^{max}(t_0) + \sigma_i^{add}(t) \right\rangle / \left\langle \sigma_i^{max}(t_0) + \sigma_i^{add}(t) \right\rangle \;, \tag{8}$$

where $\rho = P_{min} / P_{max}$, $\sigma_i^{\max}(t_0)$ − initial stresses in concrete or reinforcement at first half- cycle of load;

$\sigma_i^{add}(t)$ − additional (residual) stresses in concrete or reinforcement, which appears as a result of concrete vibro-creep deformations accumulation.

Initial stresses at first load $\sigma_i^{max}(t_0)$ are determine from conditions of external and internal forces equilibrium on the base of model of fatigue resistance of element. Additional stresses $\sigma_i^{add}(t)$, which appears at process of its repeated load, starting from the second cycle of load are determined on the base of deformation relation for normal section (1-1) at the end of shear span and inclined section (2-2), which is placed on critical inclined crack (Fig. 3).

Fatigue destruction of compressed concrete zone over critical inclined crack occurs under the action of resulting N_{R2}^{max} of transverse and longitudinal forces which appears inside plastic part of normal section 1-1. Due to stress-strain state of compressed concrete zone over critical inclined crack (inside inclined compressed force flow) and behavior of fatigue destruction are analogical to stress-strain state and behavior of fatigue destruction in flat-stressed elements at the local load action, objective fatigue strength over critical inclined crack at the moment of time t we determine:

$$f_{cd,rep}(t)=\frac{\left(k_{scf}(t)+K_{isw}(t)\right)\cdot l_t cos\gamma\cdot ctg\varphi}{x_{pl}\sqrt{\pi\cdot l(t)}Y(l)}\times$$

$$\times\left\langle\frac{1-\left\{\frac{G_c L_\varepsilon}{sin^2\varphi}+\frac{6E_s I_s L_\varepsilon\cdot n\cdot cos(\varphi-\gamma)sin\gamma}{\left(d_s\sqrt[4]{\frac{E_s}{E_c}}\left(1{,}4+1{,}25\sqrt[4]{\frac{a_s}{d_s}}\right)\right)^3 sin\varphi}\right\}\times}{\left\{\frac{1}{E_c}+C_e\prod_{k=1}^{k=n}K_k a\psi_\upsilon+\int_{t_0}^{t}H_\sigma\frac{\partial}{\partial\tau}\left[\frac{1}{E_c(\tau)}+C(t,\tau)\right]dt\right\}}\right\rangle^{-1}. \qquad (9)$$

Durability limit of longitudinal reinforcement $f_{sd,\hat{a}}(t)$ at the place of its intersection with critical inclined crack in conditions of flat-stressed state is determining by (3) and (4). Durability limit of longitudinal reinforcement anchoring $f_{ydan,rep}(t)$ by critical inclined crack we determine by (5). Durability limit $f_{ydw,rep}(t)$ at axial load we determine by (3) and (4), taking at that, $\tau_{sw}^{\max}=0$.

Testing [1] reinforced concrete beams with rectangle cross section with shear span $a_0 = c_0 = (1{,}51 - 1{,}67)h_0$ allowed to specify the following picture of appearance and extension of cracks and character of fatigue destruction in the zone of transverse forces and bending moments action. Since elements with middle shear span 1,2 $h_0 < c_0 = a_0 < 2\ h_0$ are on borders of elements with small and large shear spans, in their operating and in mechanics of fatigue destruction at middle shear spans there are determined features of first and second, i.e. on the behavior of appearance and extension of cracks in the zone of transverse force and bending moment action and fatigue destruction of these elements has influence as internal force factors, as local fields of stress state and stresses concentration in corresponding zones in places of concentrated external forces applying. Thus, at middle shear spans fatigue destruction occurs with appearance of critical inclined crack, but local fields of stressed state and concentration of stresses in indicated zones have influence on destruction. Critical inclined crack can appear on the distance (0,2…0,3)h from the tensed edge and extends in support or concentrate external force direction. In tensed zone it disclosures along line 2-2 (fig. 4) which connects inner edge of supporting plate with external edge of load plate and fully intersects (to inner edge of support plate).

But in its extension from support to concentrated force critical inclined crack after approaching point *O,* i.e. intersection of lines 2-2 and 3-3, changes its direction and resume extension along line 3-3 by the axle of inclined compressed flow. At the same time inside compressed force flow on the line of tensile stresses σ_{2t}^{max} appears and extents rupture crack $d - e$ along axle 3-3, which afterwards merges initial part OO_2 of critical crack. It is obvious that appearance, extension and disclosure of critical crack in tensed zone (area OO_2) are connected with flat rotation and shear of inclined section 2-2, and its extension and disclosure in compressed zone (*ed*) are caused by appearance and extension of rupture microcracks on the line of tensile stresses σ_{2t}^{max} (Fig. 4) action in the zone «tension-compression» inside inclined compressed force flow, formed under the action of force P_β^{max}, and following merge into macrocrack with following extension and disclosing of this rupture macrocrack. Behaviour of stress distribution inside inclined compressed force flow is the same as at crumpling.

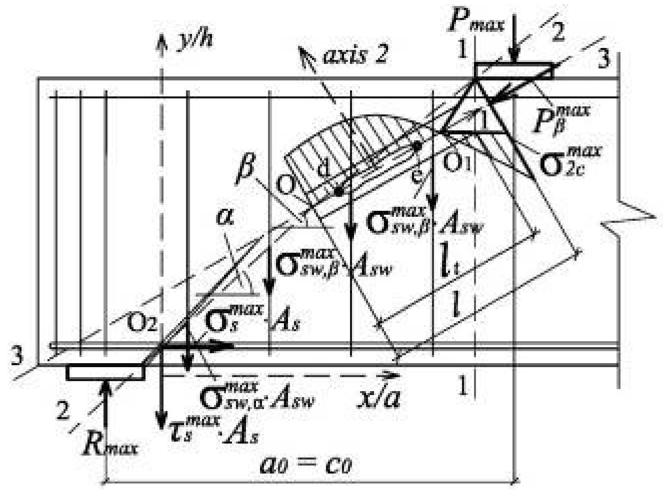

Figure 4 − Physic model and calculation scheme of fatigue resistance of inclined not over reinforced element with middle shear span

For this cause of stress-strain state and destruction character objective fatigue strength (durability limit) of inclined concrete strip over critical inclined crack becomes:

$$f_{cd,rep}(t)=\frac{\left(k_{scf}(t)+K_{isw}(t)\right)\cdot l_t\cdot ctg\varphi}{l_{sup}sin\beta\sqrt{\pi\cdot l(t)}Y(l)}\times\left\langle\frac{1-\left\{\frac{G_cL_\varepsilon}{sin^2\varphi}+\frac{6E_sI_sL_\varepsilon\cdot n\cdot cos(\varphi-\beta)sin\beta}{\left(d_s\sqrt[4]{\frac{E_s}{E_c}}\left(1{,}4+1{,}25\sqrt[4]{\frac{a_s}{d_s}}\right)\right)^3 sin\varphi}\right\}\times}{\left\{\frac{1}{E_c}+C_e\prod_{k=1}^{k=n}K_ka\psi_\upsilon+\int_{t_0}^{t}\frac{\partial}{\partial\tau}\left[\frac{1}{E_c(\tau)}+C(t,\tau)\right]dt\right\}}\right\rangle^{-1}. \quad (10)$$

Durability limits of transverse and longitudinal reinforcement and durability limit of its anchoring is determined by (3), (4) and (5).

Conclusions. Thus, the calculation of reinforced concrete structures at joint action of transverse forces and bending moments presence methods analysis, based on the researches [1 − 6, 11 − 13] shows that in most cases they performin assumption of elastic concrete work without considering its physic nonlinearity and change of deformation modes of materials in structures at cycle loading.

Considering physical models and calculation schemes of near support areas resistance of not over reinforced span reinforced concrete structures to repeated load of high level, there are envisaged different types of fatigue destruction of materials considering vibro-creep deformations, accumulation of damages in form of fatigue micro- and macrocracks.

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THE INFLUENCE OF OWN STRESSES ON TENSILE CONCRETE STRENGTH

Based on experiments results brittle duralumin samples, effect on stresses concrete (mortar) strength caused by non-uniform cross-section shrinkage (when dried) or expanding (when moistened) was inevitable. Various authors' experiments results diversity on concrete (mortar) tensile strength of moistening and drying effect were analyzed. Reasons for increasing air-dry concrete (mortar) strength storage at beginning of humidification were explained. Wetting sample duration (intensity) effect on concrete strength was analyzed in detail and opposite results causes on concrete strength moistening (drying) effects were obtained and justified.

Keywords: *own stresses, concrete strength, concrete water saturation, shrinkage, creep.*

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ВПЛИВ ВЛАСНИХ НАПРУЖЕНЬ НА МІЦНІСТЬ БЕТОНУ ПРИ РОЗТЯЗІ

Спираючись на результати дослідів над крихкими дюралюмінієвими зразками, доведено неминучість впливу на міцність бетону (розчину) напружень, викликаних нерівномірною по поперечному перерізу усадкою (при висушуванні) або набряканням (при зволоженні). Проаналізовано різноманітність результатів дослідів різних авторів із впливу зволоження і висушування бетону (розчину) на міцність при розтязі. Пояснено причини збільшення міцності бетону (розчину) повітряно-сухого зберігання на початку зволоження. Детально проаналізовано вплив тривалості (інтенсивності) зволоження зразка на міцність бетону та обґрунтовано причини отримання протилежних результатів дослідів впливу зволоження (висушування) на міцність бетону.

Ключові слова: *власні напруження, міцність бетону, водонасичення бетону, усадка, повзучість.*

Introduction. Own structural stresses influence on concrete strength, caused by long compression or tension continued, was investigated in many experiments. Unevenly distributed over cross-section of own stresses influence on concrete strength has not been studied, although it is advisable to consider factor that determines concrete strength.

Recent sources analysis of research and publications. To main factors influencing own stresses cross section occurrence and uneven distribution, except for concrete composition and conditions for its hardening and storage, concrete age [1], undoubtedly concrete samples water saturation and drying are inclued [2 – 4]. Many authors devoted their works to investigation of concrete samples moisture content influence on their compressive strength [5 – 8].

Analysis and justification of unevenly distributed intrinsic stresses influence on concrete strength under compression caused by shrinkage or expanding are given in [9].

Moisture influence results on concrete strength under tension, which do not have significant contradictions, are given in works [10 – 12]. It should also be noted that in all above papers, unevenly distributed influence analysis on proper stresses cross-section on concrete strength during tension was not carried out.

Among many factors that influence concrete strength under tension, it is rather difficult to isolate influence of unevenly distributed in own stresses cross-section. To prove effect of such own stresses on material strength, experimental studies were carried out on brittle (silicate) aluminum alloy. In cylindrical specimens, intrinsic stresses fields with opposite sign were created in comparison with own stresses fields caused by temperature distribution unevenness in cross-section during samples hardening under natural conditions [13].

Confirmation of own stresses influence on material strength is result of experiments carried out with own stresses duralumin samples higher strength with opposite sign to their own stresses that arise under normal hardening conditions. Basing on duralumin experiments results, influence of unevenly distributed intrinsic stresses on concrete strength under compression was proved [9].

Identification of general problem parts unsolved before. Own stresses unevenly distributed cross-section influence on concrete tensile strength was not investigated.

The research **goal** is to determe impact unevenly distributed over own section stresses on concrete strength in tension.

Basic material and results. Difference in own stresses influence (unevenly distributed cross-section) on concrete strength in compression and maturation is that at such stresses compression can both increase and decrease concrete strength (depending on nature of cross-section distribution), and when tensile, at any distribution of own stresses on cross-section, concrete strength will decrease.

It is no coincidence that various conclusions were drawn from experiments on moistening (or drying) effect on concrete strength when tensile. Shestoperov S.V. and Lyubymova T.Yu. studied moisture influence on tri-calcium aluminate mortar strength. Authors came to conclusion that when wet samples dry in atmosphere of saturated vapor or when immersed in water mortar tensile strength decreases, respectively, increase in moisture samples.

Tsiskreli G.D. [14] in experiments on moistening and drying influence on concrete strength received opposite results. Four samples groups were made for research, which two groups were stored in an aqueous medium, and two ones in air. Each pair was tested in storage by the first group. Water storage was dried before testing by the second group, and dry storage was saturated water by other group.

As tests result, average samples of water storage tensile strength during drying decreased from 2.33 MPa to 1.93 MPa. Average samples of dry storage with moisture tensile strength increased from 1.13 MPa to 1.38 MPa. Such opposite results can be explained by their own stresses samples state at test time.

In experiments Leshchinsky M.Yu. [2] samples were stored in moist environment for ninety days. During this period, moisture managed to penetrate sample center, and there was significant own stresses relaxation. Before test, samples were dried to achieve constant weight at t = 60°C. During this period, there were own stresses caused by intensive concrete shrinkage on samples surface: external layers were tensiled, and internal compressed.

When loading such samples, external layers will be more extended. Sample will destruct at lower external load than sample without its own stresses will do. When further drying, internal layers will decrease in size, which will reduce own stresses values, and as consequence, increase tensile strength. Such conclusions were confirmed by experiments of Leshchinsky M.Yu. (Fig. 1).

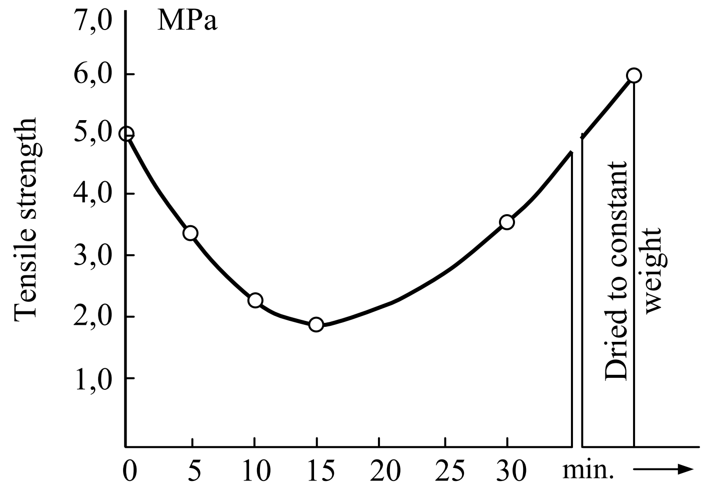

Figure 1 – Water-saturated samples prolonged drying influence on mortar eensile strength [3]

Moisture influence on concrete (mortar) strength experiments characteristics during tensile analusis, an unevenly distributed stress on concrete strength [3 – 5, 14] influence analysis were not conducted.

When water-saturation is created, own stress with opposite sign comparatively with stresses arises when drying concrete. Moistening sample correlates with external layers. External layers of moisture increase (expands) in size, tensioning internal layers.

Sample central part will be overloaded with tensile force and from it the destruction begins, when load of such a sample. Thus, sample will destruct at lower load comparatively with sample without having its own stress. Water enters sample central part when further moisture and thereby reduces its own stress and in end they can go down to zero.

Own stresses influence on mortar strength depends on stress state at wetting beginning. For example, Maltsov K.A. moisturized air-dry storage samples. In such samples, external layers are tensile (shrinkage), and internal layers are compressed. First torn external layers, when tested for breaking of such samples reduce entire sample strength.

When moisturizing such sample, external layers expend first reducing their own stresses, and as consequence, increasing strength. When moisture passes to sample depth, central fibers l swell, creating their own stresses in sample: internal fibers are compressed, and external tensiled reducing sample strength. Such conclusions are confirmed by Maltsov K.A. experimentally [3] (Fig. 2).

Practically the same influence character of unevenly distributed along stresses section on concrete strength is shown in experiments by Arkhypova A.M. [4]. Moisture uneven distribution influence possibility on cross section of sample on concrete (mortar) tensile strength is also considered. But such impact analysis is not provided (Fig. 3).

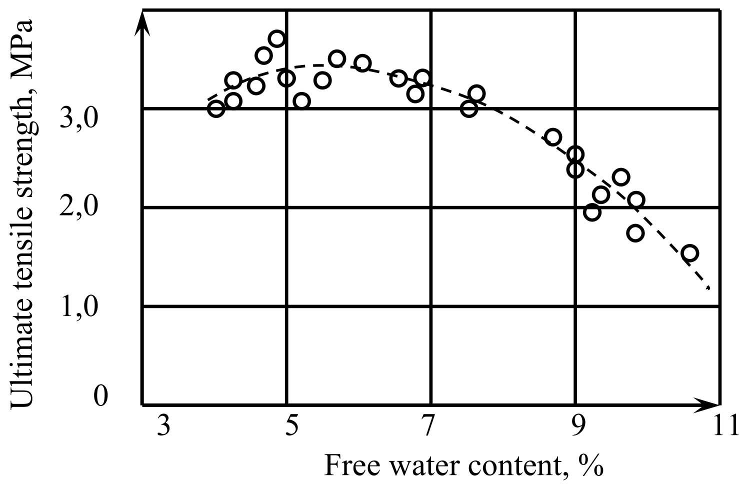

Figure 2 – Mortar strength -moisture dependence

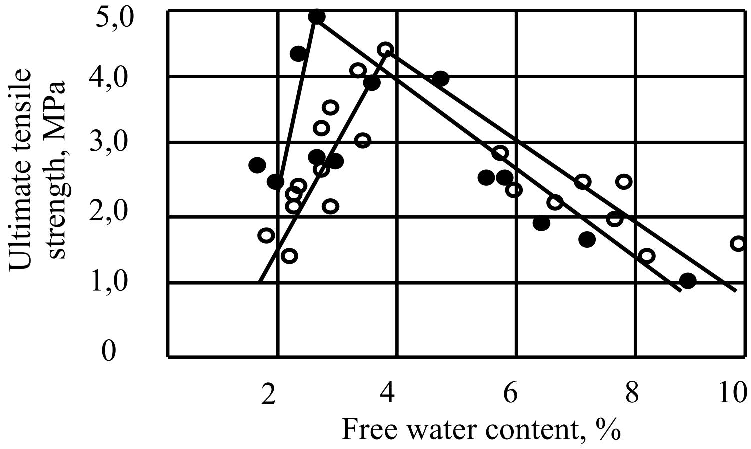

Figure 3 – Water saturation influence on cement mortar tensile strength

Similar results were obtained by Moskvin V.M. and Basement A.M. in water influence study saturation on concrete strength. Air-dry storage concrete disks were tested, which were humidified during test up to 6% [14] (Fig. 4).

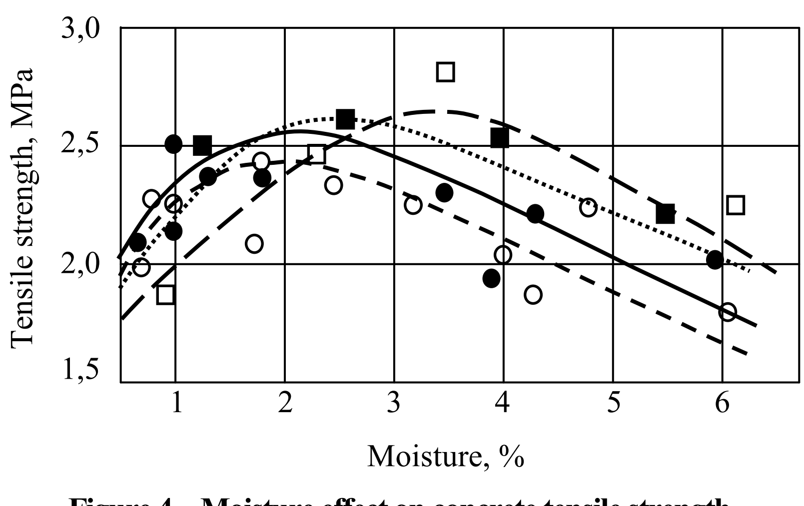

Figure 4 – Moisture effect on concrete tensile strength

Some researchers believe that changes in concrete strength during wetting (expanding) and drying (shrinkage) are affected unevenly distributed by deformation section, but such influence mechanism analysis is not given either in bending tests [14] or in compression [7, 10].

Mileykovska K.M. in its experiments, strength reduction during concrete moistening binds to concrete deformation, but coupling deformations mechanism with water saturation with decrease in strength is not clear. It was concluded that, with expanding termination, decline in concrete strength stops, and with sufficiently long water saturation, strength is completely restored, which can be explained by their own stresses relaxation.

Experiments results disadvantage on their own stresses influence on concrete strength (mortar) is that tests were carried out in short time. During experiment, as a rule, there is not enough time to manifest either own stresses concrete creep or relaxation, and they affect concrete (mortar) strength.

Similar experiments were carried out on cement mortar samples. Samples were dried at 105-110 ° for constant weight and stored in desiccator after making and steaming. For each test stage, samples were saturated with different fluids (water, ethanol, benzol, calcium chloride water solution) for several days. It should be noted that at tests beginning there was in samples practically absent (or brought to minimum value) their own stresses, unevenly distributed across cross-section.

When moisturizing such samples, external layers swell first. As external layers expand internal layer tension expend. When loading such sample with tensile force, internal layers will be overloaded and destroyed first, therefore, sample l withstands less load compared to sample without its own stress. Further, fluid penetration, for example, leads to increase in size (swelling) of sample central part, which leads to decrease in difference in sizes of external and internal layers, and as consequence to reduce their own stresses. As own stresses reduction result, samples l withstands greater external tensile force.

It should also be noted that at the beginning sample wetting, during swelling external layers caused creep when internal layers are tensed. External layers were reduced in size from creep to compression. As result, with further expanding (mainly internal part), there are own stresses: sample internal part is compressed and outer is stretched and sample strength when tensed after reaching maximum value will decrease gradually. Such conclusions were confirmed experimentally [14] (Fig. 5).

Sown in Fig. 5 dependencies confirms that moisture (drying) experiments on effect results on concrete (mortar) strength depend on state in which stress state is sample in test. So, when testing liquid from saturation effect dry state to moisture content 2%, the mortar strength will decrease, and when tested with moisture content of 4% to 6% strength will increase due to moisture.

When testing samples saturated with liquid more than 14%, mortar strength will decrease (Figure 5). Such results explain conclusions variety obtained in water saturation (expanding) and drying (shrinkage) effect study on concrete (mortar) strength.

As experiment showed, liquid chemical composition did not affect change character (sequence) in mortar strength with increasing saturation. Cement prior chemical process involvement to creation of their own stresses when wetting and drying samples is proved by experiments results carried out over tuff [14]. Tuff samples strength dried to constant weight 24.8 MPa, samples stored in air-dry medium had strength 9.8 MPa, and when samples were saturated with water, strength was reduced to 5.6 MPa. Samples were pre-saturated with water, and then completely dried (to constant weight) showed strength 24.7 MPa. Thus, reduction in strength as saturation result with water was completely reversible. Similar picture of decrease in strength under water saturation is observed in artificial stone materials, for example, in brick.

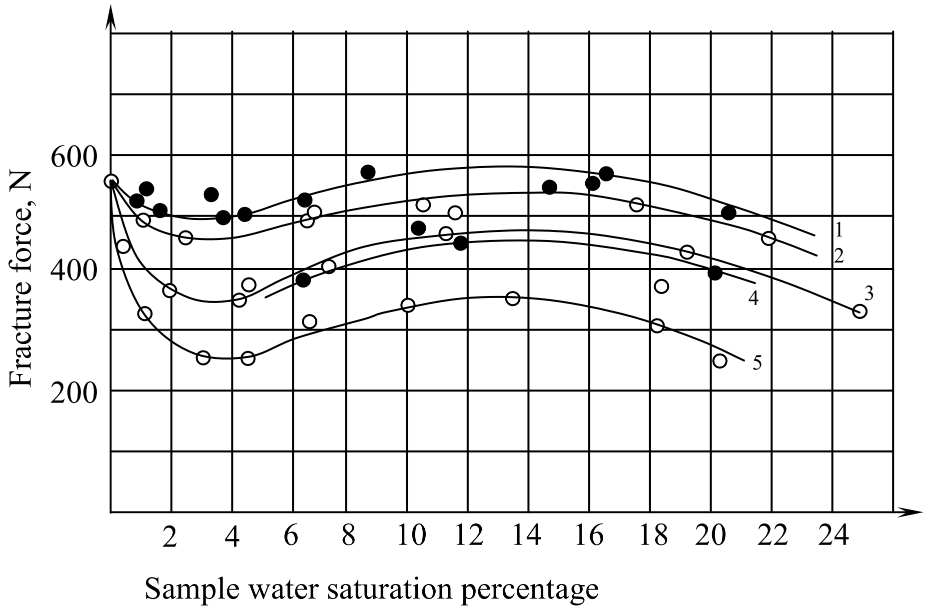

Figure 5 – Mortar strength depends on water saturation different fluids
1 – kerosene and benzol; 2 – ethanol; 3– water;
4 – saturated calcium chloride water solution;
5 – 0,5% calcium chloride water solution

Such conclusions are confirmed on cores with moisture 8%, taken from hydrosystem at depth 40 m under water. With moisture increase 1.5 – 2.5% (due to uneven moisture in cross section), core strength decreased. Cores further storage in air-dry environment has led to increase in cores strength [14].

Both experimental and theoretical additional studies are required in own stresses influence issue on concrete strength.

It should also be noted that shrinkage and expending effect described interpretation on concrete strength with tension does not claim comprehensive explanation of this issue and has in addition adsorption effect and compression by capillary pressure forces.

Conclusions. Analysis of unevenly distributed stresses in concrete (caused by shrinkage or expending), which is unevenly distributed over cross section, confirms and sufficiently substantiates their influence on concrete tensile strength.

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STRENGTH OF HIGHER STRENGTH CONCRETE ELEMENTS UNDER SHEAR ACTION

The strength design method of concrete and reinforced concrete elements is expounded in this article. The experimental program included the study of the strain condition and failure load determination for considered types of elements. The strength design method is expounded for concrete and reinforced concrete elements by means of variation method in the concrete plasticity theory that was developed in Poltava National Technical Yuri Kondratyuk University. There are the results of experimental investigation for truncated concrete wedges that simulate work of concrete compressed zone above dangerous inclined crack, Hvozdev specimens and crucial keys as well as beams. Also all elements were made of higher strength concrete in order to test the applicability of given method to these elements. The results of the experimental research have confirmed the applicability of plasticity zones assumed in the theoretical solutions. The theoretical strength is well coordinated with the experimental one. The failure character of reinforced concrete beams has been discovered. It has not differed from the flexure elements failure by cross section made of conventional concrete.

***Keywords:** shear action, variation method, truncated wedge, Hvozdev specimen, key, beam, higher strength concrete.*

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МІЦНІСТЬ ЕЛЕМЕНТІВ ІЗ БЕТОНУ ПІДВИЩЕНОЇ МІЦНОСТІ ПРИ ЗРІЗІ

Викладено методику розрахунку міцності бетонних і залізобетонних елементів, котрі перебувають під дією зрізувальних сил, а також програму експериментів, яка містить дослідження деформованого стану та визначення граничного навантаження зазначених типів елементів. Застосовано методику розрахунку, котра базується на варіаційному методі у теорії пластичності бетону, розробленому в ПолтНТУ. Отримано результати експериментального дослідження зрізаних бетонних клинів, які моделюють роботу стиснутої зони бетону над небезпечною похилою тріщиною, зразків Гвоздєва та залізобетонних шпонок, а також залізобетонних балок на дію поперечної сили. Дослідні зразки було виготовлено з бетону підвищеної міцності для з'ясування можливості застосування запропонованого методу до розрахунку їх міцності. Виконано порівняльний розрахунок теоретичної міцності з дослідною, котрий показав їх достатню близькість. Виявлено характер руйнування досліджуваних балок, який принципово не відрізняється від руйнування згинальних елементів за похилим перерізом із бетонів середньої міцності.

***Ключові слова:** зрізувальні сили, варіаційний метод, зрізаний клин, зразок Гвоздєва, шпонка, балка, бетон підвищеної міцності.*

Introduction. Concrete and reinforced concrete elements under the action of shear forces are important and widely used in the practice of construction. The mentioned elements include the beams and slabs, short elements, monolithic massive constructions. They all differ from each other in the construction determination, dimensions, forms and the character of stress-strain state in the shear plane.

Traditional heavy concrete has being replaced by multi-component modified concrete that differs by high strength and corrosion durability, water resistance and freeze-thaw durability. At the same time it characterizes by higher brittle behavior that makes condition for its homogeneous structure.

Analysis of the latest researches and publications. Data concerning main characteristics of high strength concrete made by traditional technology using high mark cement and precise selection of components are given in works of O. Ya. Berg [1], O. Ye. Desov [2], V. I. Sytnyk [3] and others.

Main concepts of high strength concrete development based on high-range water-reducers are observed by V. H. Batrakov [4], S. S. Kaprielov [5], A. V. Korsun [6] and others.

Experimental investigation of deformation and strength characteristics of high strength concrete in Ukraine almost did not go beyond testing small specimens (mainly prisms). Therefore, a comprehensive analysis of the characteristics of high-strength concrete can be done only after testing full-scale structural elements and nodes of their connection. Such investigations are famous abroad [7–11], as well as were carried out in PoltNTU [12–14].

Emphasis of not determined earlier parts of general requirements. Usage of design method which is based on concrete plasticity theory for elements with higher embrittlement raises some questions.

The action of DBN B.2.6-98:2009 is being distributed on the traditional concrete and the development of the normative document «Concrete and reinforced concrete constructions of high-strength concrete (concrete class for compression higher than C 50/60)» is being only envisaged. So, there is a problem of strength design of high-strength concrete elements.

Variation method in the concrete plasticity theory for concrete and reinforced concrete elements was developed in Poltava National Technical Yuri Kondratyuk University [15]. It can be referred to the engineer design methods that bring to really easy relations, don’t need an involvement of the complex computer programs and found wide distribution in the design practice. The method is widely tested by strength design of reinforced concrete constructions under shear action using heavy and lightweight concrete [16].

Objective of the work is the attempt to expand the variation method on the strength design of high strength concrete elements by shear.

Summary of main information. Two famous technologies of getting high strength concrete were used by author during conducting experimental investigation. Traditional one is based on high mark cement and precise selection of components. The second one is based on complex application of high-range water-reducer and silica fume.

Following elements were considered as the experimental models: truncated concrete wedges that simulate work of concrete compressed zone above dangerous inclined crack, Hvozdev specimens and crucial keys as well as beams.

Truncated concrete wedges were made of concrete with addition of silica fume and high-range water-reducer [17]. Two series of specimens were tested. The first one consisted of 5 wedges where angle α was constant and equaled to 30°. Angle β with constant α varied in next sequence: -20° (V_c directed to square corner), 10°, 0°, 5°, 20°. The second series included 3 specimens with following parameters: with angle $\alpha = 45°$ angle β was 20° and 30°, and with $\alpha = 15°– \beta = 10°$. The height of compressed zone was $h_w \approx 50$ mm for all specimens and the wedge thickness was $b_w \approx 150$ mm. Low parts of specimens were reinforced for failure prevention.

The concrete deformations of wedges were measured by strain gages that were placed in some distance from the wedge top (fig. 1).

a b

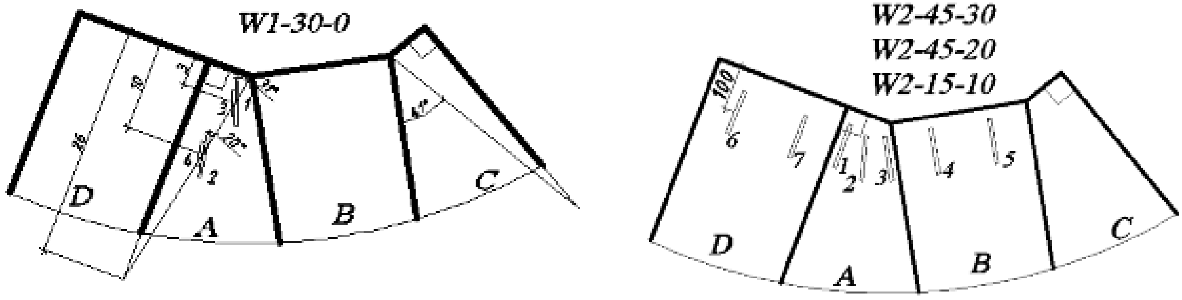
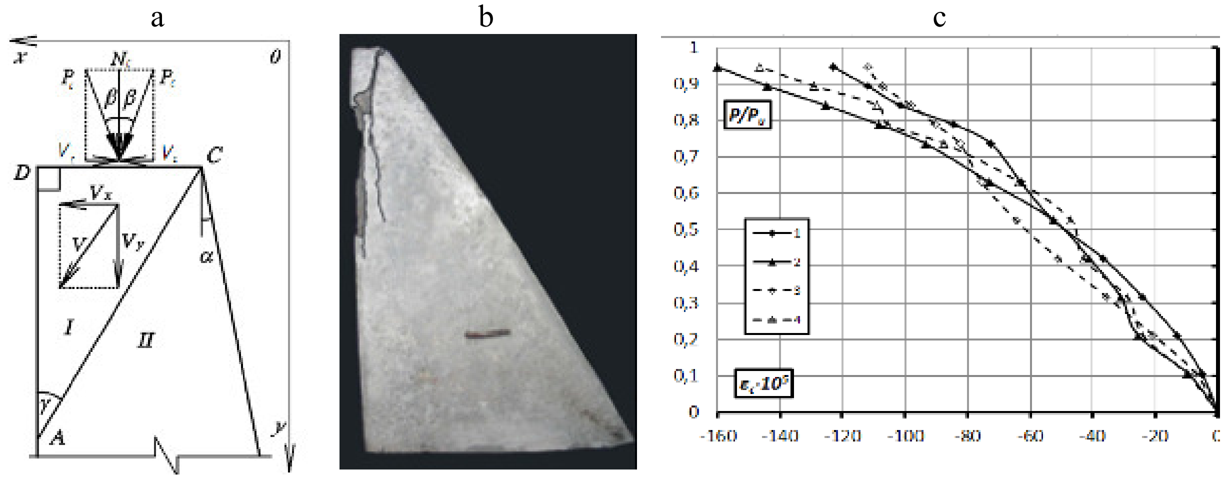


Figure 1 – Placing scheme of tensoresisterson truncated concrete wedges

There were two forms of wedge failure. The first one characterized by the failure area that crossed the top of obtuse angle and facet of the right angle (fig. 2, b). While applying force V_c to the right angle with increasing load angle β wedge strength is decreasing, and it is increasing with force V_c from the right angle.

a b c



Given: h_c=DC, b_w, α, β, f_{cd}, f_{ctd}

Figure 2 – Kinematically possible scheme (a), the failure character (b)
and diagram «ε_c – P/P_u» for wedges that collapsed according to the first case (W1-30-0)

Formula for the ultimate load determination of such wedges is used:

$$P_c = m\left[2B\sqrt{(k - tg\gamma)^2 + 0{,}25(ktg\gamma + 1)^2} - (k - tg\gamma)\right]\times\frac{h_w b_w}{tg\gamma\cos\beta(1 \pm k_0 k)}, \qquad (1)$$

where $m = f_{cd} - f_{ctd}$, $B = \sqrt{(1 + \chi/(1-\chi)^2)/3}$,

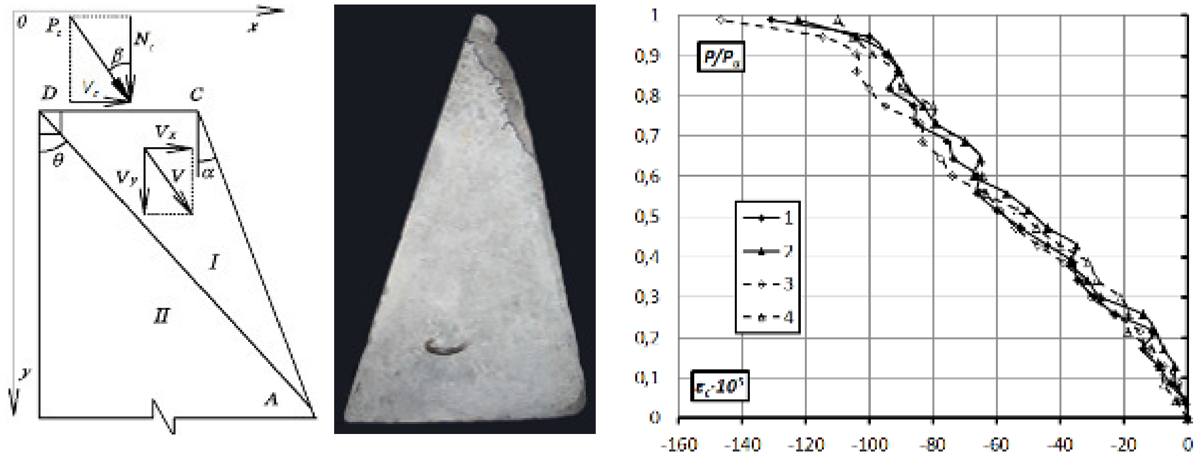
$\chi = f_{ctd}/f_{cd}$, $k_0 = V_c/N_c$,

$P_c = N_c/\cos\beta$;

unknown parameters are P_c, $k = V_x/V_y$ and the angleγ (fig. 2, a).

The shear plane crossed the top of right angle and the facet of obtuse angle when the second failure form occurred (fig. 3, b). The failure mode corresponded to the accepted kinematical scheme (fig. 3, a).

a b c



Given: $h_c, b_w, \alpha, \beta, f_{cd}, f_{ctd}$

Figure 3 – Kinematically possible scheme (a), the failure character (b) and diagram «ε_c – P/P_u» for wedges that collapsed according to the second case (W1-30-20)

In this case the strength has being proposed to evaluate as:

$$P_c = m\left[2B\sqrt{(k - tg\theta)^2 + 0{,}25(ktg\theta + 1)^2} - (k - tg\theta)\right] \times \frac{h_w b_w}{\cos\beta(tg\theta - tg\alpha)(1 + k_0 k)}, \qquad (2)$$

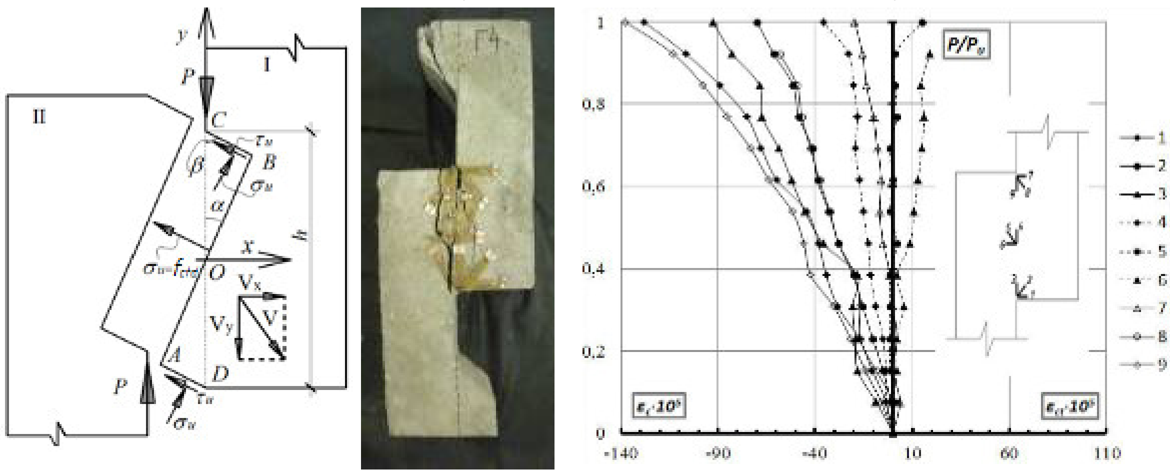
here unknown variables are P_c, $k=V_x/V_y$, angleθ (fig. 3, a).

Despite the seemingly brittle failure character of wedges the substantial compression deformations were fixed on the failure plane. They are close to εcu values on the diagrams of concrete mechanical state. Diagrams «ε_c – P/P_u» became warped that testified about the local plasticity zones existance (fig. 2,c and 3, c).

Gvozdev specimens (4 twins) were made of the same batch with wedges.

The specimens failure took place by the surface that crossed the shear plane and was almost congruent with it (fig. 4, b). The strain gages (fig. 4, c) indicated the presence of compressed and tensile sections of failure surface: in the middle part of design section tension was fixed, whereas compression has been found near inlet corners.

a b c



Given: h, b, f_{cd}, f_{ctd}

Figure 4 – Kinematically possible scheme (a), the failure character (b) and the diagram of relative concrete deformations from the load level «ε_c – P/P_u» (c) forGvozdev specimens

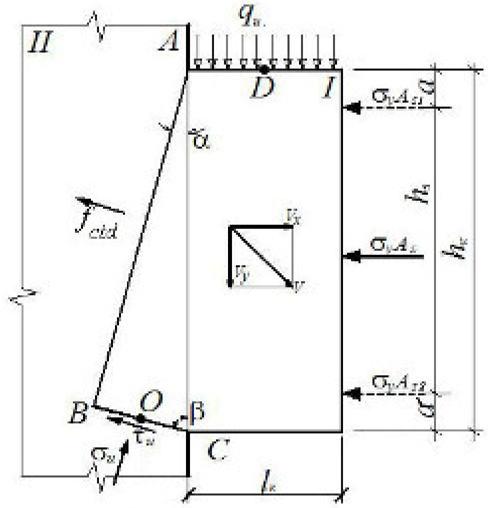
The Gvozdev specimens strength has been calculated as:

$$P = m\left[2B\sqrt{(k - tg\beta)^2 + 0{,}25(ktg\beta + 1)^2} - (k - tg\beta)\right] \times \frac{hbtg\alpha}{tg\beta + tg\alpha} + \frac{hbf_{ctd}tg\beta(k + tg\alpha)}{tg\beta + tg\alpha}, \quad (3)$$

where unknown variables are P, $k = V_x / V_y$ and angles α and β (fig. 4, a).

The limitation ΣX=0 was used by the task solution.

Crucial keys had the thickness b_k = 150 mm and height h_k = 200 mm with correlation of their dimensions l_k/h_k = 0,25 that provided non-shear failure with maximum strength provision. Bar reinforcement of reinforced concrete keys has been selected so that two bars located in the middle height of key corresponded to the four bars located in the upper and lower parts of the key (for two twin specimens). One sample was concrete.



Given:

$h_k, b_k, l_k, A_s, \sigma_y, f_{cd}, f_{ctd}$

Find q_u

Figure 5 – Kinematically possible scheme of reinforced concrete keys failure

For making the specimens the concrete of higher strength was used with cement M700 mark addition [18].

The failure character of keys principally didn't differ with different reinforcement distribution by the section height (fig. 6, a, c). Spacing the reinforcement in two levels led to the increasing of concrete compression deformations up to 50% (fig. 6, b, d). Besides the reinforcement of the upper level begins to work earlier comparing to its central placing. In the reinforcement of the lower level the dowel effect has being observed [19]. The keys strength with dual reinforcement is up to 10% bigger comparing with single.

The ultimate load of keys has being proposed to calculate by the formula that answered to the design scheme on the fig. 5:

$$q_u = (m\left[2B\sqrt{(k - tg\beta)^2 + 0{,}25(ktg\beta + 1)^2} - (k - tg\beta)\right] \times \frac{tg\alpha}{tg\alpha + tg\beta} + f_{ctd}(k + tg\alpha) \times$$

$$\times \frac{tg\beta}{tg\alpha + tg\beta} + \frac{\sigma_y (A_{s1} + A_{s2})k}{b_k h_k})\frac{1}{\gamma}, \quad (4)$$

where $\gamma = \dfrac{l_k}{h_k}$, and the unknown parameters are q_u, $k = V_x / V_y$, angles α, β. Calculating of q_u is made within the realization of limitations: $\sum M_B = 0$, $\sum M_O = 0$, $\sum M_D = 0$.

In the case of one-level reinforcement placing the space of reinforcement in formula (4) equals $A_s = A_{s1} + A_{s2}$.

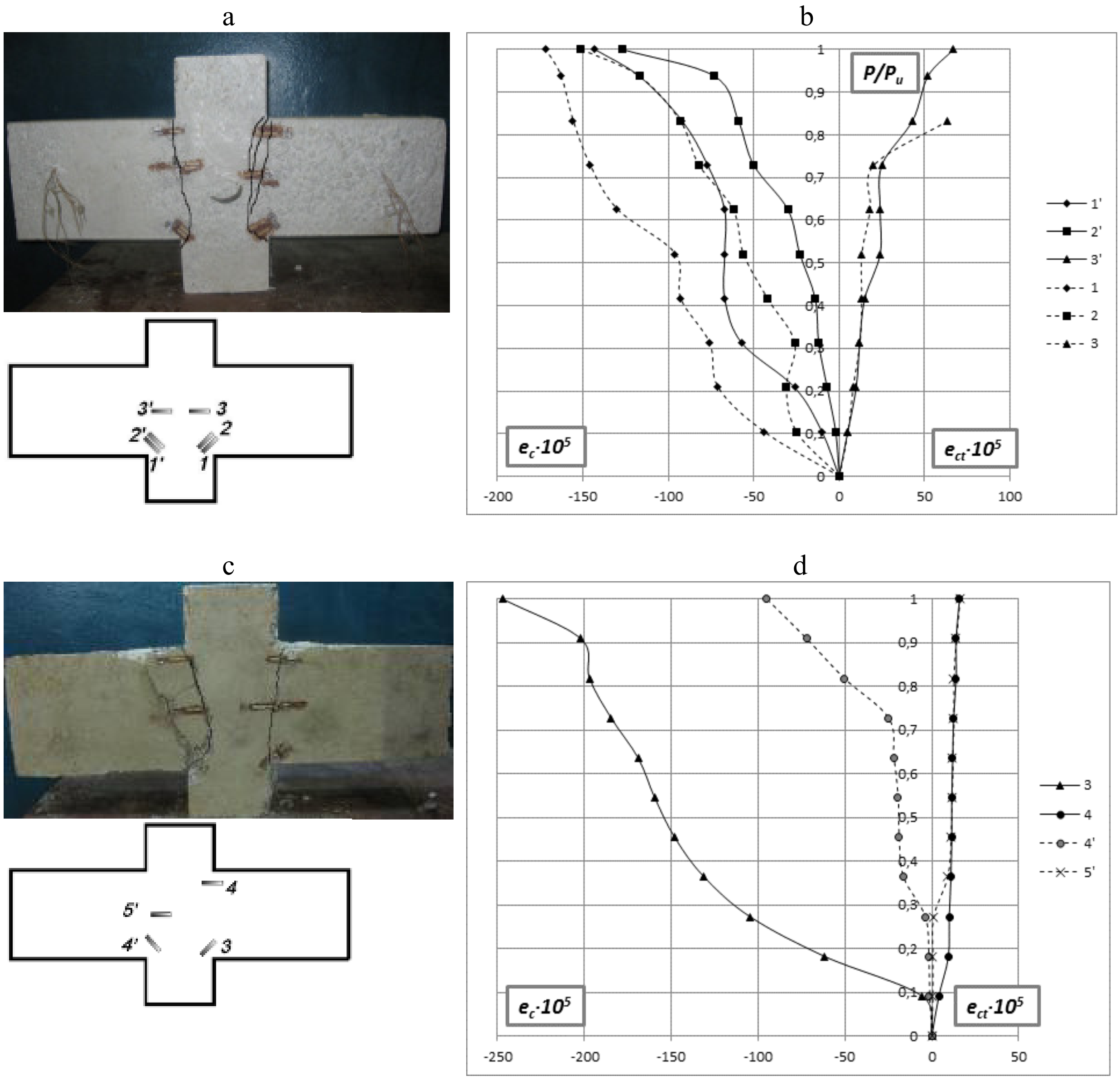


Figure 6 – The failure character (a) and diagram «ε_c (ε_{ct}) – P/P_u» (b) for reinforced concrete keys with one-level and two-level (c, d) plasing of reinforcement

The ultimate loads of all elements were calculated with the help of variation method of concrete plasticity theory. The method is based on the model of rigid-plastic solid and solution in dissoluble functions of velocities. The main point of the method consists of composition of kinematic scheme of failure (fig. 2, a, fig. 3, a, fig4, a, fig. 5). In terms of the scheme the method function is written. It is positive and on its real stress-strain state reaches its minimum that equals zero. From this condition we have the function for the ultimate load determination (1) – (4).

For each of the shear cases as the failure forms the mean arithmetic relation of theoretical $f_{sh}^{calc} = q_u l_k / h_k$ to experimental f_{sh}^{test} strength $\overline{X} = f_{sh}^{calc} / f_{sh}^{test}$, square mean deviation σ_{n-1} and factor of a variation v specified relation were determined. The general factors are $\overline{X} = 0{,}958$, σ_{n-1}=0,051, v=5,3%. For better visualization the theoretical and experimental results comparison is represented in fig. 7.

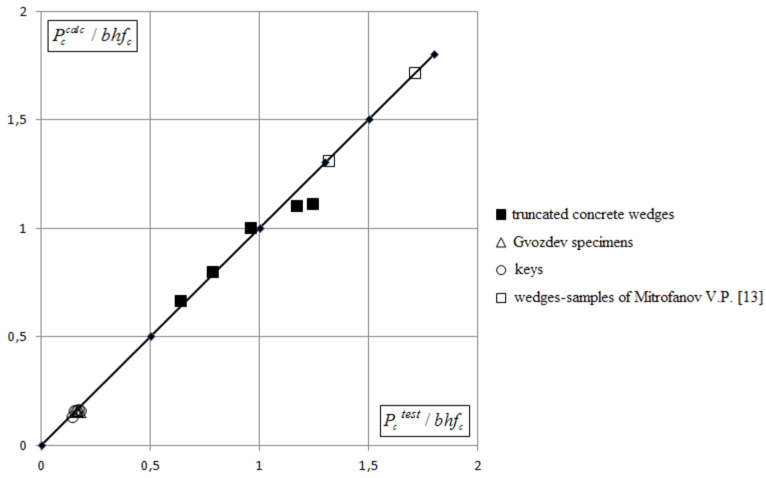


Figure 7 – Comparison of theoretical and experimental strength of specimens

Besides of mentioned above elements the beams investigation was conducted on the shear force in cross section. The first series consisted of 9 samples where concrete of three different classes was used. Concrete placing took place in a climate of «Poltavtransbud» plant within the metal formwork using concrete mixer of constrained action and shaker table for concrete mix packing. The second series consisted of two specimens where concrete of two different types was used: with adding of polypropylene fiber and without it. The aim of this testing series was the verification of possibility of reinforced concrete structures making with higher strength concrete in laboratory conditions.

Testing samples presented the beams of rectangular cross section with the dimensions b×h = 120×180 mm, and length 1500 mm. The first series were reinforced by space frames with longitudinal bars 4Ø16 A400C, transverse bars 2Ø6 A240C with 200 mm interval. As the assembling reinforcement bars were 2Ø6 A240, joint bars were Ø6 A240C. In the beams of second series bars were 2Ø20 A400C as the longitudinal reinforcement (fig. 8).

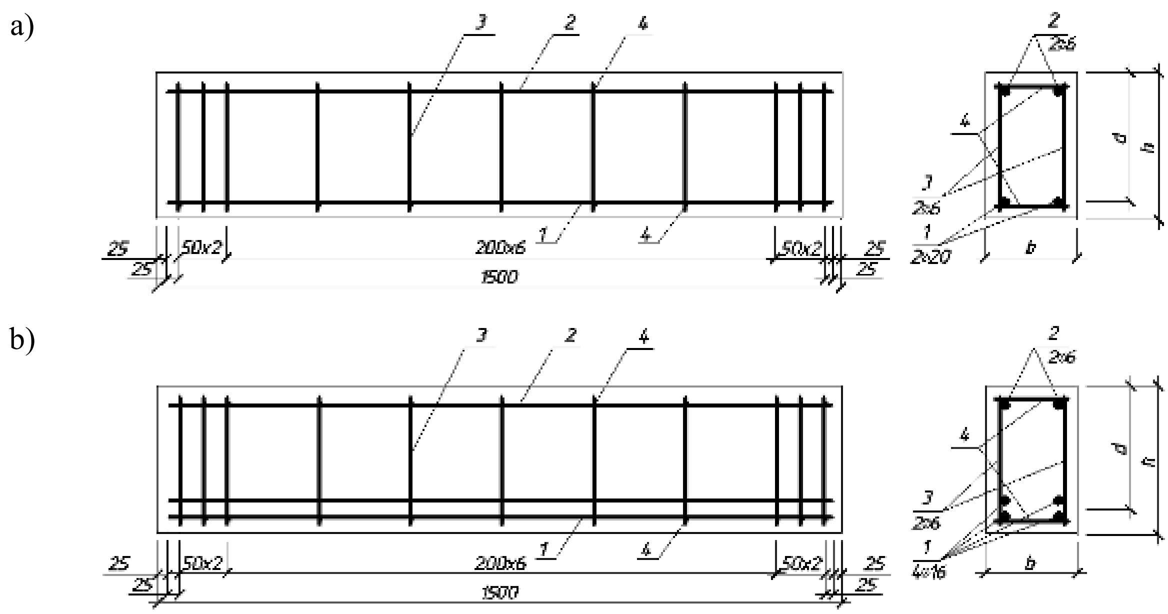


Figure 8 – Geometrical dimensions and reinforcement of beams:
a – first series; б – second series;
1 – longitudinal main reinforcement; 2 – handling reinforcement;
3 – transverse reinforcement; 4 – binding bars

Investigation of testing samples took place in laboratory setup of reinforced concrete and masonry structures and strength of materials chair according to the scheme of «pure bending», the relative span was *a/d*= 2,3 (fig. 9).

Figure 9 – Investigation of beam 1B – FC – 1 on flexure

In all beams by the load level of $V/V_u \approx 0{,}4$ normal cracks appeared first in the middle third of span. During next load stages their opening width increased, inclined cracks appeared (by $V/V_u \approx 0{,}55$). Beams failure took place by means of shear of compressed zone above dangerous oblique crack (fig.10). Displacement of certain blocks along the failure plane took place that is being realized only in the presence of non-elastic deformations on it. By holding on the last load level or load decreasing (over-bound state) failure accompanied with concrete breaking and essential bending of longitudinal reinforcement in the place of inclined crack crossing. It confirmed the presence of dowel effect. Ultimate beams deflections were 10 – 13 mm on the average.

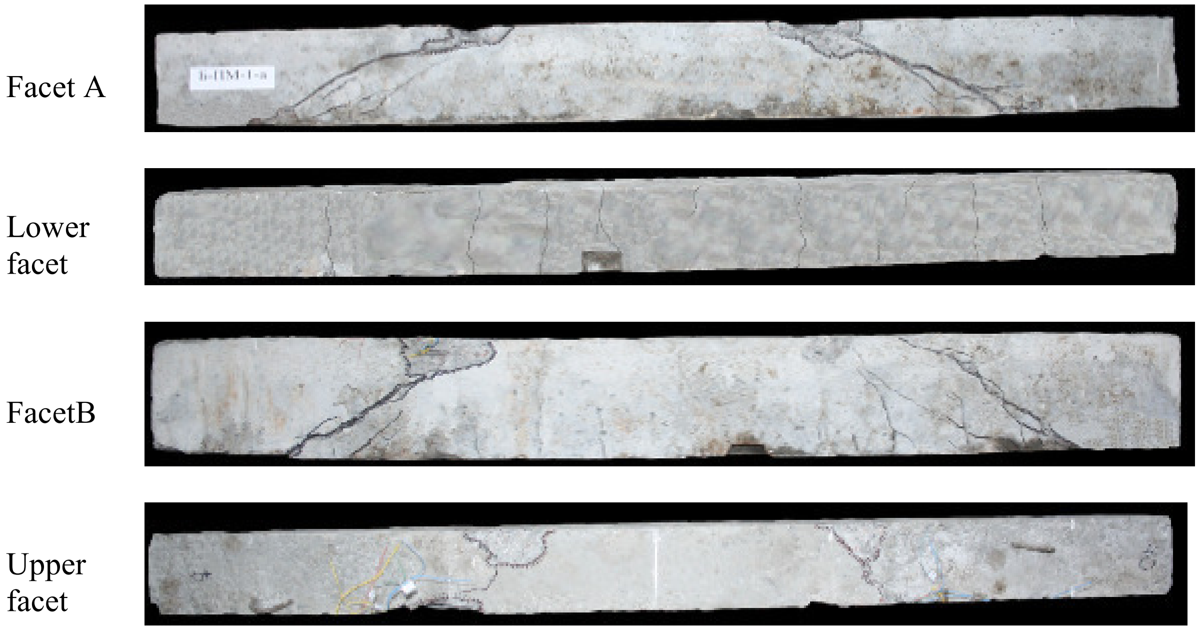


Figure 10 – Failure character of testing beam B-HS-1-a

Theoretical strength of beams was calculated according to the SNiP [20] and DBN [21], results of theoretical and experimental strength comparison are represented in the fig. 11.

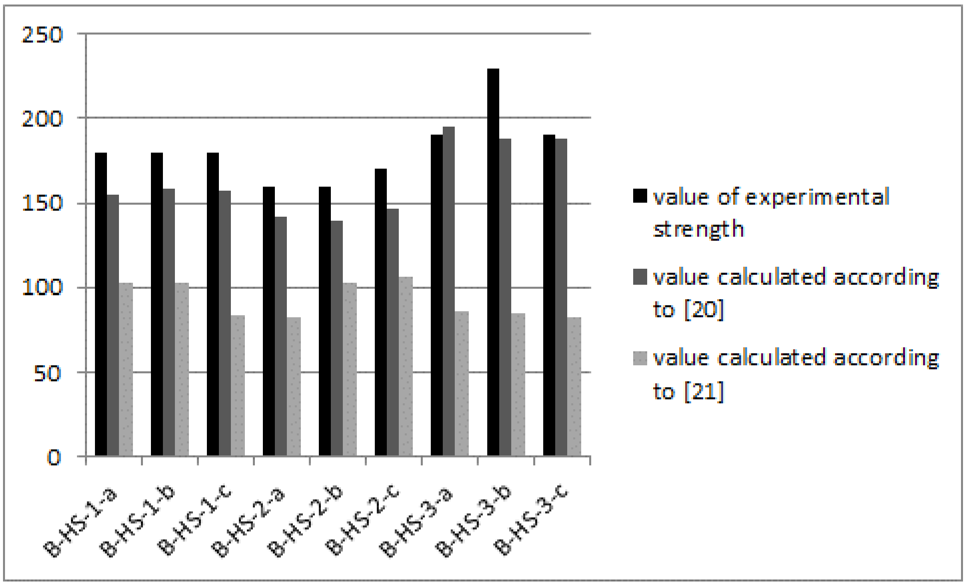


Figure 11 – Comparison of theoretical and experimental strength of beams

Conclusions. Experiments confirmed theoretical kinematical schemes of failure and presence of local plasticity zones regardless of brittle failure character of higher strength concrete elements. Comparing design of theoretical strength calculated with variation method of plasticity theory with experimental strength showed their sufficient proximity. Failure character of testing beams principally didn't differ from failure of bending elements by sloping section of medium strength concrete. Standard design method of beams strength by cross sections showed essential deviation with results of experimental investigation. So, it needs further improvement.

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RESEARCH OF DURABILITY COMPRESSED REINFORCED CONCRETE ELEMENTS FROM HIGH–STRENGTH CONCRETE BASED ON THE DEFORMATION MODEL WITH EXTREME STRENGTH CRITERION

Results of experimental research strength of compressed concrete elements from high-strength concrete are presented. Effect of applying high-strength concrete on ultimate deformation of most compressed concrete brink ε_{cu} and strength of compressed concrete elements are investigated. On the basis of theoretical calculations statistical analysis of strength compressed concrete elements from high-strength concrete in normal sections with obtained experimental data on the basis of improved strength calculating methods of concrete elements from high-strength concrete in normal sections by using program complex «CRC-12» are compared.

***Keywords:** reinforced concrete element, ultimate concrete deformation, deformation model, strength criterion, program complex, high-strength concrete.*

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ДОСЛІДЖЕННЯ МІЦНОСТІ СТИСНУТИХ ЗАЛІЗОБЕТОННИХ ЕЛЕМЕНТІВ ІЗ ВИСОКОМІЦНИХ БЕТОНІВ НА ОСНОВІ ДЕФОРМАЦІЙНОЇ МОДЕЛІ З ЕКСТРЕМАЛЬНИМ КРИТЕРІЄМ МІЦНОСТІ

Наведено результати експериментальних досліджень міцності стиснутих залізобетонних елементів із високоміцних бетонів. Установлено вплив застосування високоміцних бетонів на граничну деформацію найбільш стиснутої грані бетону ε_{cu} й міцність стиснутих залізобетонних елементів. На основі статистичного аналізу порівняно теоретичні розрахунки міцності стиснутих залізобетонних елементів із високоміцних бетонів у нормальних перерізах з отриманими експериментальними даними на основі розробленої вдосконаленої методики розрахунку міцності залізобетонних елементів із високоміцних бетонів у нормальних перерізах з використанням програмного комплексу «CRC-12».

***Ключові слова:** залізобетонний елемент, гранична деформація бетону, деформаційна модель, критерій міцності, програмний комплекс, високоміцний бетон.*

Introduction. In European construction practice reinforced concrete structures with high performance concrete is widely used. Their use is regulated by normative documents [1]. Widespread use in the global construction practice concretes with high performance and technological properties, which is denoted by the term High Performance Concrete (HPC) is evident and irreversible. This trend is caused by growing need of society in unique and reliable engineering buildings and constructions use the high-strength concrete. Characteristic for such concrete is high and ultra-high strength, low penetration, high corrosion resistance and durability, superior deformation characteristics. It should be noted that these properties are achieved with the use of highly mobile mixes, and sometimes self-compacting [5, 6].

Review of recent sources of research and publications. Presently process of harmonization of normative documents for designing of concrete and reinforced concrete structures and their elements with Eurocode-2 is conducted. Concrete durability class is calculated within the range of *C 12/15* to *C 90/105*. In standards functioning in Ukraine and the rules for designing reinforced concrete structures classes of concrete, only durability within the range of C *3,5* to C *60* was calculated. In comparison with the standards [1] this fact corresponds to the classes of concrete durability to *C 50/60*. In [1] classes of concrete durability the limit *C 90/105* is. in Ukrainian functioning norms and rules [2] there are no data regarding the calculation of reinforced concrete elements (RCE) durability from HPC which is true for different types of deformation. That is why the study of these issue as well as definite physical and mechanical characteristics of RCE from high strength concretes is an urgent task.

Parts of the common problems unsolved earlier. Deformation model (DM) application in the theory of the reinforced concrete is the proper step forward, as it makes use of deformed solid body mechanism equations complete set: physical ones for concrete and reinforcement, geometrical and equilibrium equations. As a result, DM allows determining precisely the limit of reinforcement modification, RCE reinforcement modification durability; to consider the character of concrete complete diagrams and to reinforce behavior of other characteristics. Considering descending branches of compression charts which reflects the process of material bearing capacity decrease as a result of increase of its destruction together with the use of DM with extreme strength criterion (ESC); to provide an opportunity to analytically get boundary deformation of RCE concrete compressed brink [3 – 5, 8 – 18].

Experimental research of ultimate characteristics of concrete compressed zone, deformability and strength of RCE from HPC, theoretical calculations at central and noncentral compression and comparison based on the methodology of DM with ESC with experimental research data and their statistical analysis are actual tasks.

Theoretical research of boundary characteristics concrete compressed zone and mentioned above RCE strength were conducted on the basis of DM with ESC and were given in [8 – 18].

Purpose of the research. In connection with lack of experimental data, researches of influence concrete strength class on ε_{cu} value – concrete ultimate deformation compressed zone, and compressed RCE strength, including strength received with HPC, experimental RCE research and executed comparative analysis of obtained results with analytical calculations on the basis DM with ESC were conducted.

The main purpose of experimental research is determination of stress-strain state and internal forces distribution, compressed concrete brink ultimate deformations of high-strength concrete reinforced concrete elements, working under flat central and noncentral compression, and also verification improved methods of calculating strength RCE from HPC in normal sections on the basis DM with ESC [18].

Main material and results. In DM applying the full set of Deformated Solid Mechanics equations is used: physical ones (for concrete and armature), geometrical and equilibrium equations. As a result, it is possible to find overreinforced RCE strength and overreinforced limit more accurately, considering concrete and armature character and other characteristics complete diagrams. Application the DM with ESC gives an opportunity to calculate the strength of reinforced concrete elements in normal section and to receive physical and mechanical characteristics for a wide range of concrete classes (from C 3,5 to C 120 and more) [3 – 5, 17, 18].

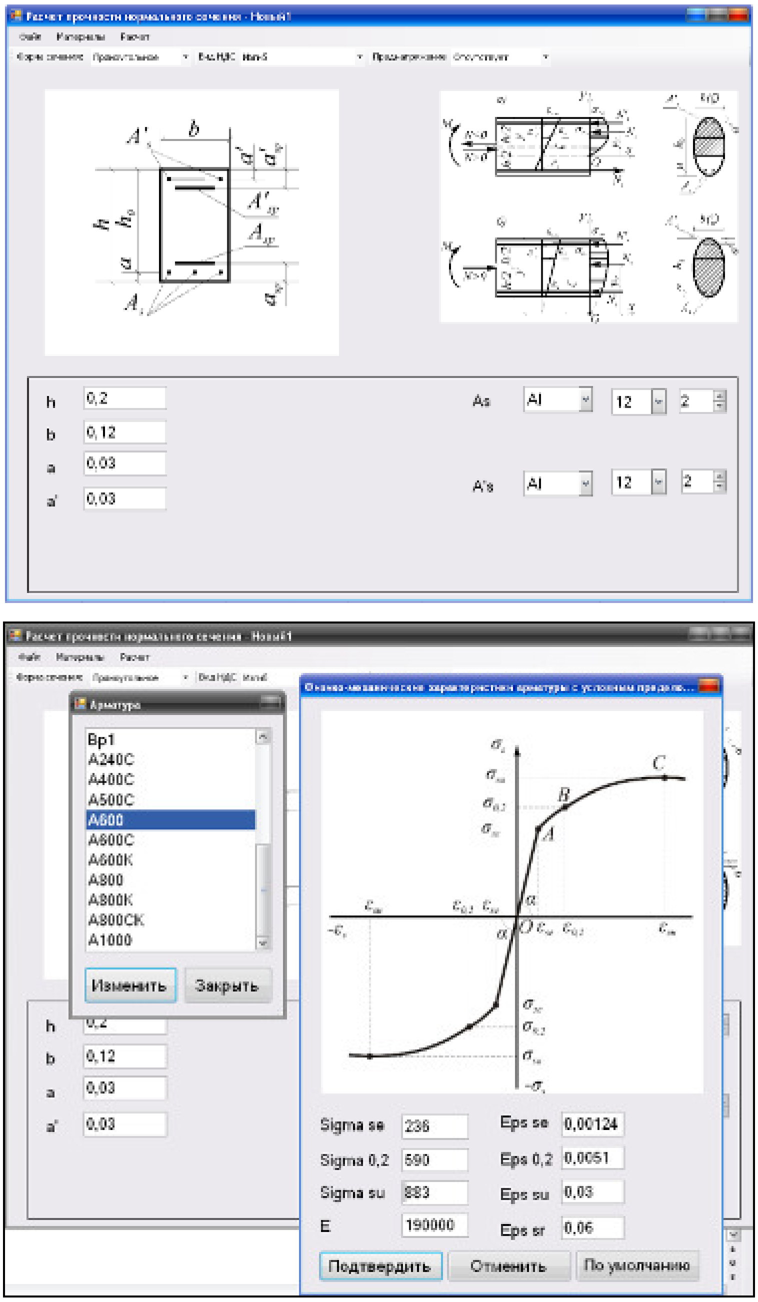

Figure 1 – The software complex «CRC – 12»

For the solution of the problem checking strength of compressed reinforced concrete elements in the normal section on the basis on DM with ESC on PC algorithm of engineering methods of RCE calculation and software package «CRC – 12» (Figure 1) [18] with application numerical and optimization methods for different classes of concrete strength was developed, including high strength (from 10 to 120 MPa) and for different classes of reinforcing steel.

In the process of experimental research of concrete compressed reinforced concrete elements deformation and reinforcement in specific cross sections of reinforced concrete elements were measured. Particular attention was given to determination of concrete ultimate deformations and reinforcement in specific sections of reinforced concrete elements.

For studying, concrete class and load application eccentricity on strength and deformation reinforced concrete elements were impacted; test samples have been divided into three groups:

1. Columns, concrete compressive strength $f_{cm,cube}$ = 61 MPa, eccentricity of load application e_0 = 0 mm (К-1-1-Ц, К-1-2-Ц), e_0 = 3 mm (К-1-3-МЕ, К-1-4-МЕ), e_0 = 12 mm (К-1-5-ВЕ, К-1-6-ВЕ);
2. Columns, concrete compressive strength $f_{cm,cube}$ = 69 MPa, eccentricity of load application e_0 = 0 mm (К-2-1-Ц, К-2-2-Ц), e_0 = 3 mm (К-2-3-МЕ, К-2-4-МЕ), e_0 = 12 mm (К-2-5-ВЕ, К-2-6-ВЕ);
3. Columns, concrete compressive strength $f_{cm,cube}$ = 75 MPa, eccentricity of load application e_0 = 0 mm, (К-3-1-Ц, К-3-2-Ц), e_0 = 3 mm, (К-3-3-МЕ, К-3-4-МЕ), e_0 = 12 mm (К-3-5-ВЕ, К-3-6-ВЕ);

Central and noncentral compressed reinforced concrete elements test results were analyzed on specific samples according to the class of concrete, type and eccentricity of load application. The samples by length have a rectangular cross section with a constant height and dimensions – 120×120 mm for transfer of eccentricity application of load in the supporting part of the column. Schemes of geometric dimensions, reinforcement and location of strain gauges on concrete were shown at Figure 2 and Figure 3.

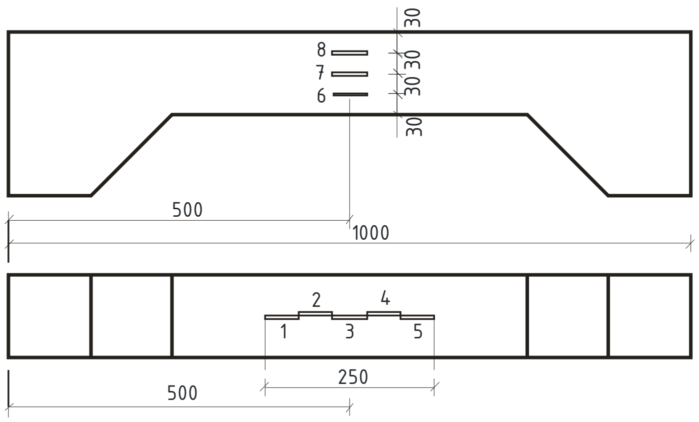

Figure 2 – The scheme of arrangement a strain sensors on the concrete surface

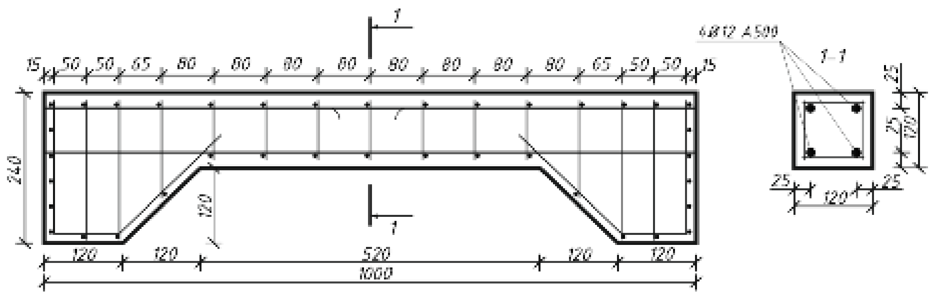

Figure 3 – Scheme of reinforcement and geometrical dimensions of experimental samples (concrete columns)

Reinforcement of the samples was performed symmetrically, four longitudinal armature rods class A500, diameter 12 mm, connected with spatial cage using welding clamps armature with class A240C, diameter 6.5 mm, with increments 80 mm (Figure 3, 4).

Figure 4 – Spatial reinforcing cage for reinforced concrete columns

According to data obtained from experimental researches ultimate characteristics of concrete compressed zone and normal sections tested samples strength were analyzed. Statistical analysis of the results is shown in Table 1.

Table 1 – Experimental and theoretical values of deformation and strength of concrete elements normal cross section in the stage, close to destruction

#	Model code	The maximum relative deformation of the most compressed brink of concrete		$\varepsilon_{cu,teor}/\varepsilon_{cu,test}$, %	Longitudinal force N_u, *MH*		$N_{u,teor}/N_{u,test}$, %
		$\varepsilon_{cu,test}$, ‰	$\varepsilon_{cu,teor}$, ‰		$N_{u\ test}$	$N_{u\ teor}$	
1	К-1-1-Ц	-2,40	-2,436	1,015	0,803	0,8059	1,0036
2	К-1-2-Ц	-2,38	-2,436	1,0235	0,804	0,8059	1,0023
3	К-1-3-МЕ	-3,24	-3,293	1,0163	0,405	0,4054	1,0009
4	К-1-4-МЕ	-3,22	-3,293	1,0226	0,404	0,4054	1,0034
5	К-1-5-ВЕ	-3,50	-3,763	1,0751	0,109	0,1118	1,0256
6	К-1-6-ВЕ	-3,67	-3,763	1,0253	0,11	0,1118	1,0163
7	К-2-1-Ц	-2,47	-2,507	1,0149	0,900	0,9097	1,0107
8	К-2-2-Ц	-2,48	-2,507	1,0108	0,920	0,9097	0,9888
9	К-2-3-МЕ	-3,31	-3,337	1,0081	0,450	0,4531	1,0068
10	К-2-4-МЕ	-3,28	-3,337	1,0173	0,445	0,4531	1,0182
11	К-2-5-ВЕ	-3,41	-3,565	1,0454	0,115	0,1174	1,020
12	К-2-6-ВЕ	-3,46	-3,565	1,0303	0,116	0,1174	1,0120
13	К-3-1-Ц	-2,55	-2,577	1,0105	1,00	1,0128	1,0128
14	К-3-2-Ц	-2,56	-2,577	1,0066	1,05	1,0128	0,9645
15	К-3-3-МЕ	-3,39	-3,383	0,9979	0,49	0,4998	1,02
16	К-3-4-МЕ	-3,37	-3,383	1,0038	0,50	0,4998	0,9996
17	К-3-5-ВЕ	-3,36	-3,392	1,0095	0,12	0,1221	1,0175
18	К-3-6-ВЕ	-3,35	-3,392	1,0125	0,118	0,1221	1,0347
Mean arithmetic value				**1,0192**			**1,0088**
Mean squared deviation				**0,0177**			**0,0155**
Coefficient of variation, %				**1,7337**			**1,5331**
Asymmetry				**2,1017**			**-1,2412**
Excess				**5,4891**			**3,1178**

Analysis of experimental data made it possible to confirm their good convergence and accuracy of strength theoretical research results, physical and mechanical properties of concrete compressed zone, etc., obtained by the method on the basis DM with ESC.

Using the DM with ESC theoretical and experimental data of the most compressed fiber of concrete (ε_{cu}) and normal sections strength of reinforced concrete element boundary deformations were compared.

Conclusions. The constant value of ε_{cu} ≈ 3,5% taken in Eurocode-2 for concrete low and medium strength is overestimated for compressed concrete elements. Considering of increased fragility in the area of high-strength concrete physical would be more reasonable introduction to calculate higher reliability coefficients or lower coefficients of working conditions, rather than as it is customary in Eurocode-2, – decline ultimate deformation ε_{cu} which does not agree with experiments and calculations by the deformation model with extreme strength criterion. At the ultimate deformation of compressed concrete elements ε_{cu} many factors are involved that must be considered in their strength calculation. By calculations results based on deformation model with extreme strength criterion, ε_{cu} its value changes significantly when changing class of concrete, class of reinforcing steel, load character, cross-sectional form, etc. Conducted experimental research provides an opportunity to make conclusion about the reliability of deformation model with extreme strength criterion. Deformation models with extreme strength criterion allows to analyze a full complex of normal sections limit parameters in the stage of their destruction, also to use the corresponding calculation dependencies and to detect elastic or plastic state of reinforcing steel.

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TEST METHOD OF CLT BY TENSION PERPENDICULAR TO GRAIN

It is known that cross laminated timber (CLT) is a relatively new material in timber design. There is no information about its behavior and strength in tests by tension perpendicular to grain. Little information available is based only on simple tests for assessing the bearing capacity of some variants of devices which are used for the installation of CLT panels. In practice tension perpendicular of CLT panels occurs in curved CLT panels and in some wall-floor connections with screws. First proposed of test setup is based on the parameters related to solid timber and glulam after detailed analysis. The main influence factors on the tensile strength perpendicular to grain were investigated, and the necessary sizes of the CLT specimens were determined considering the features of the material structure. The recommendations derived in this publication can be included in standard EN 16351 or EN 408.

Keywords: *cross laminated timber, tension perpendicular to the grain, proposing of size of test specimens, test arrangement.*

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МЕТОДИКА ВИПРОБУВАНЬ ПОПЕРЕЧНОЇ КЛЕЄНОЇ ДЕРЕВИНИ ПРИ РОЗТЯГУ ПОПЕРЕК ВОЛОКОН

Відомо, що поперечна клеєна деревина (ПКД) є відносно новим матеріалом на основі деревини. На сьогоднішній день відсутня інформація щодо міцності ПКД- панелей при розтягу поперек волокон. Деяка інформація існує та ґрунтується на результатах міцності при простих випробуваннях деяких варіантів монтажних кріплень панелей перекриттів. На практиці розтяг поперек волокон ПКД панелей спостерігається у зігнутих панелях, а також у деяких гвинтових з'єднаннях стінових панелей з панелями перекриттів. Уперше запропоновано новий метод дослідження ПКД, що базується на параметрах, які враховують модель випробувань цільної та клеєної деревини після детального аналізу. Досліджені основні впливові фактори на величину міцності при розтягу поперек волокон, та встановлено необхідні розміри зразків ПКД панелей виходячи з особливостей структури матеріалу. Рекомендації, описані у даній публікації, можуть бути включеними до стандарту EN 16351 або EN 408.

Ключові слова: *поперечна клеєна деревина, розтяг поперек волокон, пропозиція розмірів випробувальних зразків, схема випробування.*

Introduction. Moreover, the German national design code contains recommendations for the reinforcement of certain parts of timber elements which are loaded by tension perpendicular to grain. Before resorting to reinforcement, it should be known the strength of material by stressed conditions which result in certain fracture modes. As it is known, ductile fracture mechanism is a more desirable and safer type of timber elements disruption (failure) than brittle failure. Various tests show that it is brittle failure that occurs most often in timber by tension perpendicular to grain. The transfer from brittle to ductile failure mechanism was achieved by using different type of screws.

According to former standards EN 338:2003 and EN 1194:1999 strength properties of solid timber and glulam are dependent on timber density. According to Russian national rules, strength by tension perpendicular to grain of glulam is also dependent on strength classes (3 classes), but German national rules DIN 1052:2008-12 contain only one value of strength by tension perpendicular for all strength classes. Now EN 1194:1999 has been revoked and all the data of strength properties of GLT are stipulated in EN 14080:2013 [11], where strength values by tension perpendicular are changed and considered to be equal for all strength classes. A similar situation occurs in the new version of EN 338:2009 for solid timber, where strength data by tension perpendicular are also equal.

The orientation of annual rings in cross section of solid timber specimens influences the test data of strength and stiffness by tension perpendicular to grain (Fig. 1).

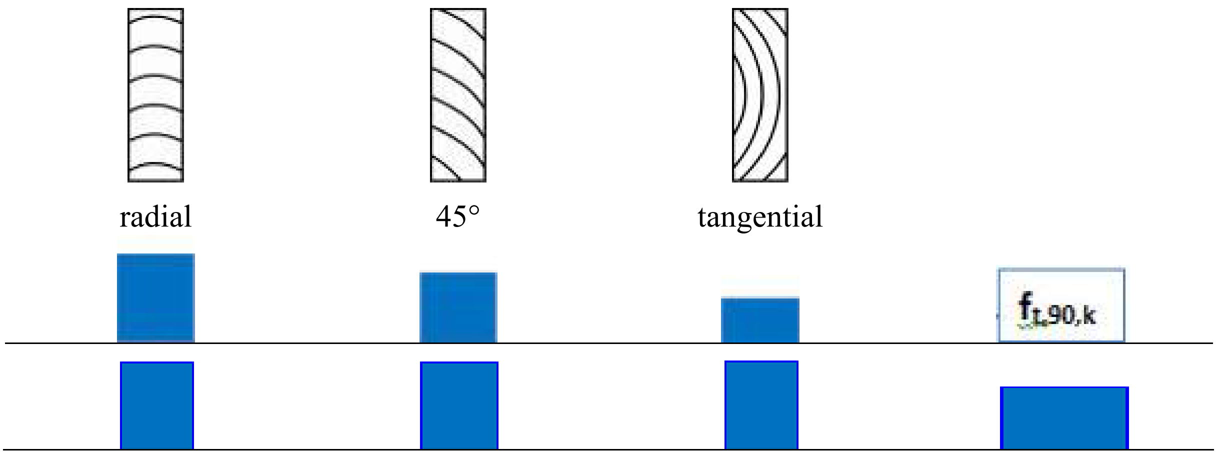

Figure 1 – Dependence of strength and stiffness on the annual ring orientation in specimens of solid timber

The dependence of strength values on size effect in the cross section parameters of solid timber, glulam and LVL occurred by bending and tension along the grain direction. Additionally, GLT shows volume effect by tension perpendicular to grain [3]. In accordance with EN 1193, ST and GLT specimens have geometric parameters as shown in figure 2, which are considered in a number of publications by Blass [1, 2], Aicher [3], Ranta-Maunus [4] and others. The reference volume is accepted as V_0=0,01m^3 and it is taken into account during the design of double tapered, pitched cambered and curved beams where tension stresses perpendicular to the grain appear in the apex zone.

The maximum tensile stress perpendicular to grain by bending of tapered beams should be calculated according to equations which are proposed in EC-5, clause 6.4.3 (8). This clause is a matter of national choice. It is well known that climate change leads to cracks and consequently results in residual working area in longitudinal section.

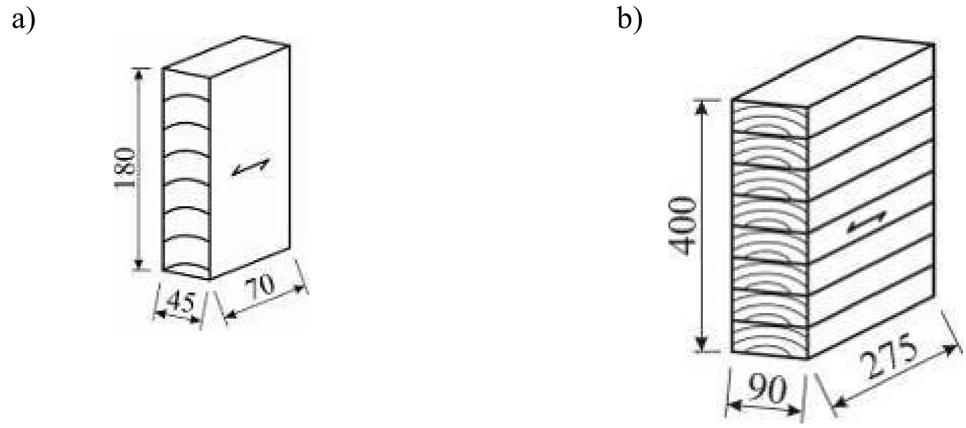

Figure 2 – The size of test specimens according to EN 1193:1998:
a – solid timber; b – glued laminated timber

Cross laminated timber is a new and modern timber plate orientated material in building industry. In practice tension perpendicular of CLT panels occurs in wall-floor connections with screws, as well as in notched connections of floor panels supported on wall panel and in curved CLT panels, as it is shown in figure 3.

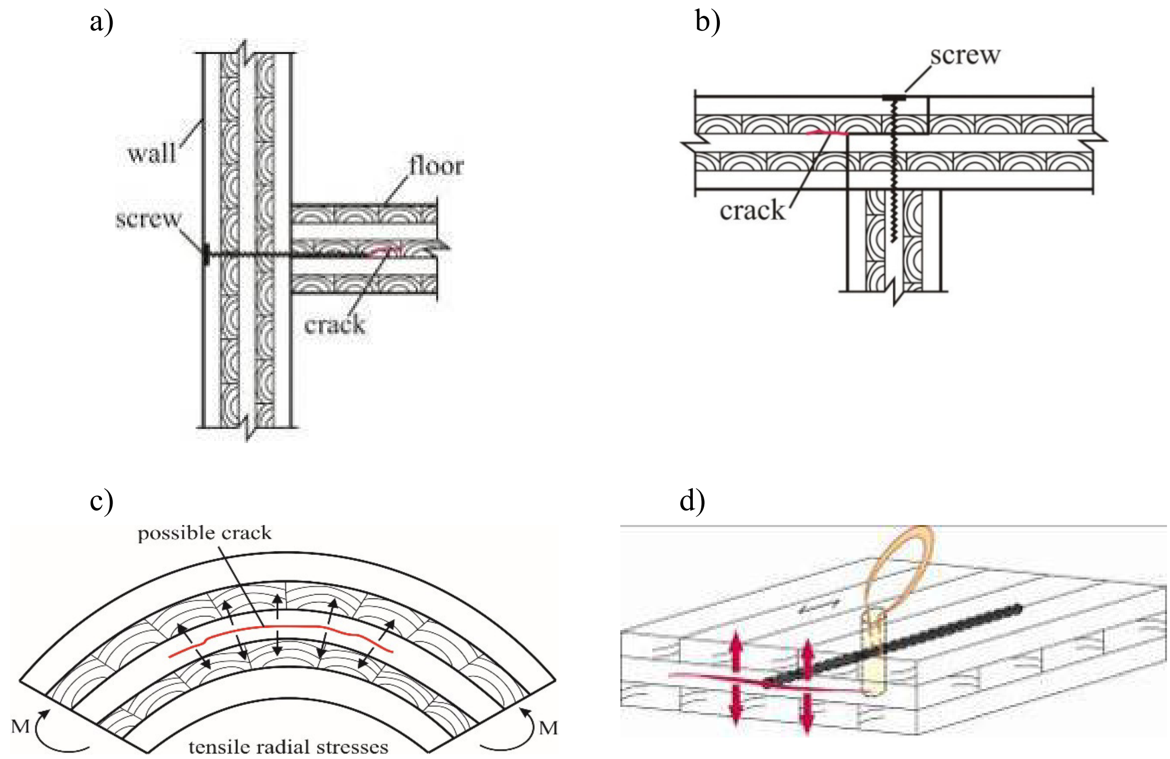

Figure 3 – Examples of tension perpendicular of CLT in practice:
a – wall-floor connection; b – floor-floor -wall connection;
c – curved CLT panels; d – mounting parts

Review of the latest research sources and publications. As it was introduced in section 1, strength by tension perpendicular to grain is dependent on a number of major factors such as density, volume of tested specimens and grain orientation in cross section of solid timber elements (Fig. 4).

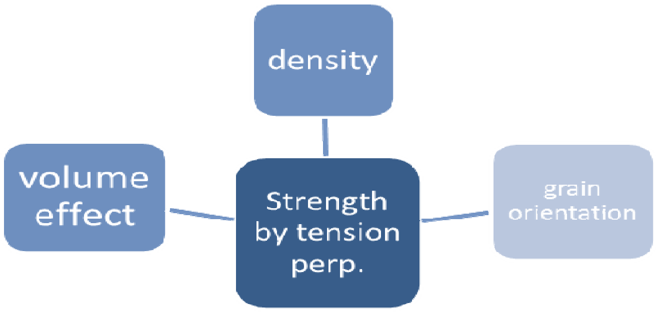

Figure 4 – Main factors that influence the strength by tension perpendicular

The tensile strength for solid timber according to former versions of standards EN 384:1995 [10] and EN 338:2003 was derived on the basis of the relationship between tensile strength perpendicular to grain and density:

EN 384: 1995 $$f_{t,90,k} = 0{,}001 \cdot \rho; \tag{1}$$

EN 338:2003 $$f_{t,90,k} = \min\begin{cases} 0{,}6 \\ 0{,}0015 \cdot \rho_k . \end{cases} \tag{2}$$

Strengths properties of glued laminated timber by tension perpendicular to grain according to EN 1194:1999 are based on the following equation:

EN 1194:1999 $$f_{t,90,g,k} = 0{,}2 + 0{,}015 \cdot f_{t,0,1,k}, \tag{3}$$

where $f_{t,90,g,k}$ is glulam strength perpendicular to grain and $f_{t,0,1,k}$ is the tensile strength parallel to grain of these planks.

The characteristic value of strength $f_{t,90,g,k}$ is related to the reference volume of V_0=0,01m^3. The test method for determining the tensile strength perpendicular to grain of glulam using this reference volume V_0 is stipulated in EN 1193 where the specimen parameters are 90 mm×275 mm×400 mm (Fig. 2) and the volume is 0,0099 m^3.

In order to show more vividly the correlation between density and strength by tension perpendicular in solid wood and glulam, «X» parameter is introduced. Equation (1) and (2) can be written in the following modified way:

$$f_{t,90,k} = \frac{1}{X} \cdot \rho_k \quad \text{or} \quad f_{t,90,k} = \frac{\rho_k}{X} , \tag{4}$$

which means that X parameter equals:

$$X = \frac{\rho_k}{f_{t,90,k}} . \tag{5}$$

Tables 1 and 2 show the values of X parameter for solid timber and glulam referring to data taken from EN 338 and EN 1194. The graphic illustration of X parameter is shown in figures 5 and 6.

Table 1 – X parameter for glued laminated timber

	GL24c	GL24h GL28c	GL28h GL32c	GL32h GL36c	GL36h
$f_{t,90,k}$	0,35	0,4	0,45	0,5	0,6
ρ_k	350	380	410	430	450
x	1000	950	911,1	860	750

Table 2 – *X* parameter for solid timber

	C14	C16	C18	C20	C22	C24	C27	C30	C35	C40	C45	C50
$f_{t,90,k}$	0,4	0,5					0,6					
ρ_k	290	310	320	330	340	350	370	380	400	420	440	460
x	**725**	**620**	**640**	**660**	**680**	**700**	**617**	**633**	**667**	**700**	**733**	**767**

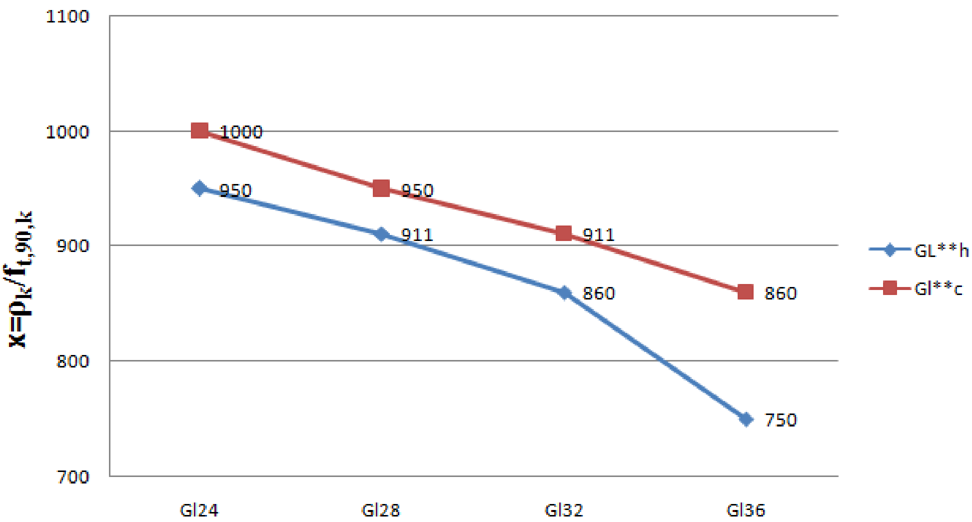

Figure 5 – The illustration of *X* parameter for glued laminated timber

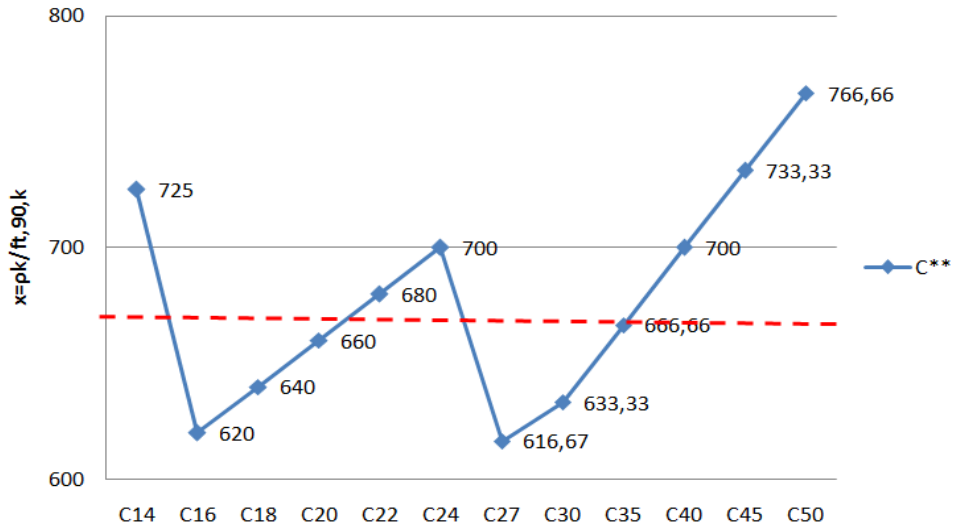

Figure 6 – The illustration of *X* parameter for solid timber

After some transformations, the equation (2) for *X* parameter is expressed in the following form as shown in equation (6):

$$f_{t,90,k} = \min \begin{cases} 0{,}6 \rightarrow \dfrac{\rho_k}{X} = 0{,}6 \rightarrow X = \dfrac{\rho_k}{0{,}6} \\ 0{,}0015 \cdot \rho_k \rightarrow X = \dfrac{1}{0{,}0015} = 666{,}666. \end{cases} \quad (6)$$

For maximum and minimum values of soft solid timber density, X parameter acquires the following values:

$$X_{\min} = \frac{\rho_{k,\min}}{0,6} = \frac{290}{0,6} = 483,33; \qquad X_{\max} = \frac{\rho_{k,\max}}{0,6} = \frac{460}{0,6} = 766,66. \tag{7}$$

The critical value of X parameter, when equation (2) is divided into two sub-equations, can be found by simple substitution:

$$\frac{\rho_k}{X} = 0,6 \rightarrow \rho_{k,crit} = 0,6 \cdot X_{crit} = 0,6 \cdot 666,667 \cong 400 \text{ kg/m}^3$$

The analysis of the relationship between strength perpendicular and density is important to understand the underpinnings behind the equations and data in the revoked standards or former versions of these standards. Nowadays, there is a certain simplification and averaging in some of the strength and elastic characteristics of glulam and solid timber.

Problem statement. The method of conducting tests into tension perpendicular of ST or GLT is described in EN 1193, as it is shown in figure 7. Now Standard EN 1193:1998 has been revoked and new recommendations are provided in EN 408:2010 [12] with some amendments. Intermediate solid timber or glulam stressed parallel to grain are bonded to the specimens. It is reasonable to install two inductive measuring gauges in diagonal position.

As it is shown in figure 7, in the new version of standard EN 408:2010 it is possible to connect a steel plate immediate to the investigated test specimen or the apply tensioned loading directly to the intermediate wood elements.

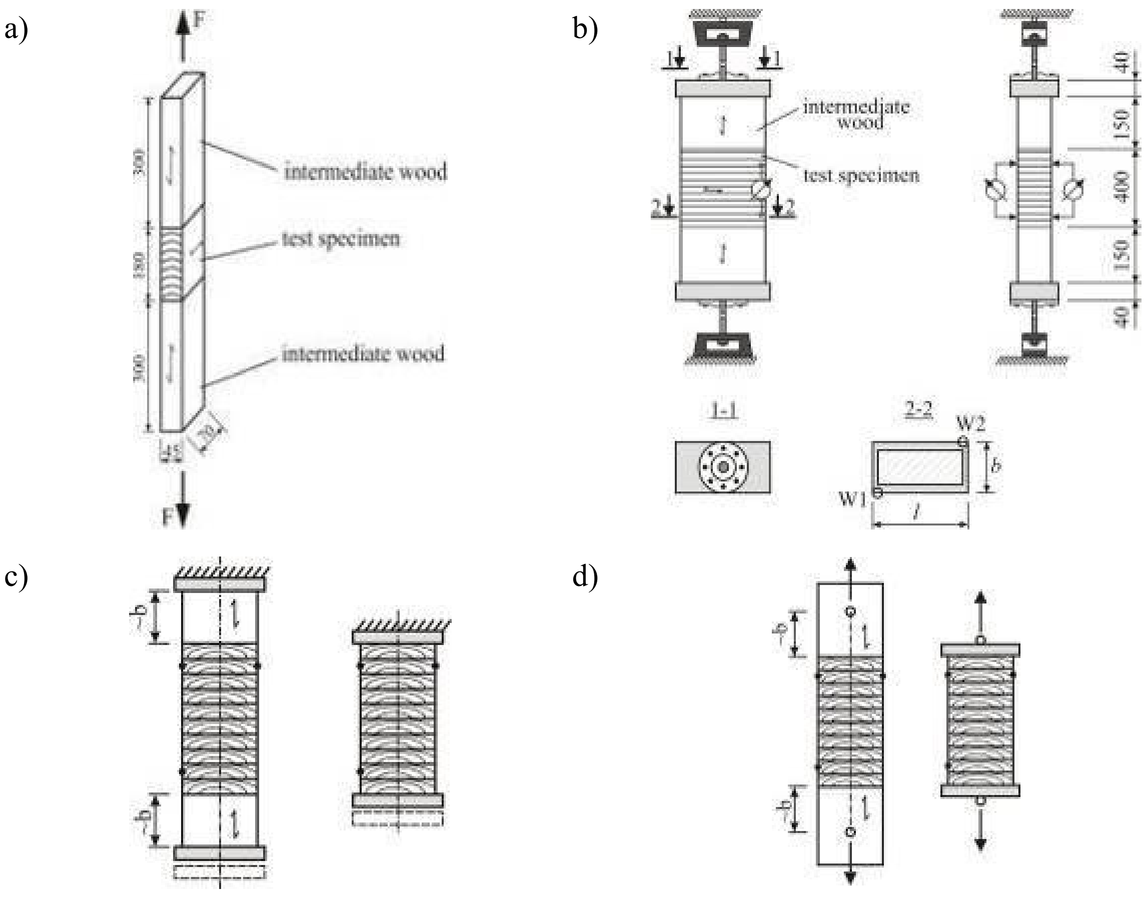

Figure 7 – Test arrangement:
a – solid timber; b – glued laminated timber EN 1193;
c – glued laminated timber EN408:2010

There are data from national standards of different countries and from different periods shown in table 3 that allow a comparison between strength values of solid timber and glulam by tension perpendicular to grain. The strength of glulam is a little higher than that of solid timber and this fact explains the system effect of glued lamellas. CLT is composed of an uneven number of layers which are arranged crosswise to each other at the angle of 90° and the system effect will occur in a different way, influencing the value of strength by tension perpendicular.

Table 3 – Characteristic and design values of strength properties of solid timber (ST) and glued laminated timber (GLT) by tension perpendicular to grain according to standards of different countries

Country	Standard	ST (N/mm^2)		GLT (N/mm^2)		Equation
		$f_{t,90,k}$	$f_{t,90,d}$	$f_{t,90,k}$	$f_{t,90,d}$	
Euro standard	EN 338	0,4 0,6				softwood hardwood
	EN 384	0,4-0,6				
	EN 1194 (revoke)			0,35-0,6		
	EN 14080:2013	0,5		0,5		for all classes
Germany	DIN 1052:2008-12	0,4		0,5		for all classes
UK	BS 5268-2-2002	0,2-0,47		-	-	$f_{t,90}$=fv/3
USSR	СНиПІІ-25-80	-		0,6-0,8	0,25-0,35	
Ukraine	ДБН В.2.6-161:2017	0,4* 0,5**		0,5		*softwood **hardwood
Russia	СП 64.13330.2016	0,4-0,6		0,25-0,35	0,12-0,23	
Belarus	ТКП EN 1995-1-1-2009	0,4-0,6		0,35-0,6		
Swiss	SIA 265:2012		0,1		0,15	for all classes

While considering CLT panels with timber based plate material (LVL and plywood) located inside, we need to take into account the strength by tension perpendicular to grain of these materials. If the strength of timber-based materials proves to be lower than the strength of solid timber lamellas, there occurs a system with a weak link.

As it is known, the strengths of plywood and LVL in tension perpendicular to plate area are absent in technical specifications of different producers and technical reports of research organizations, see table 4. That is why the assessment of strength properties of non-homogeneous CLT plates becomes rather difficult.

Table 4 – Characteristic values of strength properties of LVL by tension perpendicular to grain

	Timber based plate material	$f_{t,90,k}$ (N/mm^2)	Organization, № of technical report or certificate	Refe-ren-ces
LVL	Steico Ultralam, 2015	-	DIBT Z-9.1-811	[7]
	Ultralam (Russia), 2010	-	TSNIISK (Moscow) СТО 36554501-021-2010	[9]
	Ultralam (Russia), 2009 2009	- -	MPA Stuttgart Certificate CE 0672-CPD-I 14.04.1 Bericht 51220-901.6453.000/2	[8]
	Kerto –S and –Q Metsa: Wood (Finland) 2004, Update 2016.	-	VTT Certificate № 184/03	[6]
	Kerto – S and -Q Metsa: Wood (Finland)2011	-	VTT Statement VTT-S-05156-11	[5]

The basic material and results. EN 16351 [13] proposed tests for CLT plates with the cross section of lamellas 30×150 mm. Besides, it is known that the technology of CLT production allows some gaps between lamellas in the layers. In addition, lamellas of middle layers in some cases have relieves (see figure 8, right). Gaps and relieves reduce longitudinal section. This weakening needs to be taken into account when calculating strength values.

Figure 8 – Gaps and relives in CLT cross section

Different strength of planks depending on the grain orientation leads to uneven distribution of stresses in the layers of CLT plate. Gaps and relieves create constructive heterogeneity which also influences the uniformity of stress distribution (see figure 9).

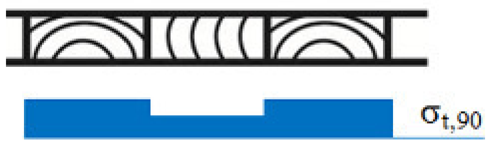

Figure 9 – Uneven distribution of stresses by tension perpendicular to grain

As it is shown in figure 7, the investigated test specimens are glued to both sides of the timber element and then glued to steel plates. Grain direction in auxiliary timber elements coincides with the direction of the applied load. The steel plate can be 40 mm thick, as in the test for GLT in accordance with EN 1193:1998.

Test setup can be arranged in two ways with difference volume. The first version is simple and can be used to obtain the difference and calculate the volume factor in CLT. In this proposed arrangement the tensioned perpendicular area of timber specimens is different and the system effect will influence the obtained test data. The volume of specimen in figure 10 (on the right) is 0,00945m^3 which is close to the reference volume of glulam

($0{,}01m^3$). The area of cross section exceeds 2500 mm^2 as it is recommended by standard EN 408:2013 for GLT. The increased area of tested specimen is explained by the fact that CLT is a plate timber material. The width of the specimen is similar to GLT (more than 100mm) and it is proposed to use 150mm as a reference size of planks.

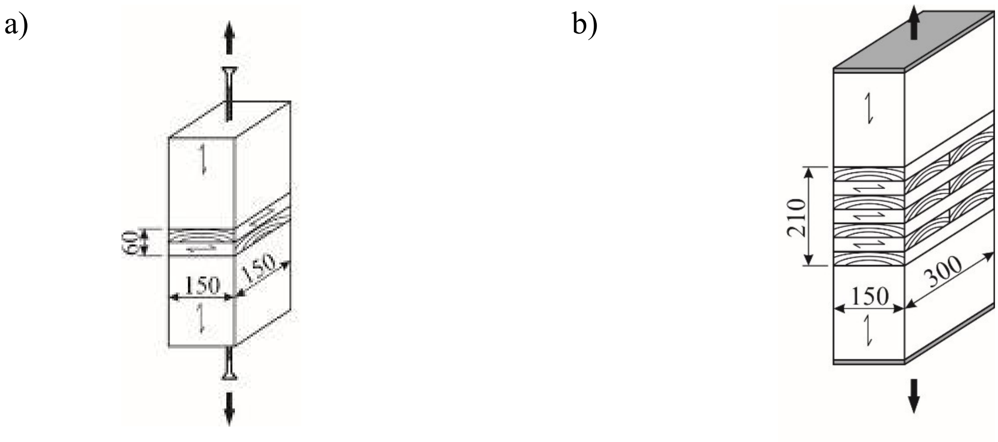

Figure 10 – Test arrangement for CLT:
a – small specimen; b – CLT specimen

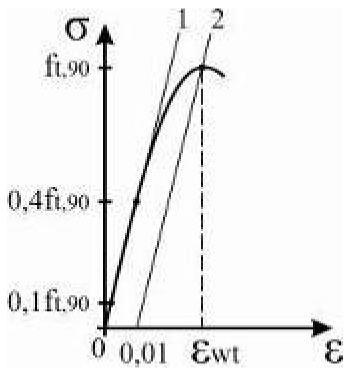

Figure 11 – Proposal for the loading protocol

The load F shall be applied at the constant rate of cross head movement throughout the test. The rate of loading shall be adjusted so that the maximum load $F_{t,90,max}$ is reached within (300 ± 120) s.

The structure of CLT panels assumes the use LVL and plywood in the middle layers. However, the values of strength of timber-based materials by tension perpendicular to grain are unknown and this fact makes it more difficult to analyze and predict the strength this type of CLT panels. Table 4 shows that research laboratories have not yet considered the strength by tension perpendicular to grain in flatwise position of timber based materials such as LVL and plywood. EN 16351 [13] does not contain exact information as to how many layers of LVL or plywood a CLT panel can possibly have (see figure 12). Nor does it say anything about the acceptable percentage of these layers in timber-based plate materials.

Figure 12 – Variants of CLT panels with LVL or plywood

Basic principles of arranging tests into tension perpendicular to grain for CLT have been transferred from the methods proposed for ST and GLT. The suggested proposal for a similar test into CLT panels takes into account the fact that CLT is a plate building material and the plate configuration of specimens is preferable.

The issues of size effect (area factor) and volume effect remain open and require further investigation effort. The reference volume V_0=0,01 for CLT is obviously small in comparison with the volume of CLT panel. Without experimental results it is not possible to determine what type of law of distribution corresponds to CLT by tension perpendicular to grain.

The strength of CLT panels with LVL and plywood in the middle layers is difficult to assess by tension perpendicular to grain because the strength of these veneer-based materials is unknown in this type of stress condition.

The knowledge of strength by tension perpendicular allows us to take decisions which will decrease action perpendicular or suggest reinforcement of basic connections, such as wall-floor connection (see figure 13).

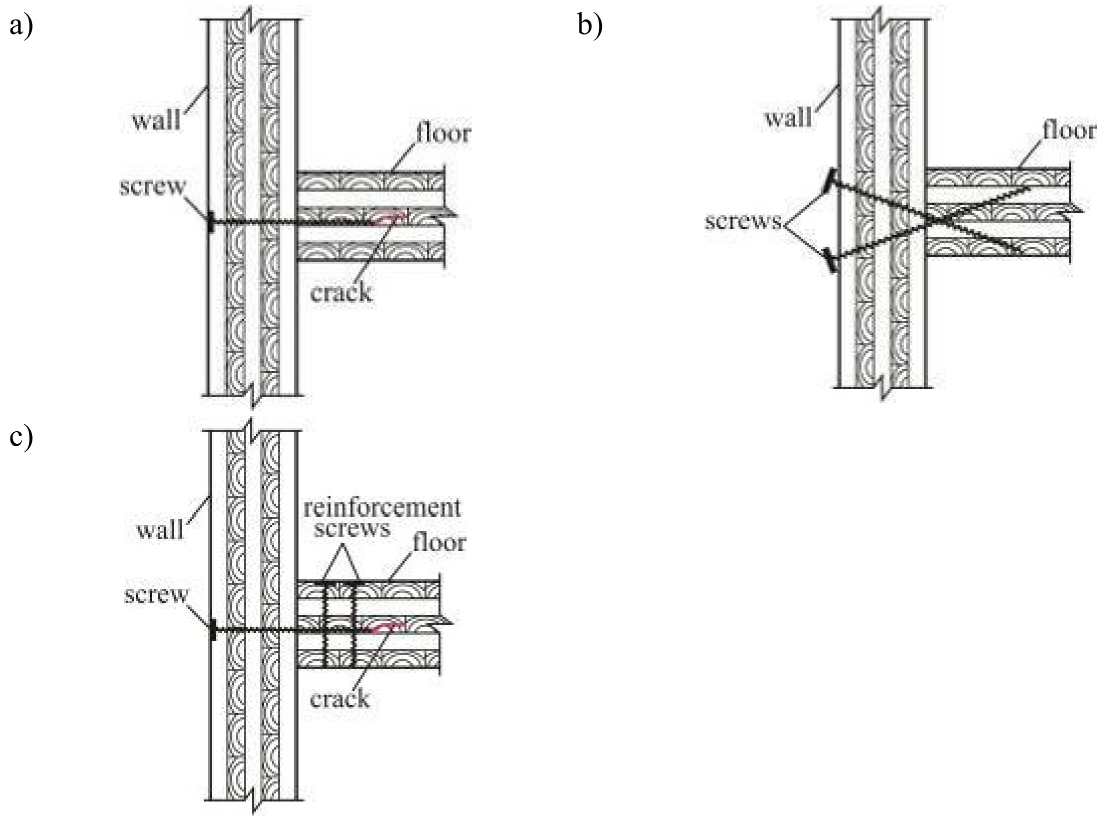

Figure 13 – Variants of connection modification:
a – basic connection; b – decrease action perpendicular; c – reinforced basic connection

The reduction of action perpendicular is achieved by installing screws at angles different from the layers direction in CLT floor element. The inclination of the screws can occur in two planes.

Conclusions. The resulting proposal for test setup is based on the parameters related to solid timber and glulam, also taking into account the test model for CLT. The recommendations derived in this publication can be included in standard EN 16351 or EN 408. In addition, there is a plan to conduct a comprehensive test considering the above-described investigation into CLT by tension perpendicular, allowing further discussion on the dependence between strength and the above-mentioned factors.

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MONITORING OF THE SOIL-CEMENT PILES BUILDINGS SETTLEMENTS

In the paper it is discussesed new experimental and theoretical developments on soil-cement pile produced by drilling-mixing method. The biggest focus were made on field samples and full scale tests of the soil-cement piles, long term geodetic observations of the constructions settlements. Those experiences were implemented in Ukrainian normative documents. Geodesic monitoring of nine-ten storey building (with soil-cement piles length 6 m and diameter 500 mm in wet loess foundations) have shown that the settlements of its sections is much smaller than the ultimate values in national requirements.

Keywords: *wet loess soil, pile, soil basement, foundation, soil-cement, mixing technology, strength, settlement.*

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МОНІТОРИНГ ОСІДАНЬ БУДІВЕЛЬ НА ҐРУНТОЦЕМЕНТНИХ ПАЛЯХ

Розглянуто нові експериментально-теоретичні розробки авторів з питань бурозмішувального методу виготовлення ґрунтоцементних паль, дослідних натурних виготовлень і випробувань ґрунтоцементних паль, тривалих геодезичних спостережень за осіданнями будівель на ґрунтоцементних палях, які було враховано при складанні сучасних Державних будівельних норм України. З'ясовано, що ґеодезичні спостереження за дев'яти-десятиповерховим житловим будинком на ґрунтоцементних палях довжиною 6 м і діаметром 500 мм за умов замоклих лесових основ показали, що осідання всіх його секцій значно менші за допустиму нормами величину.

Ключові слова: *замоклий лесовий ґрунт, паля, ґрунтова основа, фундамент, ґрунтоцемент, бурозмішувальна технологія, міцність, осідання.*

Introduction. Volume of piles using in modern construction is constantly increasing. Therefore, using of new ways to improve its construction and manufacturing methods is important problem for today. Boring piles are the most commonly used type of piles in the world because of its technological efficiency and production convenience at the construction site. But, concrete boring needs additional technical measures to use it in different geotechnical conditions. For example, when the soil does not hold borehole walls then casing pipes are used, but even those measures does not fully protect the well from breakthrough by movable soil. Then drilling should be protected by clay mud which circulates in the well, brings out ruined soil, and at the same time supports walls in a stable position.Such measures are complicating production of the boring concrete piles, there additional time and costs are required.

Development of the soil cementation boring-mixing method during last few years has led to the new boring piles arising – soil-cement piles. It has all advantages of the bored piles, but in the same time problem of the borehole walls stability is completely excluded in the any kind of the geotechnical conditions [1 – 7].

Analysis of recent sources of research and publications. Known data [2 – 12] indicates that physical and mechanical properties of the soil-cement piles in the different places are quite clos for piles in the same soil conditions and with the same cement percentage content.

Experimental data of the Poltava geotechnical school [5 – 8, 13] show that mechanical properties of the soil-cement depend from lithology of dispersed soils; pH index of the water and of the water-soluble salts; cement percentage content and its quality; cement quality; water-cement ratio (W/C) and degree of soil-cement mixture compaction; presence of the steel reinforcement in the soil-cement; presence of the chemical additives.

In particular, laboratory and field researches and statically tests of the soil-cement physical and mechanical characteristics have shown that:

– cement content increasing from 5 to 50% leads to the soil-cement mechanical characteristics increasing by linear dependence, so, structural strength of soil-cement is possible to regulate by cement content even for the complete replacement of soil by cement in mortar;

– in soil with lower content of clay grains there arehigher mechanical characteristics, for production strong soil-cement sand with low content of clay grains is most effective;

– additives (sands and tails) using leads to soil-cement strength R and deformation modulus E increasing, so it is recommended to use additives and tails more efficient;

– soil-cement piles reinforced by steel frame allow to increase the carrying capacity by material to a value that exceeds the value of their carrying capacity by the soil.

Identification of general problem parts unsolved before. In weak soil massive, for example wet loess soil, due to their small deformation module, significant settlement of the buildings and structures base foundations is possible, even under the condition of the soil-cement piles use.

Therefore, in order to expand the normative base of the GCE design and increase its reliability, the method of objects settlement determining with strip rafts on the soil-cement piles. The most reliable option for solving this problem is comparing the calculated and measured long-term geodetic observations of the natural objects settlements values.

That is why the **goal** of this article is the geodetic observations results analysis in time for the building with strip rafts settlements on the soil-cement piles and the substantiation of the most reliable method of forecasting settlements of such objects.

Basic material and results. Soil-cement piles were used during house construction in the str. V. Kozak, 14 in Poltava (Fig. 1).

Building is placed on the territory with dense housing (Fig. 2). Because there are a lot of close buildings, pile driving is impossible.

Figure 1 – General view of the three-sectioned nine-ten storey house in the str. V. Kozak, 14 in Poltava (order of sections – left – right)

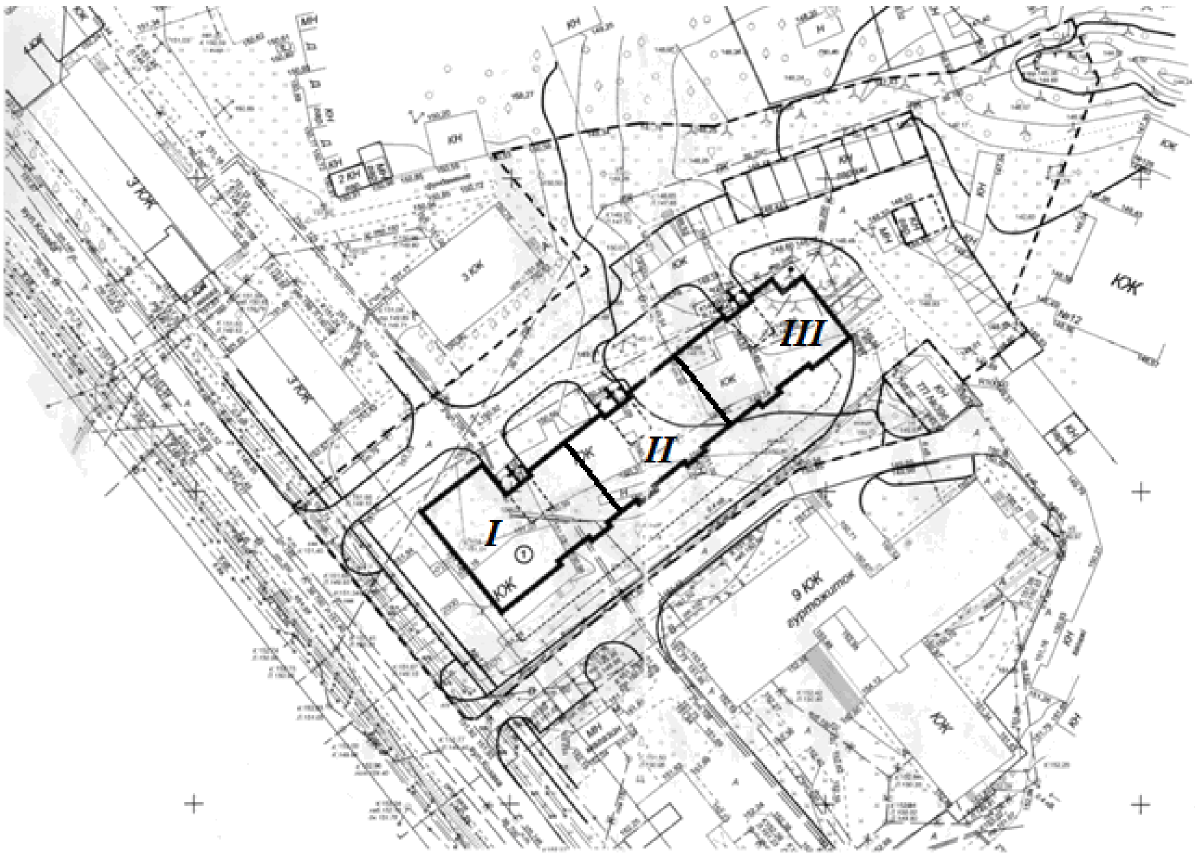

Figure 2 – The site of the three-sectioned nine-ten storey house on the str. V. Kozak, 14 in Poltava

In terms of geomorphology site is dedicated to Poltava loess plateau. Engineering-geologic section of the building site is on the Fig. 3.

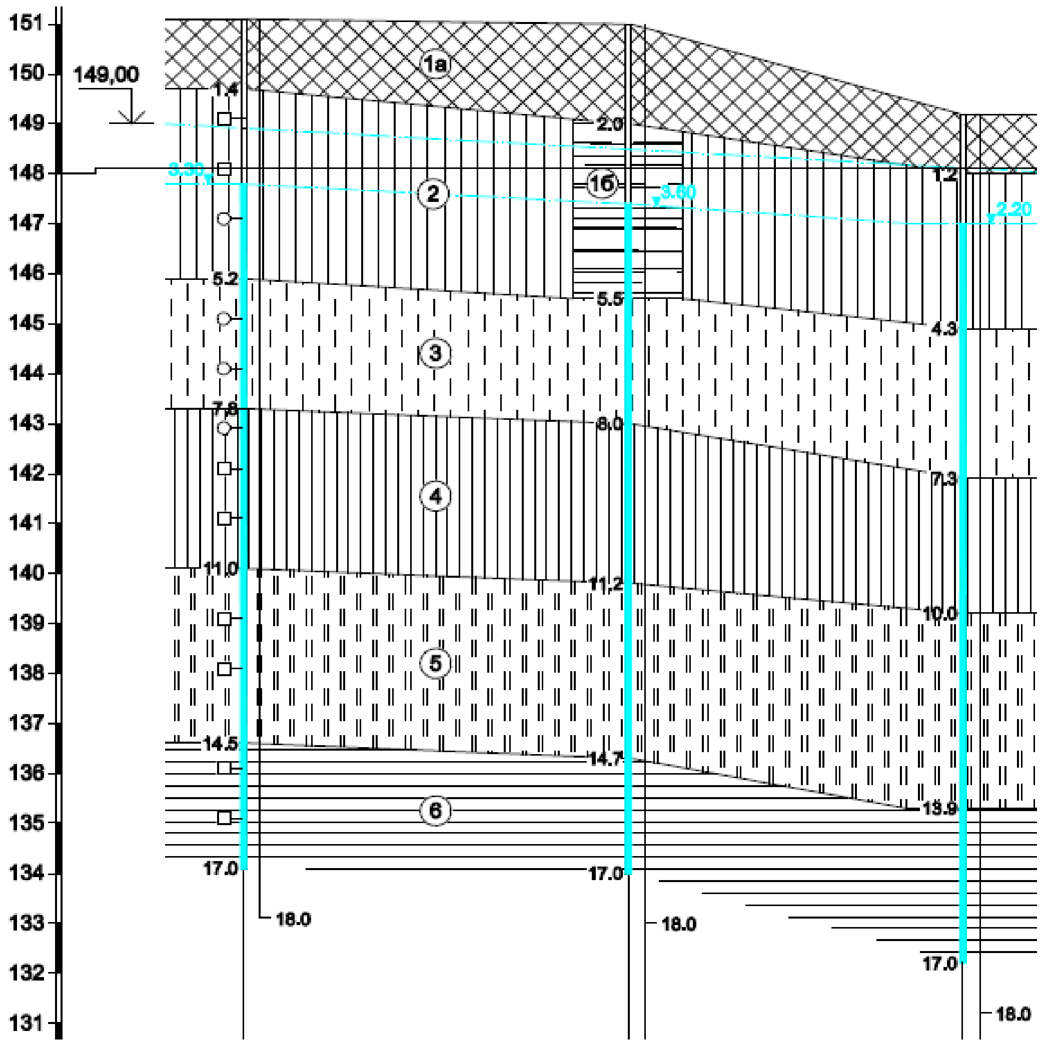

Figure 3 – Engineering-geologic section of the construction site

At the construction site normative values soils physical and mechanical characteristics are presented in the Table 1.

Soils 2 – 6 can serve as the natural foundations, their calculated physical and mechanical characteristics are given in the Table 2.

The level of the ground water is 2.2 – 3.6 m underground so bore-mixing technology efficient method of pile arranging use is possible in such conditions. As pile is almost below the groundwater level, hydro geological conditions are also favourable for normal soil-cement strength increasing.

Soil for piles is soil 3 (Tables 1 and Tables 2).

For sections I and II there were constructed reinforced soil-cement pile with length 6 m and a diameter 500 mm. Welded reinforcement space frames was used for piles reinforcement.

Piles in three rows are performed under the middle wall of the building, distance between the rows axes is 1000 mm, and distance between the piles axes in the one row is 1000 mm.

Piles in three (distance between rows axes is 1000 mm, distance between piles axes in row is 1100 mm) and two three (distance between rows axes is 1000 mm, distance between piles axes in row is 1000 mm) rows are performed under external walls of the building.

Under section III root piles were arranged. Separating screen between the sections was not found.

Table 1 – Normative physical and mechanical soils characteristics of construction site

Characteristics	Soil number					
	1	2	3	4	5	6
Moisture on the liquidity point W_L		0.37	0.32	0.36	0.30	0.43
Moisture on the border of plasticity W_p		0.21	0.20	0.21	0.19	0.23
Plasticity number I_p		0.16	0.12	0.15	0.11	0.20
Natural moisture w	0.18	0.24	0.30	0.28	0.25	0.24
Moisture at full water saturation W_{sat}		0.32	0.30	0.28	0.25	0.24
Liquidity index I_L		0.19	0.83	0.47	0.55	0.05
Water saturation ratio S_r		0.75	0.89	0.92	0.93	0.88
Gravity weight of the soil particles Y_s (kN/m³)		26.56	26.46	26.56	26.46	26.66
Gravity weight of the soil Y(kN/m³)	14.80	17.64	18.01	18.63	19.16	18.98
Gravity weight of the dry soil Y_d (kN/m³)		14.23	13.85	14.55	15.33	15.30
Gravity weight of the water saturated soil γ_{sat}		18.78	18.01	18.63	19.16	18.98
Gravity weight of the pushed soil in water γ_{sb}		8.98	8.72	9.18	9.65	9.68
Porosity n		0.46	0.48	0.45	0.42	0.43
Porosity ratio e		0.87	0.91	0.82	0.73	0.74
Filtration coefficient of m/day	0.40	0.20	0.40	0.20	0.50	0.10

Table 2 – Calculated physical and mechanical soils characteristics

Characteristics	Measur.	Soil number, values				
		2	3	4	5	6
Gravity weight of the soil, γ_n	kN/m³	17.54/18.68	17.91	18.53	19.06	18.88
Specific cohesion, c_n	MPa	0.015	0.010	0.016	0.011	0.054
Angel of the internal friction, φ_n	deg	23	28	25	28	19
Deformation modulus, E	MPa	8.5/4.5	7.0	12.0	11.0	21.0

Permanent monitoring of building by geodesic leveling settlements was used to determine actual settlements of the building foundations. On the building sections there were installed leveling marks in special places. Observation of building settlements started in June 2013 after the construction of the building cap, where geodetic marks had been established.

Observations continue by present. Building is taken in the exploitation on 1.02.2015. Scheme of the geodetic leveling marks is shown on the Fig. 4.

During the observations volumes of work were measured (masonry, floors, loads presence on the floors and so on). Behind them linear load on grillage and unit pile were expected.

Graphics of the minimum, medium, maximum settlements in time for building marks are shown on the Fig. 5.

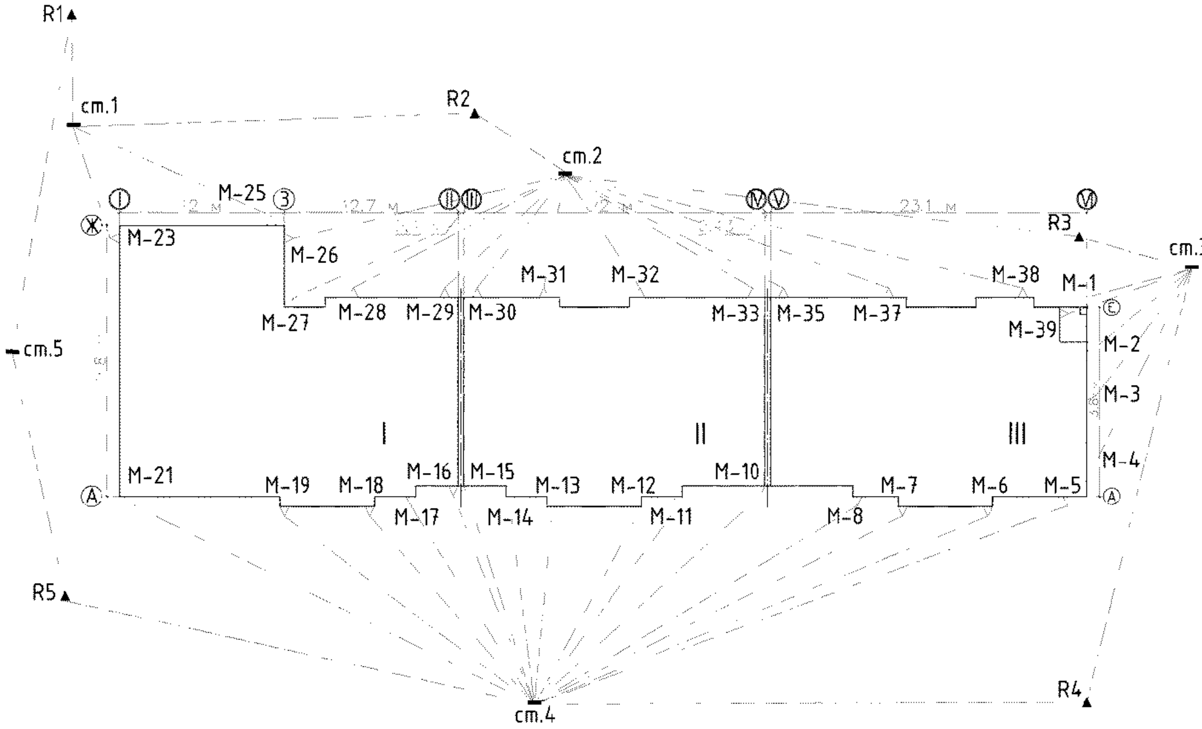

Figure 4 – Scheme of the geodetic leveling marks of nine-ten storey house on (str. V. Kozak, 14 in Poltava) I-III – building sections; M-1…M39 – wall leveling marks; R-1…R-5 – bench marks; st. 1… st. 5 – leveling intermediate stations

As a result of field tests were found that on 02.01.2015 settlements are:

– first section marks minimum settlement – 29 mm, average – 38,9 mm, maximum – 60 mm;

– second section marks minimum settlement – 32 mm, average – 45,8 mm, maximum – 67 mm;

– third section marks minimum settlement (with root piles) – 40 mm, average – 54 mm, a maximum – 70 mm;

– absolute settlement of all sections, and their relative uneven subsidence are less than the ultimate values in national requirements [1];

– there is tendency to pile foundation settlement stabilization;

– sections mutual influence that caused their uneven deformation is not recorded.

Conclusions. Thus, geodesic monitoring of nine-ten storey building (with soil-cement piles length 6 m and diameter 500 mm in wet loess foundations) has shown that the settlements of its sections are much smaller than the ultimate values in national requirements.

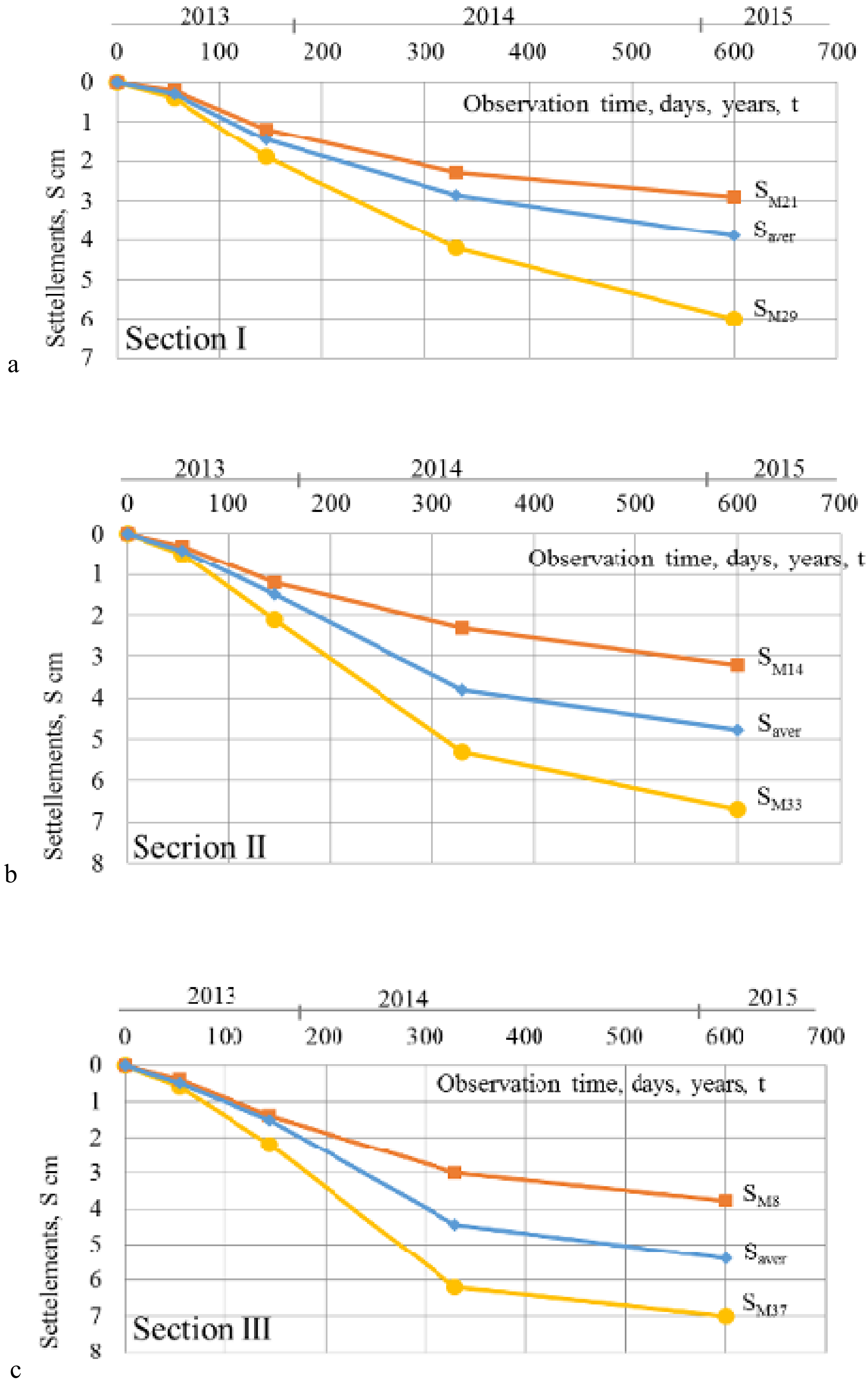

Figure 5 – Graphs of the geodetic marks minimum, average and maximum settlements in time (apartment building str. V. Kozak, 14: a – section I; b – II; c – III)

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CONSOLIDATION ZONES FEATURES FORMATION OF A GROUND AROUND SQUARE SHAPED PILES

Necessity to resolve the problem of concerning the analytical dimensions definition and constructing forms of compacted zones of ground around the lateral surface of square cross section pile in normal to pile axis direction are shown in this article. Various analytical methods of ground compacted zones radius determination around cylindrical elements driven into the ground considering its initial state were analyzed. Graph-analytical method of ground compaction zones construction around square cross-section piles was developed. Efficiency of the developed methodology and analytical expressions using possibility to determine the radius of compaction zone of a ground around the driven or jacked-in elements is confirmed by comparing obtained solutions with the experimental data. The developed methodology allows increasing the efficiency and reliability of piles and pile foundations use due to their rational design.

Keywords: *pile, bearing capacity, compaction zone of a ground, movement zone of a ground, cross-section, jacking.*

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ОСОБЛИВОСТІ УТВОРЕННЯ УЩІЛЬНЕНИХ ЗОН ҐРУНТУ НАВКОЛО КВАДРАТНИХ ПАЛЬ

Доведено доцільність вирішення питання щодо аналітичного визначення розмірів та побудови форми ущільнених зон ґрунту навколо бічної поверхні квадратної палі в нормальному до осі палі напрямку. Проаналізовано різні аналітичні методи визначення радіуса ущільненої зони ґрунту навколо циліндричних елементів, що заглиблюються у ґрунт з урахуванням його початкового стану. Розроблено графоаналітичну методику побудови ущільнених зон ґрунту навколо паль квадратного поперечного перерізу. Ефективність розробленої методики та можливість використання аналітичних виразів для визначення радіуса ущільненої зони ґрунту навколо забивних або вдавлюваних елементів підтверджено шляхом порівняння отриманих рішень з експериментальними даними. З’ясовано, що розроблена методика дозволяє підвищити ефективність та надійність використання паль і пальових фундаментів за рахунок їх раціонального конструювання.

Ключові слова: *паля, несуча здатність, зона ущільнення ґрунту, зона переміщення ґрунту, поперечний переріз, вдавлювання.*

Introduction. The present-day constructions in large cities situation is often accompanied by negative engineering-geological conditions and processes. They require from the constructors to use safety and efficiently foundations and in most cases it can be piles. However in spite of their fairly significant distribution a considerable volume of factors are not currently studied. They affects diving conditions and pile working in pile foundations. The construction experience shows that diving of square cross-section piles in a pile group at a distance up to 4d leads to a pile turning around its axis by 45° during pilling (Fig. 1). It is due to the uneven distribution of stresses and as a consequence uneven ground base compaction around the square pile.

Figure 1 – Square piles turning around its axis by 45° during pilling

Moreover, the numerous numbers of analytical calculation methods for determining pile foundations base deformations under the action of vertical compressive static load does not allow to obtain reliable results in comparison with experimental data. In this case the deformation characteristics of the soils given in the reports of engineering surveys are proposed to use during calculations. They do not consider the real state of the ground base, which is associated with the shape and dimensions of piles cross section, its plan position, the angle of rotation around the vertical axis passing through the cross-section of the pile and the sequence of the pilling.

Analysis of recent researches and publications. Construction practice shows that the foundations installation without removing soil is accompanied with compaction zones formation. There is a change of natural physical and mechanical soil characteristics within these zones. The creation of such zones makes it possible to improve the base of foundations, enhance their bearing capacity and increase efficiency. The dimensions of these zones, their numerous and qualitative indicators should be considered in the calculation methods for determining bearing capacity and deformations of jacked or driven piles and pile foundations. However, confirmation of the developed methodology results for soil compaction zone sizes determination around jacked or driven piles can be based only on data obtained by experimental research results, for example, after digging or probing. Thus, stress distribution character in sandy base of circle cross-section pile models is given in [9, 12]. The authors of works [2, 3, 10, 11, 13] investigated the deformations of sandy soil in the base of jacking and driving piles, and they have also established the difference between them [4]. Experimental semi-field researches of pile-stamp models with different cross-sections in clay soils are given in [7]. And the peculiarities of the compaction and deformation soil zones formation around the piles under the action of horizontal loading are considered in [14 – 16].

The analytical expressions for determining the size of the soil compaction zones around a cylindrical element divined into the ground are given in [1, 5, 6]:

$$R_1 = \frac{r}{2} + \frac{r}{2}\sqrt{\frac{3(4+3e)}{e}}\ , \tag{1}$$

$$R_2 = r\left(\sqrt{4+6\frac{m}{n}\text{-}1)}\right), \tag{2}$$

$$R_3 = \sqrt{\frac{2\pi \cdot r^2\left(\frac{W\rho_d}{\rho_w} + \frac{\rho_d}{\rho_s} + V_{c.a.}\right)}{1-\left(\frac{W\rho_d}{\rho_w} + \frac{\rho_d}{\rho_s} + V_{c.a.}\right)}\frac{1}{\pi} + r^2}\ , \tag{3}$$

where r – average pile radius; e – soil porosity coefficient; m – relative content of solid particles; n – soil porosity; W – soil moisture; ρ_d – dry density; ρ_w – water density; ρ_s – soil skeleton density; $V_{c.a.}$ – a volume of compressed air between solid particle aggregates and water, $V_{c.a.}$ is accepted ≈0.

Selection of previously unbundled parts of the general problem that the article is devoted to. With a help of formulas (1), (2), (3) it can be determined the radius of compaction zones around the diving into the ground cylinder, or in other words, of the circular cross-section piles. The numerous volumes of jacked or driven piles with significant distribution in construction practice have a square cross-section. So the use of formulas (1), (2), (3) does not allow to obtain the results similar to real conditions, which is not correct.

The purpose of the work is to develop a method for determining sizes and construction forms of compaction zones around the square cross section pile and to perform a comparison of the obtained results with the experimental research data.

Main material and results. For the development of the soil compaction zone radius determination method around the square cross-section pile the symbols are introduced: $R_{i,j}$ is a radius of the compaction zone which directions correspond to the form of the compaction zone obtained experimentally (Fig. 2) [8], and determined in accordance with average radius r_i (i = I, II, III, ..., n), by the formula j (j = 1, 2, 3).

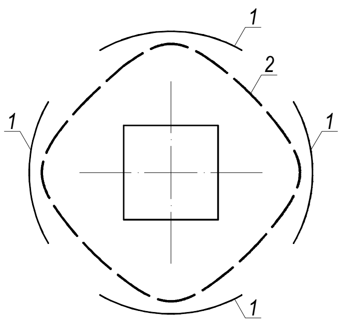
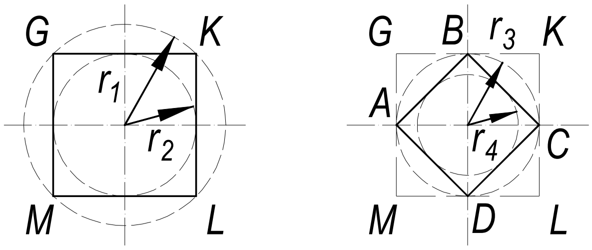


Figure 2 – Compaction (1) and deformation (2) zones of ground around experienced piles determined in field conditions

To construct ground compaction zones around the square cross-section pile the following algorithm is used:

1. Separate calculated elements are allocated in the piles cross-section (Fig. 3).



a) b)

Figure 3 – Calculated elements for determining average radii r_I (a) and r_{II} (b)

2. Average radius r_i is defined for each element as the average between the described and inscribed radius.

3. Compaction radius of ground $R_{i,j}$ is determined according to formulas (1), (2), (3) with a help of average radius r_i.

4. Line segments $R_{i,j}$ are plotted from the each element center of gravity (Fig. 4).

5. The tips of all radii $R_{i,j}$ are connected. Zone of compacted ground around the piles lateral surface is obtained as a result.

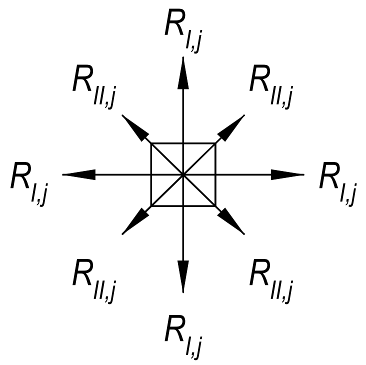


Figure 4 – Compaction zone radius directions around square cross-section pile

Using presented algorithm and formulas (1), (2), (3) there is ground compaction zones around square cross-section pile constructed and compare the obtained results with the experimental data compared given in [8].

Output data: a square cross-section pile with a side size of 300 mm. Pile was driven on a depth of 4m in medium density fine sands, low degree of water saturation with the following characteristics: ρ = 16.28 kN/m^3, ρ_d = 15.6 kN/m^3, e = 0.671, W = 0.043 .

The radii of ground compaction zones around a square cross-section pile in directions shown in fig. 4 are determined. Two clearing elements are allocated in the pile cross-section. The first is square GKLM element (Fig. 3, a) with a side of 300 mm. The second is square ABCD element (Fig. 3, b) which vertices match with each side centers of square GKLM. For the first element the reduced radius r_I was calculated as the arithmetic mean of the described and inscribed radius (r_1, r_2) around the square GKLM (Fig. 3, a). Then the values of the compacted zone radii are calculated by the expressions (1), (2), (3): $R_{I,1}$ = 56 cm, $R_{I,2}$ = 47 cm, $R_{I,3}$ = 40.4 cm.

Also define reduced radius r_{II} is defined as arithmetic mean of the described and inscribed radius (r_3, r_4) of ABCD element (Fig. 3, b) and compaction zones radii are $R_{II,1}$ = 39.6 cm, $R_{II,2}$ = 33.3 cm, $R_{II,3}$ = 28.6 cm.

Ground compaction zones around square cross-section pile in horizontal direction are shown in fig. 5.

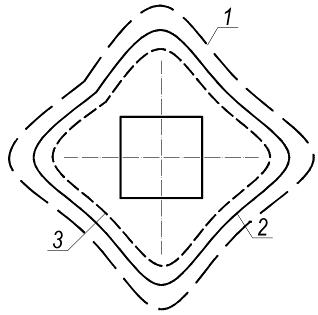


Figure 5 – Compaction ground zones (1, 2, 3) around square cross-section pile determined by the corresponding formulas

In order to compare the analytical calculations values with the experimental data, the compaction zones constructed analytically (Fig. 5) on zones of movements and compactions of a ground based on field investigation results were imposed (Fig. 2). These zones were determined around the lateral surface of the square pile with the help of dynamic probing tests, cone penetration tests, cutting ring and visually after the digging [8]. The overlay results are presented in Fig. 6.

Analyzing Fig. 6 it is possible to note that the character of ground compaction zones constructed analytically are similar to the displacement zones obtained experimentally in field tests. The dimensions of the compacted ground zones determined experimentally are less than analytical ones determined by the formula (1) up to 26%. The radius of the compacted soil zone determined experimentally coincides with the analytical determined by the formula (2), which at the diagonal points coincides with the experimental deformation zones and is by 7% higher than the experimental deformations in the normal direction of the pile side. Expression (3) allows determining the maximum values of the ground deformation zone in the pile side normal direction, coinciding with the experimental data, but they are smaller than experimental compaction and deformation zone of a ground at diagonal points up to 20%.

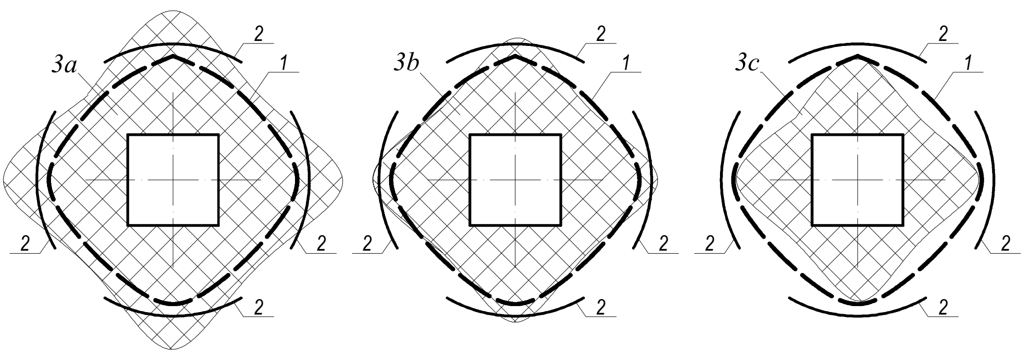


Figure 6 – Comparison of analytical calculations and experimental data:
1 – deformation and 2 – compaction zones defined in field conditions;
3a, 3b 3c– analytically constructed compaction ground zones
with a help of formulas (1), (2), (3) respectively

Conclusions

1. The analytical method of a ground compaction and deformation zone sizes determination around the lateral surface of square cross-section pile was developed. Its use allows obtaining analytical results close to the experimental ones.

2. Analytical values of the ground compaction zone radius around the piles lateral surface in a comparison with the practical results show their satisfactory convergence with maximum deviation increase to 26% using formula (1) and minimum deviation to 7% using formula (2).

3. Comparison of existing analytical methods with experimental data shows the possibility of their employment for analytical compaction or deformation ground zone radius determination around the square pile lateral surface.

4. Compaction zones construction in soils with different porosity coefficient allows placing piles more efficiently in pile foundation, improving analytical method for determining the stresses type distribution around jacked or driven piles with square cross-section, improving their calculation method by the second group of boundary states and increasing their efficiency.

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LABORATORY EXPERIMENTAL RESEARCH OF LOADING FORCES DEVELOPMENT ACTING ON THE SIDE SURFACE OF THE PILES

The forces of negative friction or loading forces of friction on the piles develop due to additional deformations of near pile array and they cannot be more force of resistance on the side of pile in rest, that is formed due to own weight of the ground founding. The paper presents the results of laboratory experimental research of loading forces of friction acting on the lateral surface of the piles without taking into account the vertical load (being relatively at rest) and provided an analysis of methods for detection of negative friction forces (loading forces) acting on the lateral surface of the pile, with the appointment of their load-bearing capacity. The researches have confirmed the theoretical statement about the equality of the loading forces of friction to ground resistance forces under the action of the torque load but not the pulling load, as it is suggested in the modern standards.

Keywords: *experiment, pile, rest, soil subsidence, side surface, loading friction force.*

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ЛАБОРАТОРНІ ЕКСПЕРИМЕНТАЛЬНІ ДОСЛІДЖЕННЯ РОЗВИТКУ ДОВАНТАЖУВАЛЬНИХ СИЛ, ЩО ДІЮТЬ ПО БІЧНІЙ ПОВЕРХНІ ПАЛЬ

Установлено, що сила негативного тертя, або довантажувальна сила на палю, розвивається за рахунок додаткових деформацій навколо пальового масиву і не може бути більшою від сили опору по бічній поверхні палі в стані спокою, що формується за рахунок власної ваги ґрунтової основи. Наведено результати лабораторних експериментальних досліджень довантажувальних сил тертя, що діють по бічній поверхні модельної палі без урахування вертикального навантаження (умовно у стані спокою), та аналіз методик визначення сил негативного (довантажувального) тертя, що діють по бічній поверхні паль, при призначенні їх несучої здатності. Виконаними дослідженнями підтверджено теоретичне положення щодо рівності довантажувальних сил тертя і сил опору ґрунту під час дії крутильного моментного навантаження, а не висмикувального навантаження, як запропоновано у сучасних нормах.

Ключові слова: *експеримент, паля, стан спокою, просідання ґрунту, бічна поверхня, довантажувальні сили тертя.*

Introduction. Designing buildings and structures on pile foundations, the force of loading friction should be considered in cases where the conditional rate of soil deformation of the mass around the pile can exceed the rate of the pile foundation sediments which, as a rule, is in the base of structurally unstable soil as well as in other cases of the same ground layer deformations.

The issues of the loading forces of friction development in the pile foundation are discussed in work of researchers from our country and foreign countries: Brom Beng B., Felenius N., Krafford K., Endo M., Berrum L., Johansen I., Karisel J., Dalmatov B.I., Lapshin F.K., Rossikhin Yu.V., Grigoryan A.A., Zaretsky Yu.K., Morozov V.N. and others.

Except theoretical researches, there are practical ones investigating the forces of negative friction by means of the field methods, that are most reliable. It is possible to mark the use of tenzopile, and also normative and patented methods and methods of friction acting forces determination on the lateral surface of foundations and piles. However, offered field tests with the use of the considered methods or are labour intensive at application of tenzopile or the forces of negative friction determined on the basis of pile tests on the action of the pressing and pulling out loads, and the equality of the soil resistance forces along the lateral surface of the pile is assumed.

Analysis of recent sources of research and publications. The analysis of recent publications [1, 2] shows the great interest of researchers in the development of negative forces which are determined as loading forces of friction in the latter norms [3]. Existing norms [3 – 4] and «Guide» [5] regulate the detection and calculation of negative friction forces in determining the load bearing capacity of piles when they are tested with compressive and pulling out loads. This position is incorrect because the development of the loading forces of friction on the lateral surface of the piles is due to deformations s_{sl} of the soil mass where the change in the stress state of the base can be neglected [6], and in the case of application to the pile the experimental vertical load, the intense state of the soil base around the pile changes significantly. It points out to the fact that the normative method [3] of setting the calculated load on a pile in the collapsible soils does not consider the peculiarities of the friction loading forces formation in the soil and as a result it is incorrect.

Identification of general problem parts unsolved before. In the paper [6 – 8] Samorodov A.V. proposes the theoretical idea about the equality of the loading force of friction P_n and the strength of the resistance at rest T_0 on the lateral surface of the piles (Fig. 1) revealed previously [7 – 9]. T_0 is the resistance force acting on the pile of the torque load M formed depending on the stress state of the soil mass around the pile due to its own weight when there is not the vertical load on the pile which is conditionally at rest, kN.

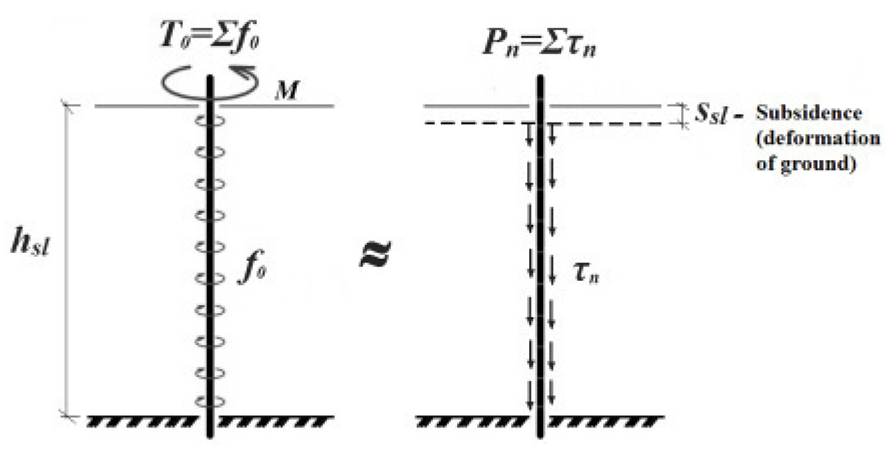

Figure 1 – Schemes of distribution of the resistance forces of f_0 and loading forces of friction τ_n on the lateral surface of the pile

To confirm the theoretical position: $P_n \approx T_0$ (Fig. 1), laboratory experimental researches with similar parameters of the previously investigated system for the determination of T_0 by means of reeling torque load M [9] were performed.

The purpose of this work is to do laboratory experimental research to confirm the theoretical position: $P_n \approx T_0$ (Fig. 1).

Basic material and results. As an experimental installations (Fig. 2) it is used a specially equipped tray in the form of a metal barrel of height H = 900mm, Ø 560mm and thickness of 10 mm, in the lower part of which was a «double» bottom with a gap (h = 100 mm), which was filled with a rubber air «pillow».

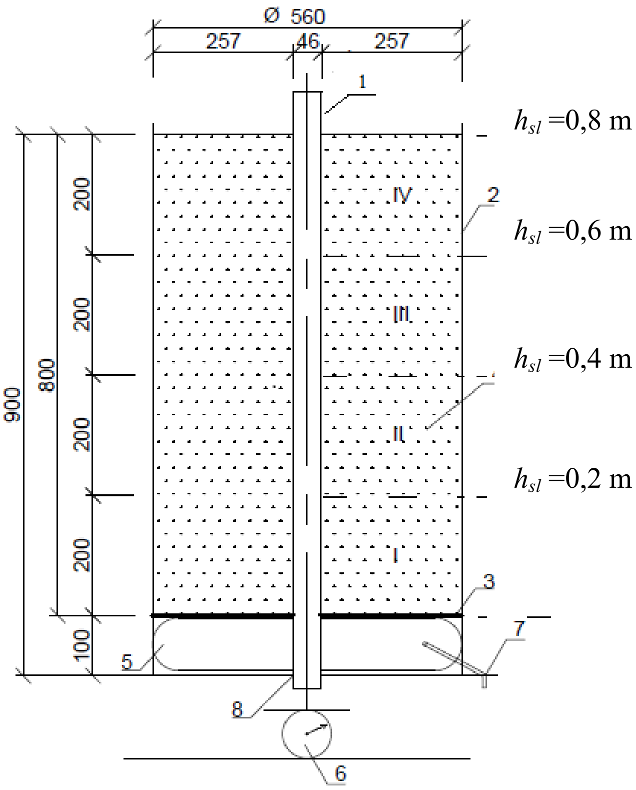

Figure 2 – Plan of installation during the experiment to determine the loading forces of soil friction P_n on the lateral surface of the model pile at different height of filling h_{sl}:
1 – model fluoroplastic pile Ø46 mm; 2 – tray; 3 – partition wall (flake board); 4 – sand (fine, dry, homogeneous sand: $\gamma \approx$ 15 kN/m^3, $\varphi \approx 30°$); 5 – rubber air «pillow»; 6 – measuring instrument (electronic scales, model: AXIS A20); 7 – hole for hose used for inflating-blowing out «pillow»; 8 – throughput system (rubber pressure seal)

A model of fluoroplastic pile with similar parameters of the previously investigated system was used to determinate T_0 by means of the torque load M [9]: Ø 46 mm and a height of 1000 mm, which has been made basing on the theory of similarity [10] on a scale of M ≈ 1:10, that is the relator of two numerical values of the model pile, such as the length of the pile L and its diameter D, was similar to the ralator of the same numerical values for a real pile (Fig. 3, *a*). Fine, dry, homogeneous sand was used as filling (γ = 15 kN/m^3, φ = 30°). The average grain of sand size is from 0,20 to 0,25 mm. The tray has been filled with the sand evenly, not in large portions, like rain to the required filling height h_{sl} = 0,2; 0,4; 0,6 and 0,8 m. After complete filling before the beginning of the first series of experiments, the installation was kept in the design position for a minimum of 30 minutes.

The distance between the pile and the walls of the tray is approximately 257 mm.

Preparation for the experiment and its implementation included several stages (Fig. 3):

– the modeling pile was installed in the design vertical position by means of free hanging, where the lower end of the pile was passed through the entire buffer on the weights (Fig. 3, *b*) through the special hole in the bottoms where the pile leans on the scales surface (Fig. 3, *c*);

– the sandy ground was filled to the height of h_{sl};

– simulation of the touching down process of the entire thickness h_{sl} by a value s_{sl} = 100 mm was blowing out air from the «pillow». The beginning of the soil mass movement at each loading stage was fixed visually;

– additional weight of the P_n pile was recorded due to loading forces of the soil τ_n on the lateral surface of the pile. Electronic transmission scales (*AXIS A20* model) were used to transfer the force (Fig. 3, *c*).

Thus, a series of experiments were made to determine the loading forces of the soil friction on the lateral surface of the model pile at the height of the fillings: h_{sl} = 0,8 m (h_{sl}/D =17,39); h_{sl} = 0,6 m (h_{sl}/D =13,04); h_{sl} = 0,4 m (h_{sl}/D = 8,70); h_{sl} = 0,2 m (h_{sl}/D = 4,35). For each height of the filling h_{sl}, seven experiments were made, they allowed to do qualitatively statistical processing of the private values of the experimental data according to [11]. The results of the experiments are summarized in Table 1, which also gives comparison of the results of determining the resistance forces at the pile side surface at the rest T_0 obtained earlier by PhD Tabachnikov S.V. [9].

a) *b)* *d)*

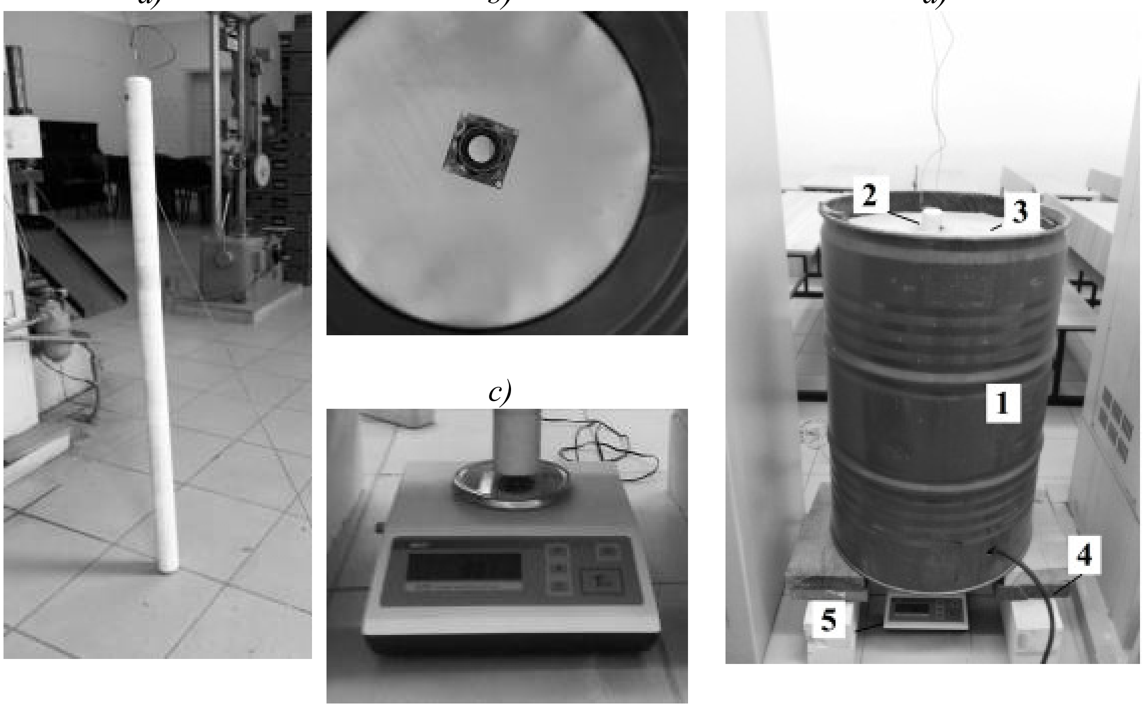

Figure 3 – Photo during the experiment:

a – model pile; *b* – a through hole at the bottom of the installation with a rubber gland seal; *c* – electronic scales (model *AXIS A20*); *d* – general view of installation: 1 – tray; 2 – model pile; 3 – sand; 4 – hose, used for in flatting-blowing out «pillow»; 5 – electronic scales

Table 1 – Comparison of the friction P_n loading forces values and the resistance forces at the side surface at the rest T_0

Height of filling h_{sl} *(z)* (m)	Correlation h_{sl}/D *(z/D)*	P_n (kgf)	T_0 (kgf)	Discrepancy, %
0,2	4,35	0,648	0,466	28
0,4	8,70	1,828	1,819	0,49
0,6	13,04	4,229	4,100	3,1
0,8	17,39	7,501	7,085	5,55

For this experiment, the friction P_n loading force dependence of loose ground (sand) on the relative value of h_{sl}/D is given in the form

$$P_n = 0{,}0245 \cdot (z/D)^2 + 0{,}002 \cdot (z/D) + 0{,}0486 \quad (1)$$

and is shown graphically in Figure 4, where the plot of the dependence on the ground resistance force at rest T_0 is also shown [9].

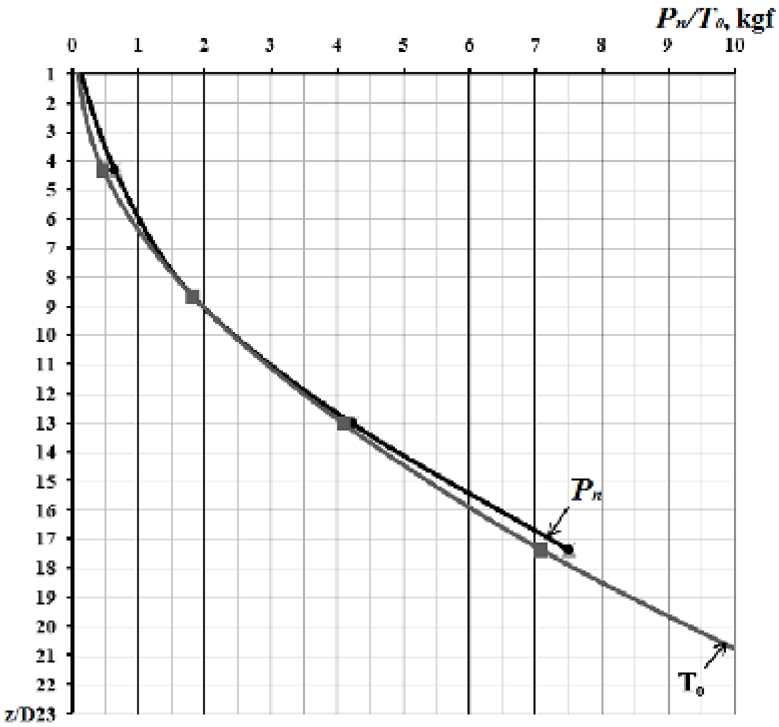

Figure 4 – Graphs of dependencies of loading forces of friction P_n and ground resistance forces at rest T_0 from h_{sl}/D *(z/D)*

The results of the experiment confirm the theoretical statement about the equality of forces: $P_n \approx T_0$ (Fig. 1), that is the loading forces of friction P_n must be determined with the torque load M but not the pulling load F_{du}, as has been proposed in modern standards [3]. During the additional real experiments it allows to reduce the effect of the potential loading forces of friction P_n on the lateral surface of the piles in structurally unstable soils, and, as a consequence, to increase the bearing capacity of the piles F_d under difficult conditions.

At Figure 5 the results of calculation are evidently presented: the forces of negative friction P_n [12] using different to methodologies [3 – 5], other things being equal, where higher adopted – «methodology KNUCEA» approach is offered .

According fig. 5 application of different methodologies gives considerable distinction in the values of the loading (negative) forces: the value of P_n obtained by the proposed methods of «KNUCEA» is in 2 times less than in the methods of the «Guide to the Design of Pile Foundations» [5] and «Ukrainian SBN» [3] and in 1,5 times less than the «Russian methods of SP» [4].

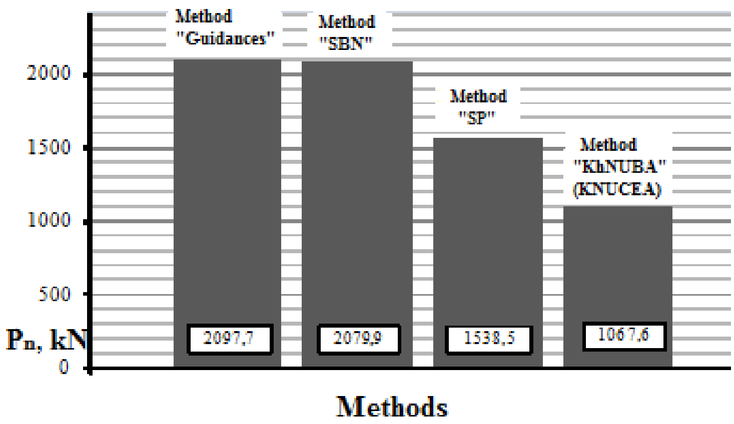

Figure 5 – The values of loading forces of friction P_n

Conclusion. On the basis of experiments and the obtained results the following conclusion can be done:

1. The conducted laboratory investigations confirmed the theoretical statement about the equality of the loading forces of friction on the lateral surface of the piles P_n and the resistance of the soil T_0 under the torque load: $P_n \approx T_0$, that is the loading forces of friction P_n must be determined by the torque load M but not the pulling load F_{du}, as it is proposed in the modern standards [3].

2. The developed methodology (method «KNUCEA») for determinations of resistance forces on the lateral surface of the piles in a state of rest T_0 [8] allows to adapt it for determination P_n, as concrete example [5] shows considerable distinction in the values of loading forces of friction P_n at application of different methodologies: the value of P_n, obtained by the method of «KNUCEA», is in 2 times less than in accordance with the methodology of the «Guidances» [5] and in 1,5 times less than the «Russian methods of SP» [4].

3. Preliminary results of the research indicate the possibility of influence reasonable reduction of the potential loading forces of friction P_n on the lateral surface of the piles in structurally unstable soils which during the additional real experimental substantiation will allow to increase the bearing capacity of the piles F_d on the compressive loads and as a consequence provide certain economic effect.

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FEATURES OF STREET-ROAD NETWORK MODERN DESIGNING AND RECONSTRUCTION IN CITIES

The main problems associated with the organization of traffic and pedestrians movement in modern cities are determined. The world experience in designing and reconstruction the street-road network of cities is considered. The factors determining current trends and features of designing and reconstruction of street-road network in Ukraine cities are identified. The priorities of street-road traffic affecing its formation and development are considered. The main goals to adap urban construction and reconstruction projects to the change of priorities, which took place in the theory of transport planning, are formulated.

Keywords: *street-road network, transport planning, priorities of traffic, optimization of city planning structure.*

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ОСОБЛИВОСТІ СУЧАСНОГО ПРОЕКТУВАННЯ І РЕКОНСТРУКЦІЇ ВУЛИЧНО-ДОРОЖНЬОЇ МЕРЕЖІ МІСТ

Визначено основні проблеми, пов'язані з організацією руху транспорту і пішоходів у сучасних містах. Розглянуто світовий досвід проектування й реконструкції вулично-дорожньої мережі міст. Виявлені фактори, які визначають сучасні тенденції та особливості проектування і реконструкції вулично-дорожньої мережі населених пунктів України. Розглянуто й обґрунтовано пріоритети вулично-дорожнього руху, що впливають на її формування і розвиток. Сформульовано основні цілі, які мають бути досягнуті при пристосуванні проектів будівництва та реконструкції міських вулиць до зміни пріоритетів, котра відбулась у теорії транспортного планування.

Ключові слова: *вулично-дорожня мережа, транспортне планування, пріоритетність руху, оптимізація планувальної структури міста.*

Introduction. The street-road network of the settlement (SRN) is one of the most stable and important city-building elements, so it must be calculated for a long period of using without expensive major rebuildings and reconstructions.

The current state of urban traffic sets the task to architects, roads designers and workers of transportation sector, which solution depends not only on characteristics of public transport, but also on city development.

Analysis of recent sources of research and publications. Nowadays, more and more researchers of transport planning use the term «Automobile Dependency» [1, 2]. During the last decades the international coordination is carried out in the field of transport, roads and urban development. The largest international organization that conducts such coordination is Permanent International Association of Road Congresses (PIARC).

Issues of road infrastructure, urban transport systems development are systematically considered in documents of profile committees PIARC [3, 4]. In 1995 PIARC conducted the specialized XX World Road Congress, devoted exclusively to problems of urban transport planning. In the United States, at the state level, acts were adopted, where special attention was paid to the organization and safety of pedestrian movement: Intermodal Surface Transportation Act of 1991 (ISTEA), Transportation Equality Act of 21th Century.

In the works [5 – 7] the classification of the main geometric structures of the street-road network is given, their influence on the parameters of transport systems functioning of is evaluated and recommendations on the use of the city territory for different planning schemes are given. But such classification based on width and operational qualities cannot fully reflect all the processes taking place on the streets. The street should be not only a city transport artery, but also the place of human interaction. During the organization of street space should be guided by a number of requirements related to activation of social and economic functions: improving the quality of life, mobility and activity of the inhabitants.

Identification of general problem parts unsolved before. The rapid rate of motorization in Ukraine creates a new situation in urban designing. A few years ago, the focus was on improving traffic conditions for road transport (increasing the capacity of SRN and the speed of the connection) and purely technical aspects of resolving this issue. Now problems of providing conditions for pedestrian traffic and especially for the low-mobility groups of the population are getting worse.

Problem formulation. The purpose of this study is to summarize the world experience of designing and reconstruction of urban street-road network and to develop proposals for its improvement in accordance with the change of priorities occurred in the theory of transport planning.

Basic material and results. The problems of reconstruction of the transport system and the street-road network are very significant. Transportation directly affects labor and cultural-domestic activity of the population, causing technical and social progress of society. Highways and street-road network make up the city framework, forming its planning structure. Moreover, transport communications is the most stable element of this structure, which maintains its functional significance, even with global changes in the organization of urban transport and life of the population. In recent decades the problems of transport in large cities have become much more complicated because of the increasing number of cars and their active using for labor, cultural, domestic and recreational travels.

In Ukraine, for example, the total number of passenger cars approached 2.5 million against 0,55 million in 1991 [8]. The level of car ownership in big and largest cities of Ukraine is 85 – 275 cars per 1,000 inhabitants. The similar rate in urban areas of developed foreign countries is 450 – 700 cars per 1,000 inhabitants. Even with the fact that the level of motorization in Ukrainian cities is low compared with the motorization level of developed Western countries, transport problems in Ukrainian cities are acute and require urgent solution [9].

In the city centers at rush hour the speed of road transport is reduced to 10 – 15 km/h. Time expenditures for travel are increasing, the average time for travel from place of residence to work exceeds 60 minutes at normal rates for 90% of passengers no more than 40 minutes. The study revealed a number of modern cities street-road network problems, the main ones are following (see Fig. 1).

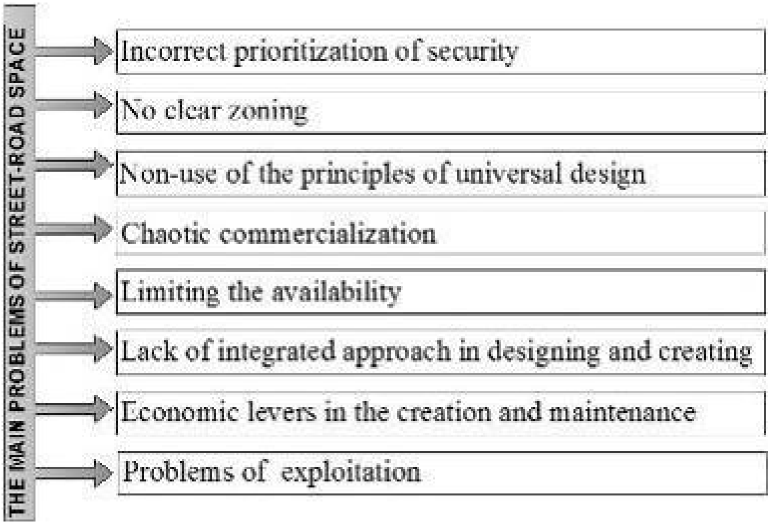


Figure 1 – The main problems of street-road space

Incorrect prioritization of street space security. Space security depends on many factors: clear and intuitively understandable boundaries of spaces for different activities (walking, moving by bicycle or car, rest in cafes or places for recreation etc.); high concentration of residents on streets, which gives a sense of unity with the city community; good lighting in the dark and so on. Still street lighting is calculated considering the value of traffic. Instead, sidewalk lighting is often relegated to the background, while lighting quality is one of the street safety keys.

No clear zoning of street space. The boundaries of private and common spaces in Ukrainian cities are still not clearly defined and there is not any graduation. The boundaries can be both physical (building front, fence, green plantations, etc.), and «mental» (paving, lighting, etc). Because of the lack of clear zoning, pedestrians cannot feel safe even on the sidewalks, car drivers can sometimes both park and ride in pedestrian areas.

Non-use of the principles of universal design. The universal design is the design of objects, environment, programs and services appointed to make them as fit for the use of all people without the need for adaptation. Universal design does not exclude auxiliary devices for specific groups with disabilities where they are necessary. Its main principles are equality and accessibility, flexibility for simple and intuitive use, affordable presented information, the right to make mistakes, the need for low physical effort, the presence of the desired size, place, space.

Chaotic commercialization of street space. City streets are filled with institutions for business, seasonal and non-sanctioned trade, promotional materials, etc. Thus, the quality of urban space and its visual perception sharply falls. At the same time, it contributes to the reduction of the efficiency of promoting goods and services, the development of the shadow economy, the establishment of «society of consumption» principles and the reduction of mental health of people (irritation due to impossibility of free passage, feeling of imperfection, etc.).

Limiting the availability of street space. In the streets of most cities there are no special facilities for people with limited mobility, the design of improvement elements often is not ergonomic, the crossing of the road often need to use an underground passage, inconvenient for the elderly and low-mobility groups of the population.

Lack of integrated approach in designing and creating of street space. A large number of stakeholders in the using of space, variety of landowners create chaotic placement of improvements and other elements of street space. Often there is no street navigation, no signing.

*Economic levers in the creation and maintenance of street space***.** In market conditions, the city is not able to maintain a high quality of free space, based in Soviet times. It is necessary to introduce new scenarios for the development of such places using investments from different business structures.

Problems of street space exploitation. When operating the streets, normative acts regulating the maintenance of street space are often violated. Garbage disposal system is ineffective, poorly performed work on the current paving repair, etc.

On the second step there is cycling transport, which has the same advantages and problems as the pedestrian, but occupies a separate place in the pyramid because it allows to overcome much larger distances (effective radius of bicycle use is 5 – 7 km) and needs parking spaces and, on separate streets, a separate infrastructure.

Also, with the development of urban traffic, the task of environmental protection revolts especially sharply. Protection from noise, vibration, air pollution of the city by harmful impurities, contained in the exhaust gases of the car are the most acute problems of modern cities.

High growth of city motorization, increasing traffic volumes on the streets, creating a network of high-speed roads and highways of continuous motion, rational organization and traffic management with the creation of the best conditions for its safety are the problems of transport in a modern city, without them the normal functioning of the city life is impossible [10, 11].

In recent years, the views on designing and reconstruction of the street- road network have undergone revolutionary are changed. The term «Automobile Dependency» got the following definition: automobile dependency is the total effect of a number of factors, which leads to a high level of car using and limits the possibility of using transport alternative modes. Another definition is automobile oriented transportation and land use patterns.

World experience shows that even investing heavily in the development of the road network, the solution of road transport services problems is complex; it is impossible to solve the problem of transportation in large cities by providing comfortable movement of cars. The best in terms of transport cities in the world (Copenhagen, Berlin and others) use the so-called pyramid of priority (see Fig. 2), which is advised to apply when making decisions in streets designing and reconstruction [12].

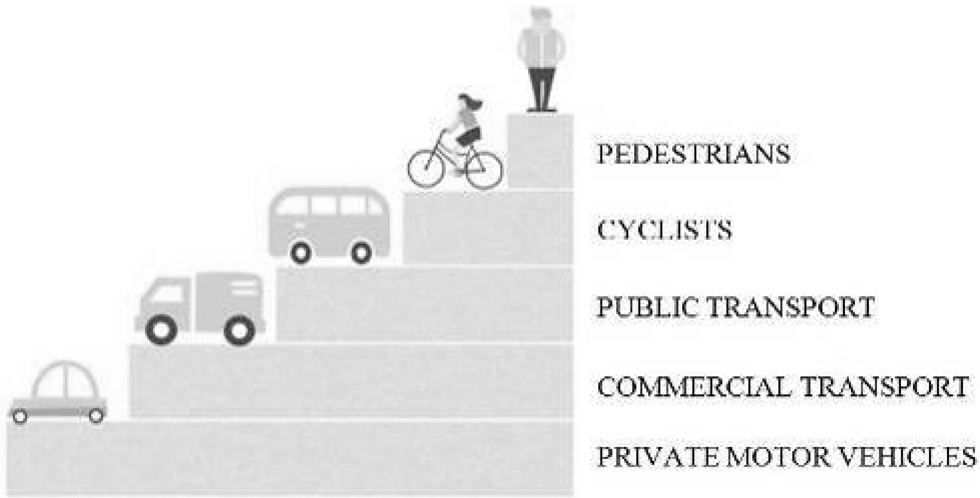


Figure 2 − The transport pyramid of the city

Considering the mass of the pedestrian movement and its safety for the environment, on the highest step of this pyramid pedestrian should be set. The long-term global experience proves that the city cannot be comfortable and attractive if it is not convenient for pedestrian traffic (the most vulnerable are low-mobility groups of population).

The third step of transport pyramid is public transportation, which carries far more people than private cars, produces considerably fewer emissions (especially trolleybuses), takes much less space on the road and is not parked for a long time in the central part of the city. The social role of public transport, which is much more affordable for private cars, is also great.

To commercial vehicles, delivering consumer goods, in cities with an efficient transport system have priority above private transport, since convenient conditions for this kind of transport stimulate business development and prevent the shortage of goods. Standard is the permission of commercial vehicles in certain hours, usually in the morning.

The last step in the pyramid of priorities are private vehicles, which, although provide high mobility, comfort and unlimited travel range, have low efficiency (large expenditure of energy efficiency relative to the weight they carry), causing noise and chemical pollution and occupies large areas.

The PIARC methodical documents of recent years and the works of 20th Congress highlight the following important directions of the development of road traffic organization: decreasing the intensity of traffic in city centers; searching for alternative, environmentally friendly modes of transport; the priority of public passenger transport and cars used by several passengers (HOV – high occupancy vehicles); parking regulation; the interaction between the street-road network and the urban environment.

The most radical means of reducing the intensity of traffic in urban centers are car-free zones. They are characterized by a total ban on traffic, with the exception of special types (ambulance, police, fire and communal services, shop maintenance). Such zones are arranged on small, usually guarded territories. As an example, the historical center of Tallinn (Old Town) and the ancient quarters of Little France (Strasbourg).

Currently, a common measure of traffic management is «traffic calming», which combines technical and architectural-planning solutions. According to the definition of the Institute of Transport Engineers (ITE) traffic calming is «a combination of physical measures that reduce the negative effect of cars using and improve the conditions for other street users» [13]. The main objectives of these measures are: improving living conditions; considering priority of the requirements that are put forward by the user of the city territory (work, recreation); creating safe and aesthetically attractive streets; reducing the negative effects of road transport (especially noise and pollution); creating favorable conditions for pedestrians, cyclists and people with limited mobility.

Among the main results achieved by the calm of the movement, the following items are indicated: decreasing of vehicles speed; reducing the number and severity of accidents; provision of conditions for various types of movement (by public transport, by bicycle, by foot); reduction of transit traffic of motor transport.

Traffic calming is achieved by technical measures and laws. First of all, creating calming zones they eliminate the transit movement, where within the zones through-streets turn into dead-end, loop, ring, etc. In addition, the speed limit is introduced, which can sharply reduce the number of conflicts between pedestrians and transport, and regulate parking. It should be emphasized that when designing zones of calming the improvement of the streets and the design of their space play very important role and they are considered as a means of influencing the mode of vehicles movement.

Service of areas often relies on public transport, which is in priority. Therefore, the combinations are possible, for example, of pedestrian traffic and tram lines

(Strasbourg, Saint-Etienne) or pedestrian traffic and bus routes (Dijon). The organization of street space, their landscaping and design ensure the priority of pedestrians and cyclists movement and stimulate the reduction of vehicles speed; in particular, it is supposed the reducing of SRN bandwidth or some its sections.

In the USA and Canada, where practice of parkway design is used, measures of calming the movement are combined with landscape design. For example, part of sustainable development programs in the great Vancouver area was the creation of so-called «green streets», «green ways». Designing of this street type involves reducing the coverage areas, increasing the areas of landing and lawns within the streets, turning them into boulevards, creation of separating strips with landings, application for sidewalk paving and construction from natural materials instead of standard asphalt.

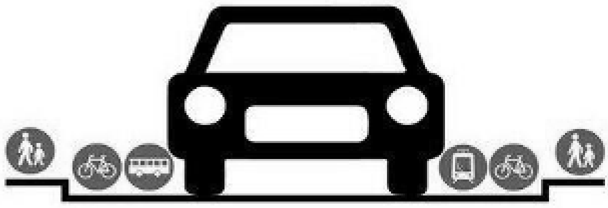
As an example, there should be considered the problems arising during the substantiation of the SRN development in disigning the plan of Poltava.

From the beginning of the XXI century, the city officials of Poltava faced the acute problems of organizing transport, cycling and pedestrian traffic and parking areas in the central part of the city, caused by a number of reasons. The level of motorization in Poltava region in 2003, 2005 – 2009 and 2011 was higher than the nationwide and in 2008 reached the maximum number – 164 cars per 1,000 inhabitants [14]. During the last decade, the so-called tertiary sector (trade, domestic services and various forms of commerce) has been rapidly developed in the settlement. Thus, the «trading core» – a functionally rich territory with an area of approximately 15 hectares, which became the focus of mass attraction of pedestrian and transport flows, has been created. The territory of the «trading core» is gradually expanding and is increasingly saturated with new objects. In fact, there is the process called in foreign urban planning literature as «urban regeneration». The organization of transport services of «trading core» requires the reconstruction of the SRN, but is in disagreement with the requirements of Poltava cultural and historical center preservation.

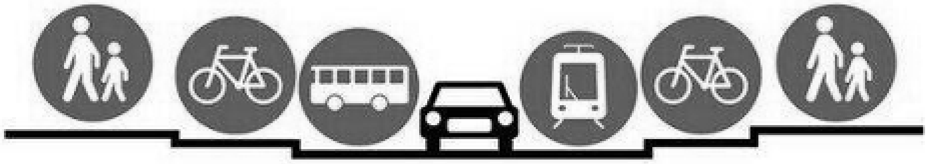
The city area needs the development of cycling, as the city is compact, most of the streets have a fairly wide passageway, sidewalks are often separated from the roadway with greenery, there are many parks, squares in the city, which promotes the development of recreational cycling. But the results of survey conducted in October 2014 – January 2015 by public organizations «SITI-LAB» and «VeloPoltava» give an idea of the obstacles that prevent Poltava residents from crossing the bike. Among the factors that influence cycling in Poltava are: low quality of roads and sidewalks (this factor is very important for 75.3% of the respondents, important, but not critical for 20.4% and not important for 4.3%); disrespect from drivers/pedestrians, discrimination on the road, violation of the road rules (48.1, 41.3 and 10.6% respectively); lack of bicycle paths (45.1, 46.8 and 8.1%), lack of bicycle parking (44.7, 41.3 and 14.0%), lack of bike parkings at work (38.7, 29.4 and 31.9%).

Therefore, the development and improvement of master plan will have to solve new problems, including the area of ensuring the availability of Poltava SRN to the needs of cyclists and people with disabilities. For execution and addition the transport section of the new master plan and, in particular, the justification of the SRN development, a number of initial indicators should be given. Among them are the projected level of motorization, population mobility (including owners of individual motor transport), distribution of passenger flows between public passenger transport and individual automobile, cycling and needs of people with disabilities.

Thus, Ukraine, like other countries of the world, has faced the problem of adapting the projects of city streets construction and reconstruction projects to changes of priorities, which took place in the theory of transport planning (see Fig. 3).



a)



b)

Figure 3 – The change of priorities, which took place in the theory of transport planning:
a) the priority of private car; b) the priority of pedestrian movement [15]

Conclusions. Considering the world experience in designing and reconstruction the street-road network of cities, the main goals to achieve adapting urban construction and reconstruction projects to the change of priorities, taking place in the theory of transport planning can be formulated:

– conducting constant researches as the basis for further design work;
– differentiation of the road network by type of prevailing types of transport and organization of movement;
– minimizing the mileage of traffic when traveling between any two points in the city;
– the maximum possible restriction of transit traffic within the city;
– reduction of the harmful effects of traffic flows on residential quarters, recreation areas, areas of historic buildings, which have architectural and artistic value;
– preparation of settlement for the future progress in the sphere of vehicles;
– creation of one-way streets and pedestrian streets;
– increasing mobility of people with disabilities, construction of main bicycle paths;
– clear zoning of streets, separation of pedestrians space and automobile streams, arrangement of parking places, etc.

To achieve these goals, it is necessary to make changes in transverse profiles of city streets (to change the distribution of width of the street, design slopes, types of coverage), to consider the priority when designing roads interconnections, to provide measures to improve the environmental situation in the urban environment. The competent implementation of these actions requires clear orientation in diversity of space organization elements.

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ENERGY PERFORMANCE OF RESIDENTIAL BUILDINGS

The problem of the Ukrainian housing stock energy performance is under consideration. Analysis of the outer walling thermal protection condition has been performed for the basic building construction solutions through the example of Poltava. The housing stock is represented by brick, large-block and panel system buildings, erected in the 50–80-th of the last century. The actual values of heat transfer resistance of the outer walls, windows, covers and other building enclosure are 3–5-fold less than the permissible minimum dimension according to the present-day requirements. The article presents recommendations for thermal modernization of the outer walls in accordance with their construction design, attic and basement floors (over the unheated basement), as well as transparent outer structures according to the present-day standard requirements.

Keywords: *thermal modernization, supplementary heat insulation, residential houses walls.*

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ЕНЕРГОЕФЕКТИВНІСТЬ ЖИТЛОВИХ БУДИНКІВ

Розглянуто проблему енергоефективності житлового фонду України. Проведено аналіз стану теплозахисту зовнішньої оболонки основних конструктивних рішень будівель на прикладі м. Полтава. Представлено житловий фонд цегляними, великоблочними та панельними будинками, які збудовані в 50–80 рр. минулого століття. Розраховано дійсні значення опору теплопередачі зовнішніх стін, вікон, покриття та інших огороджувальних конструкцій, які у 3–5 разів менші за мінімально допустимі за сучасними вимогами. Надано рекомендації щодо термомодернізації зовнішніх стін згідно з їх конструктивним рішенням, горищного перекриття та перекриття над неопалюваними підвалами, а також світлопрозорих зовнішніх конструкцій за сучасними нормативними вимогами.

Ключові слова: *термомодернізація, додаткове утеплення, стіна житлового будинку.*

Introduction. Solution of the energy saving problem significantly determines the terms of Ukraine's surmounting the long-lasting economic crisis. Maximum energy saving is reducing dependence on the countries supplying fuel and energy resources, it is minimizing energy-output ratio of national products.

The existing housing stock of the national houses was mainly formed in the postwar Soviet period, when at designing and constructing houses the primary expenditures were to be saved, and the energy exploitation costs were considered as the secondary indications in the ultimate building efficiency calculations. Change of the house-heating technological conditions, caused by the need of maximum energy saving, has lead to the mass-scale discomfort in the exploitation conditions of the residential buildings: low indoors air temperature, high humidity, carcinogenic fungi on the internal structures surface and in the premises air. These issues are requiring the immediate solution due to their great social importance, thus, at analyzing the exploitation properties of buildings, it is necessary to provide not their energy saving, but energy performance.

Entering the European environment expected by Ukraine stipulates enacting of common European regulations and their implementation ways. The major field of development is harmonizing the national standards base [1, 2] with the European Union's requirements in terms of the buildings' energy performance [11] and implementing «Energy strategy of Ukraine up to 2030» in the building construction industry.

Raising the energy consumption efficiency is impossible without taking specific measures in the sphere of building construction.

Due to the necessity of detailing the state forecasts on energy saving in the national economy, data have been updated concerning the energy saving development of the country in accordance with the Energy strategy of Ukraine up to 2030.

According to the above Strategy, in 2030, compared to 2005, the total fuel resources saving due to the engineering factor is estimated to make 128.42 mlnt.r.f. (ton of reference fuel); electric energy saving will make 108.72 bln kW/year; heat energy saving will make 231.87 mlnGcal which totally makes 198.06 mlnt.r.f.

The basic energy saving reserve is reducing energy resources consumption by the residential and public objects which share in the total energy consumption of the civil-engineering industry makes over 80%.

Latest sources of studies and publications review. Many authors devoted their studies to the issues of raising the heat shielding properties of the residential and public buildings' walls to comply the standard regulations requirements. For instance, they were considered in the works [3–5, 9, 10]. Study [6] is devoted to the analysis of the heat-retainer (insulant) type's influence on the humidity status of the walling, the insulant being located on the indoors side of the walling. Study [7] is considering the issues of the basement outer walling heat insulation. Studies [8] are devoted to the issues of the outer walling insulation in the areas of heat-conductive materials inclusions.

Setting parts of the total problem not-solved before. Publications devoted to thermal modernization of the residential houses' outer walling structures do not consider the influence of the panel walls joints on the additional heat insulation layer width. The joint's area occupies a significant part of the panel wall's area, therefore, its structure not taken into account, is leading to errors in determining the additional heat insulation layer width. Besides panel walls, the study considers other most common outer walling types used in residential houses of Poltava.

Setting the problem. The present study was aimed at developing recommendations on the outer walls thermal modernization in the existing residential houses of Poltava. While determining the width of the additional insulant in the panel walls, the influence of the panel walls joint's structure on the value of the heat transfer resistance was taken into account.

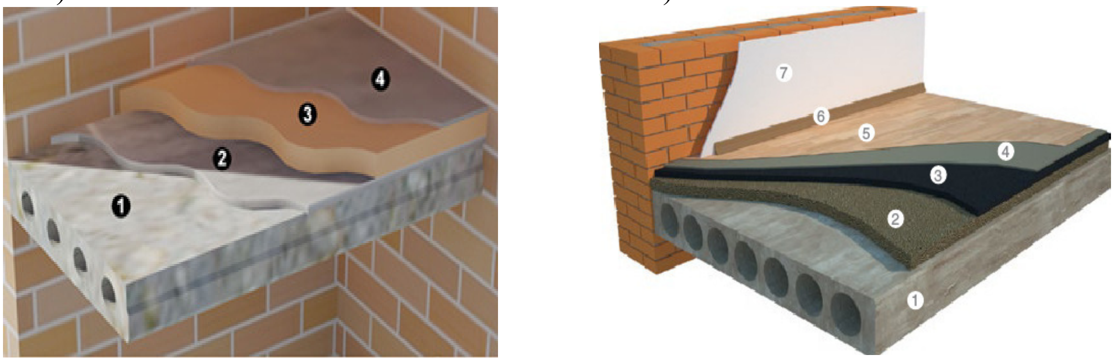
Basic material and the results. The housing stock of Poltava is represented by the brick, large-block and panel system buildings, erected in the 50–80-th of the last century (Fig. 1).

Figure 1 – Residential buildings of Poltava

The present state of residential houses walling thermal protection in Poltava does not comply with the present-day requirements [1]. The actual values of heat transfer resistance of the outer walls, windows, covers and other walling structure are 3–5-fold less than those registered in [1, 2], as the permissible minimum values.

In the Soviet period, the attic floor heat insulation was performed by means of floor filling with keramzit (expanded clay aggregate). The heat transfer resistance of the said structure makes 1.5–2 $m^2 \cdot K/W$. To obtain the standardized heat transfer resistance value (4.95 $m^2 \cdot K/W$) the floor should be insulated with the efficient slab insulant of 200 mm width which is determined by the thermotechnical calculation according to the diagram in Figure 2,a. The unheated basement floor covering structure should have heat transfer resistance not less than 3.75 $m^2 \cdot K/W$ and it should be performed according the diagram in Figure 2, b.

a) b)



a)	b)
1. Floor slab	*1. Floor slab*
2. Vapour barrier insulation	*2. IZOVAT 160 heat insulant*
3. IZOVAT 40 heat insulant	*3. Waterproofing*
4. Sand-cements creed	*4. Reinforced screed*
	5. Floor covering
	6. Skirtboard
	7. Wall

Figure 2 – Recommendations for heat insulation
a – of the attic floor; b– the unheated basement floor

The present-day heat-engineering requirements to the windows stipulate installation of double-glazed insulation windows with the low emissivity glass or the glass chambers filled with argon or krypton. For the city of Poltava, the minimal permissible value of the heat transfer resistance makes 0.75 $m^2 \cdot K/W$, therefore the following glazing variants are recommended: variant 4i-10-4M1-10-4i (double-glazed insulation window with two layers of the low emissivity glass and soft coating and one standard glassplate, the glass chambers filled with air) or variant $4M_1$-10-$4M_1$-10-4K (double-glazed insulation window with one layer of the low emissivity glass and hard coating and two standard glass plates, the glass chambers filled with krypton).

Most walling structures in residential buildings are made of panels, large-blocks and brick. Panels manufactured at Poltava Integrated House-Building Factory (HBF) are illustrated in Figure 3.

a) b)

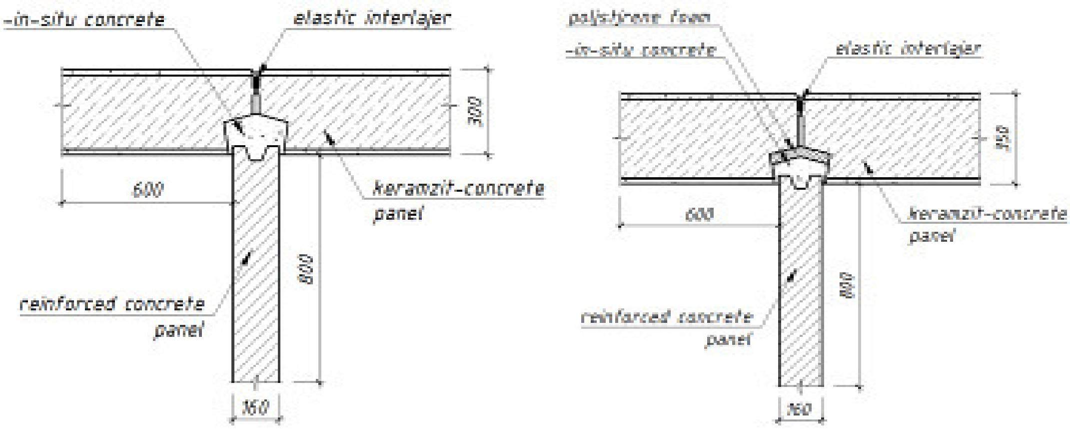


Figure 3 – Wall panels structure
a – 300 mm wide without the thermofiller; b – 350 mm wide with thermofiller

Wall panel is manufactured without the thermofiller (Fig.3, a). Having taken into account the suggestions made by PoltIBI, Poltava Integrated House-Building Factory started producing wall panels of 350 mm width and performing their joints using thermofiller containing polystyrene foam with the density $\rho_0 = 50$ kg/m^3 and width of 40 mm (Fig. 3, b).

At determining panel walls' thermal protection it is necessary to take into account the panels joints which occupy a significant part of the total panel area. Therefore, the heat transfer resistance calculation should be performed based on the structure's temperature field. The temperature field calculation is performed by means of Elcut program.

Temperature fields of the unheated panel wall are displayed in Figure 4.

Results of determining the panel wall's matched heat transfer resistance are presented in Table 1.

Table 1 – Results of determining the panel wall's matched heat transfer resistance

<table>
<tr><td>Panel wall's width, mm</td><td>Matched heat transfer resistance of the walling, m^2·K/W</td><td>Minimal-temperature on the internal surface, °C</td><td>Meantemperature on the internal surface, °C</td><td>Minimalpermissiblevalueofthewallingstructure'sheattransferresistance value, m^2·K/ W</td></tr>
<tr><td>300</td><td>0.595</td><td>12</td><td>12.2</td><td rowspan="2">3.3</td></tr>
<tr><td>350</td><td>0.721</td><td>12.9</td><td>13.3</td></tr>
</table>

a) b)

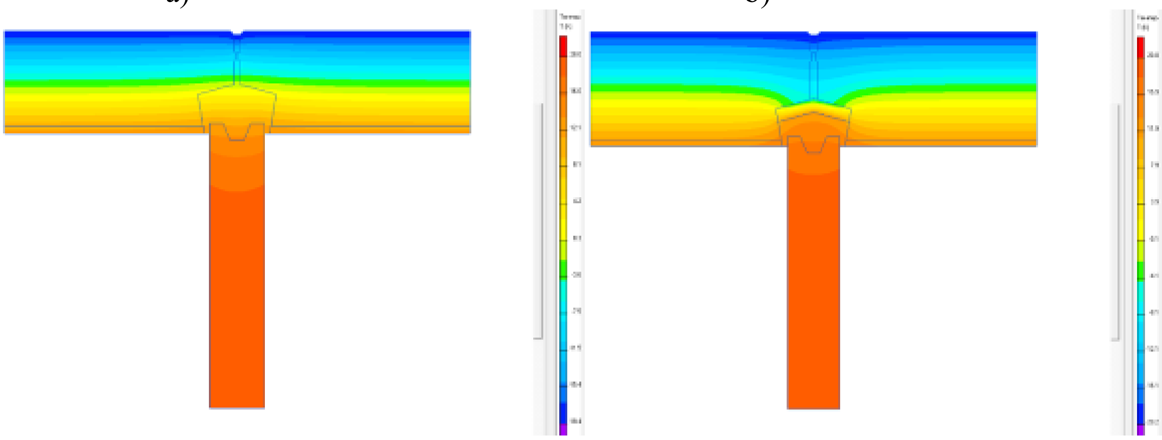


Figure 4 – Temperature field of the unheated panel wall:
a – 300 mm wide without the thermofiller; b – 350 mm wide with the thermofiller

Additional outer heat insulation can be made of the double-density rock wool FASROCKMAX (the outer layer having the density of$\rho_0 = 155$ kg/m^3, the internal layer's density is $\rho_0 = 87$ kg/m^3) or of polystyrene foam PCB-C-35 with the density of $\rho_0 = 35$ kg/m^3. Advantages of the first variantare high exploitation properties, the second variant's merit is its low cost. Results of determining the additional insulation width are presented in Table 2.

Large blocks which were used for building construction in Poltava were made of keramzit with the density of $\rho_0 = 1600$ kg/m^3. Most of the outer walling structures made of brick have the width of 0.38 m, 0.51 m and 0.64 m. Brickwork of conventional clay brick with sand-cement mortar having the density of $\rho_0 = 1800$ kg/m^3 was used.

Results of determining the matched heat transfer resistance for large-block and brick walls are presented in Table 3.

Table 2 – Results of determining the additional insulation width

Additional insulation type	Additional insulation width, mm	Walling heat transfer resistance, $m^2 \cdot K/W$	Minimal temperature of the internal surface, °C
Rock wool FASROCKMAX	110	3.433	18.6
Polystyrene foam PCB-C-35	110	3.507	18.6

Table 3 – Results of determining the matched heat transfer resistance for large-block and brick walls

Wall made of:	Wall width, mm	Walling matched heat transfer resistance, $m^2 \cdot K/W$	Minimal temperature of the internal surface, °C	Mean temperature of the internal surface, °C	Minimal permissible value of the walling structure heat transfer resistance, $m^2 \cdot K/W$
blocks	400	0.714	13.2	13.2	3.3
brick	0.38	0.649	12.6	12.6	
	0.51	0.81	14	14	
	0.64	0.97	15	15	

The order of layers location at thermal modernization of the outer walling structures for large-block and brick buildings is illustrated in Figure 5.

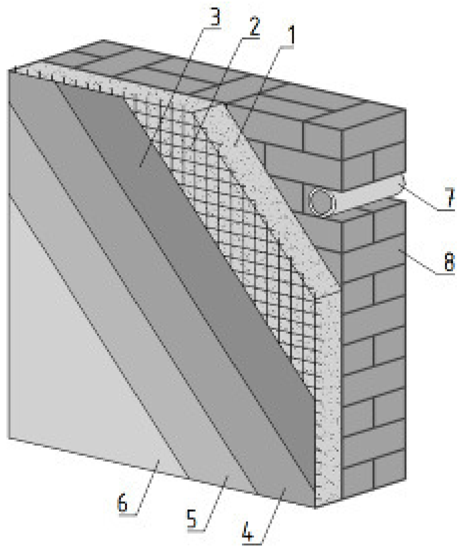


Figure 5 – Façade of the thin-layer plaster system:
1 – heat insulant; 2 – steel grid; 3 – base coating layer; 4 – dubbing layer; 5 – decorative plaster; 6 – masonry paint; 7 – steel anchoring; 8 – wall

Width of the additional heat insulation for the outer walls made of blocks and brick is presented in Table 4.

Table 4 – Results of determining the additional heat insulation width

Additional heat insulation type	Wall material	Wall width, m	Additional heat insulation width, mm	Walling heat transfer resistance, $m^2 \cdot K/W$	Minimal temperature of the internal surface, °C
Rock wool FASROCKMAX	Blocks	0.4	0.11	3.522	18.6
	Brick	0.38	0.11	3.47	18.6
		0.51	0.1	3.373	18.7
		0.64	0.1	3.534	18.6
Polystyrene foam PCB-C-35	Blocks	0.4	0.1	3.346	18.6
	Brick	0.38	0.11	3.544	18.6
		0.51	0.1	3.441	18.6
		0.64	0.09	3.338	18.6

Conclusions:

1. Recommendations on thermal modernization of the existing residential buildings' outer walls in Poltava, to be used in practical activity, have first been developed.
2. To determine the additional heat insulant's width in panel walls the influence of the panel walls joint structure on the matched heat transfer resistance.
3. Over 90% of the existing residential houses in Poltava require thermal modernization.
4. The outer walling heat transfer resistance in most of residential houses is less than the standardized value. In panel buildings it is by 78%–82%, in block ones – by 78% and in brick houses – by 71%–80 % less.
5. The slabs' heat transfer resistance is by 60%–85% and that of the windows is by 70% less than the standardized value.
6. Compliance with the standard regulations of heat transfer needs applying the additional insulant layer 100–110 mm wide, using additional insulation of FASROCKMAX rock wool, and the layer 90–110 mm wide, using PCB-C-35 polystyrene foam. The panel walls joints' influence on the heat transfer resistance should be taken into account while performing thermal modernization of panel walls.
7. Further research should be devoted to studying the influence of all panel walls joints variants on the walling matched heat transfer resistance values.

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STATIC MIXERS IN FLUSHING FLUID CIRCULATION SYSTEMS OF DRILLING RIGS

The work of three static mixer designs in flushing fluid circulation systems of drilling rigs was studied taking into account the basic parameters of drilling mud. Modelling was carried out, and parametric slurry fields were obtained in the pipe work area, that is in a static mixer installation site and in the area of pipeline behind it. The following models were obtained: velocity fields, vorticity fields, turbulence intensity fields, scale turbulence fields along the travel path. Graphs of the parameters changes regarding the tube axis were plotted. The comparative analysis of patterns and curves was carried out. The rational design of static mixer for obtaining of optimum mud mixing technology features is grounded.

***Keywords**: static mixer, mud, modelling, vorticity field, turbulence intensity, turbulence scale.*

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СТАТИЧНІ ЗМІШУВАЧІ В ЦИРКУЛЯЦІЙНИХ СИСТЕМАХ ПРОМИВАЛЬНОЇ РІДИНИ БУРОВИХ УСТАНОВОК

Досліджено роботу трьох конструкцій статичного змішувача в циркуляційній системі промивальної рідини бурових установок, ураховуючи основні параметри бурового розчину. Виконано моделювання та отримано параметричні поля гідросуміші у робочій зоні труби, а саме в місці установки статичного змішувача і в зоні трубопроводу за ним. Отримано моделі: поля швидкостей, поля завихреності, поля інтенсивності турбулентності, поля масштабу турбулентностей по довжині шляху перемішування. Побудовано графіки зміни досліджуваних параметрів відносно осі труби. Виконано порівняльний аналіз моделей та кривих. Обґрунтовано раціональну конструкцію статичного змішувача для отримання оптимальних технологічних характеристик перемішування бурового розчину.

***Ключові слова:** статичний змішувач, буровий розчин, моделювання, поле завихреності, інтенсивність турбулентності, масштаб турбулентності.*

Introduction. In recent times static mixers are widely used in a number of industries, due to their advantages, including a large number of possible options for design solutions, the lack of moving parts, drive units, power consumption, possibility of hydro and pneumatic-transport networks combination, as well as a variety of technological features, which they are able to perform: gaseous, liquid and loose solidphase components mixing, dispersing of solidphase components in poor and immiscible liquids, solidphase flocculation in fluid flows, intensification of the reagents dissolving in liquids, etc. [1].

In the flushing fluid circulation systems we use different designs of mechanical agitators, such as blade, rotary, ball ones and others. Static mixers are seen to be promising [2]. However, their design should meet the requirements of effective drilling mud mixing with reagents, to integrate into existing hydrotransport communications of the drilling rig circulation system of the surface complex. To attain this purpose it is important to perform the study of the static mixer effect on slurry parametric fields, including speed and vortacity.

Analysis of recent research and publications of sources. Development of technical solutions and active static mixers research starts in 1980 – 90 [1, 3, 4]. In [1] it is emphasized that the construction of static mixers of the first generation was based mainly on intuition with the following empirical approbation of options. The best of them was chosen on the basis of comparative analysis. Instead, modern technology widely uses slurry flow modelling. In [5], numerous tools for analyzing complex mixing devices were developed, 3D. Calculations are based on the finite elements method. In [6], the author using methods of computational dynamics of fluid for modelling showed that the static mixer forms a complex vorticity system which includes a stationary longitudinal vortex flow and transient (secondary) flows (vortexes) that, according to V. G. Levich [7], can be considered as a developed turbulence.

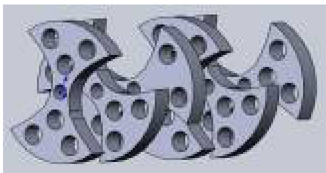
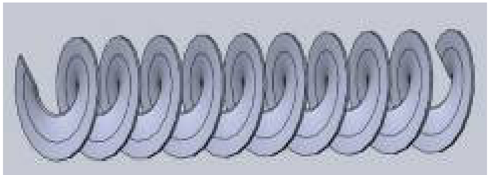
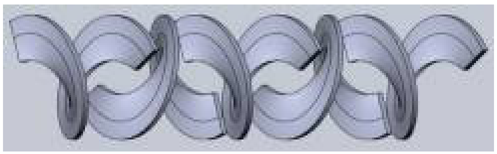
In [8], the dependence of pressure drops in turbulent flow (Re = 1000 – 5000) created in a static mixer, away from the number Re, was studied. In [9], the Kenics static mixer in a wide range of turbulent flow (Re = 1 – 25000) was examined. The picture of the velocity field and pressure difference for three modifications of the Kenics-mixer and flow of liquid and air was obtained by a picture of the velocity field and pressure difference for three modifications Kenics-mixer and flow of liquid and air was obtained by numerical modelling. In [10], the authors use «mixing efficiency» as the description of the static mixer and show its dependence on power spent on mixing, and the difference in pressure in the mixer. Using the method of mirror images the calculations of parametric mixer fields were carried out. On the base of their comparative analysis is was shown that different designs of static mixer determine different models of mixing, which are called «global» and «local». Optimization of the mixer geometry to provide the necessary process parameters was carried out.

Accentuation of unsolved before aspects of the problem. However, in domestic and foreign practice studies of static mixer in flushing fluid circulation systems of drilling rigs are unknown.

Setting objectives. The aim of this research is to study the design of three non-newtonian fluid static mixers with the following parameters of mud: density – 1250 kg/m^3, dynamic viscosity – 0.02 cPs with the use of the Flow Simulation module of Solid Works software environment.

Basic material and results. The study is provided for the design of three static mixers belonging to different classes of devices (Table 1).

Table 1 – Investigated objects «pipeline – static mixer»

Number of the experiment	Design description	3D-model mixer
1	Pipe Ø114x9 mm	Without mixer
2	Pipe Ø114x9 mm, with an additional set of static mixer № 1	
3	Pipe Ø114x9 mm, with an additional set of static mixer № 2	
4	Pipe Ø114x9 mm, with an additional set of static mixer № 3	

To obtain models of the above-mentioned parametric fields we use the Flow Simulation module of the SolidWorks software environment [12, 13]. The results of the research are presented in Table 2. The graphs of the mud velocity changes along the length of the pipeline (curves *v(L)* are given in Fig. 1. The graphs of the vorticity changes *I (%)* regarding the pipe axis *L (m)*, (curves *n(L)*) are shown in Fig. 2. The graphs of turbulence intensity changes l_m *(m)* regarding the pipe axis *L (m)* (curves *I (L)*) in Fig. 3. Schedules zoom l_t turbulence (*m*) tube axis *L (m)* (curves l_m *(L)* are shown in Fig. 4.

We calculate the following parametric fields of the slurry in the pipe work area, which covers the mixer itself and a section of pipe behind it up to 20 pipe diameters long:

– slurry velocity field *v* (*m/s*);

– vorticity field n (c^{-1}) (average circular velocity of the fluid in vortex flow);

– turbulence intensity field (%) I [11]

$$I \equiv \frac{u'}{U} \,, \tag{1}$$

where the mean-square velocity of turbulent fluctuations is

$$u' \equiv \sqrt{\frac{1}{3}u_x'^2 + u_y'^2 + u_z'^2} = \sqrt{\frac{2}{3}k} \;; \tag{2}$$

and the average turbulent flow speed is

$$U \equiv \sqrt{U_x^2 + U_y^2 + U_z^2} \;; \tag{3}$$

– turbulence scale field along mixing length l_m *(m)*.

Table 2 – Results of static mixer modelling with the help of flow simulation module

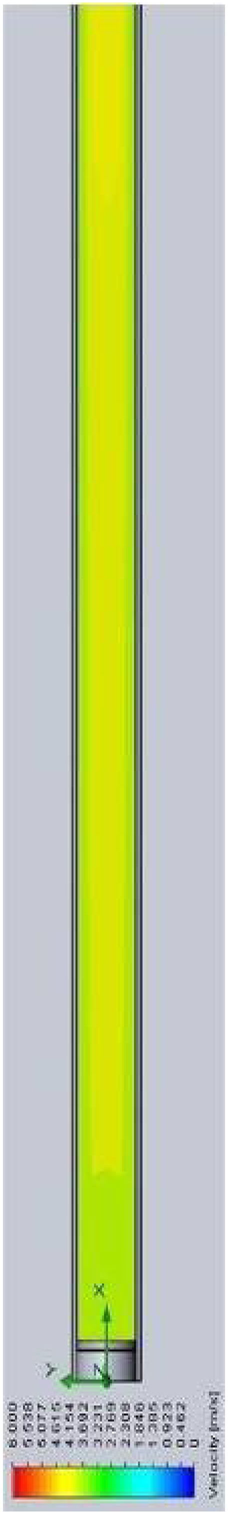
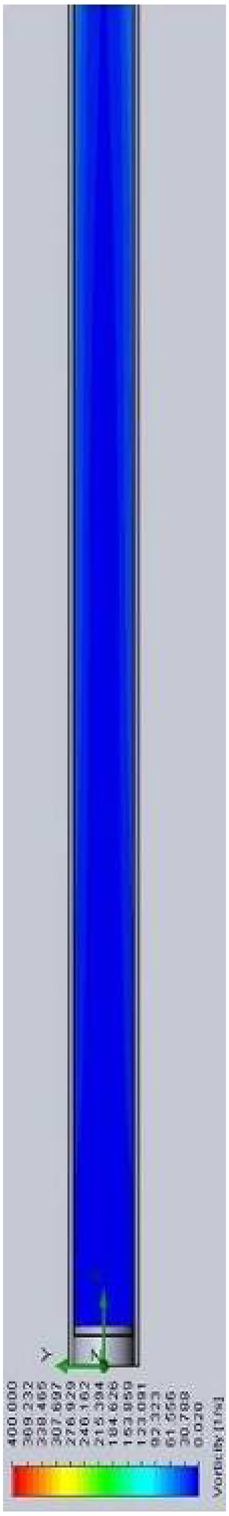
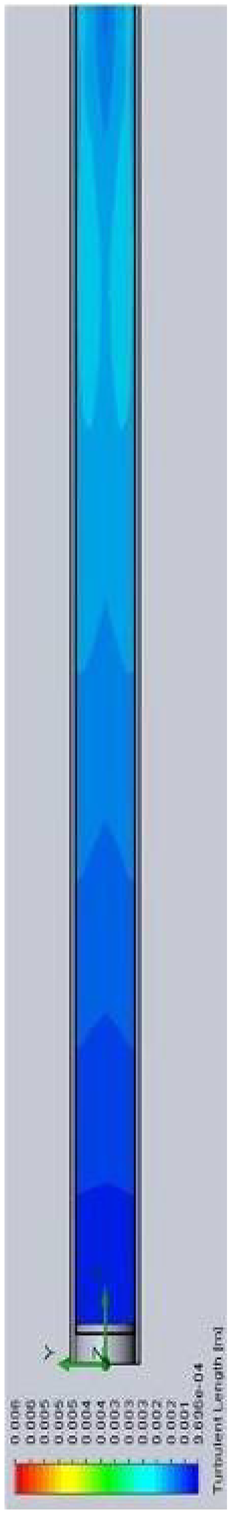
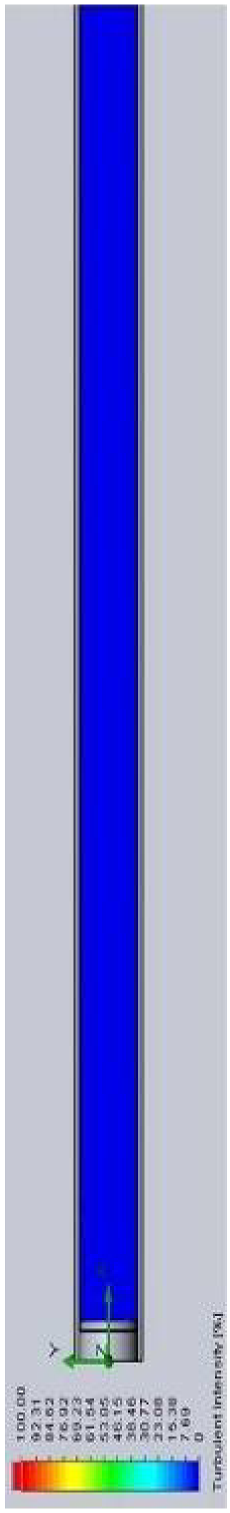
Experiment number	Experiment 1 – Pipe Ø114×9 mm with out static mixer			
Model of parametric experiment	Velocity field model	Vorticity field mode	Turbulence intensity model	Turbulence scale model
				

Table 2 ctd

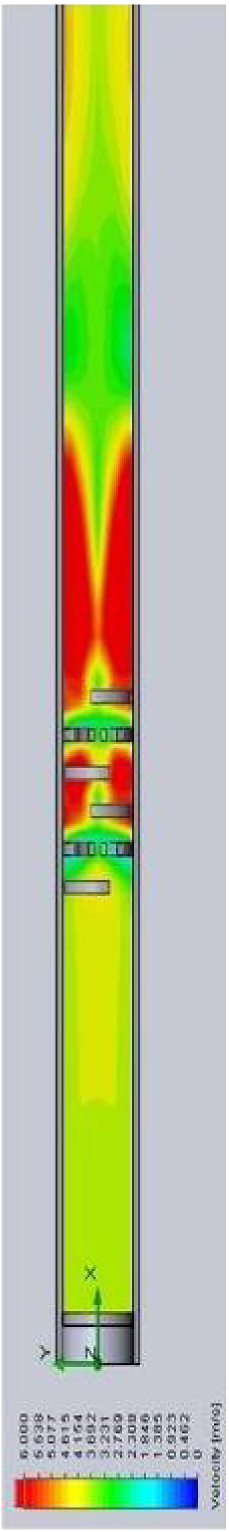
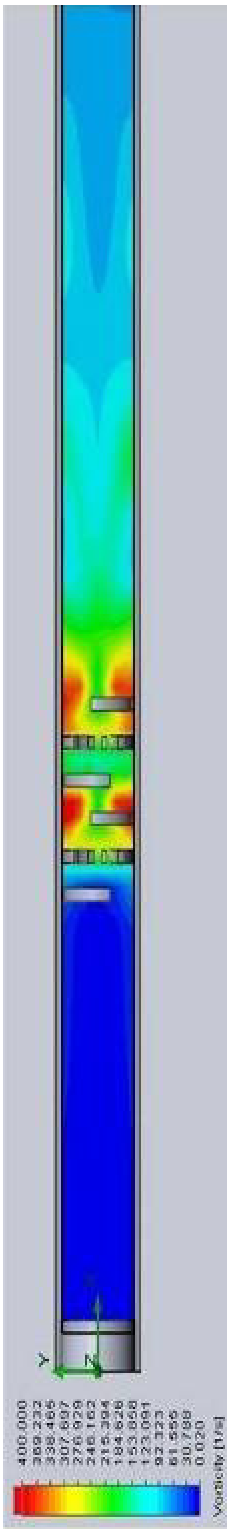
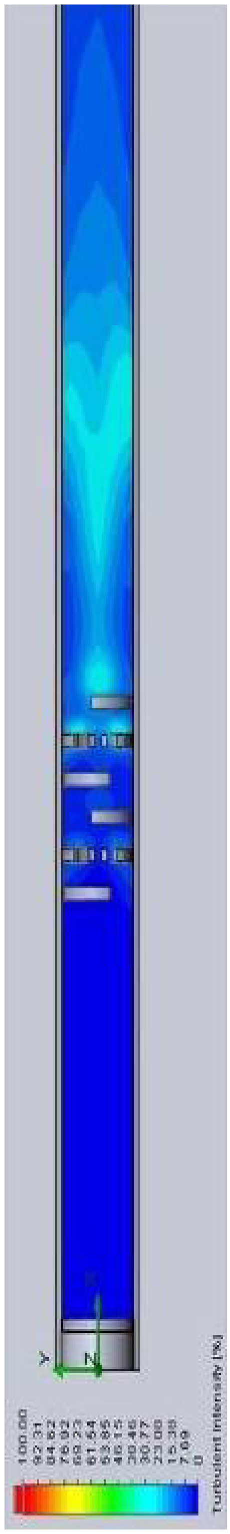
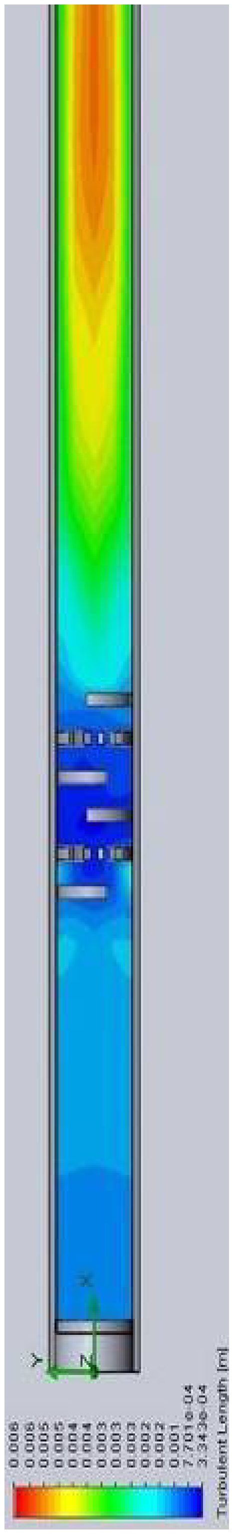
Experiment number	Experiment 2 – Pipe Ø114×9 mm with an additional static mixer № 1			
Model of parametric experiment	Velocity field model	Vorticity field mode	Turbulence intensity model	Turbulence scale model
				

Table 2 ctd

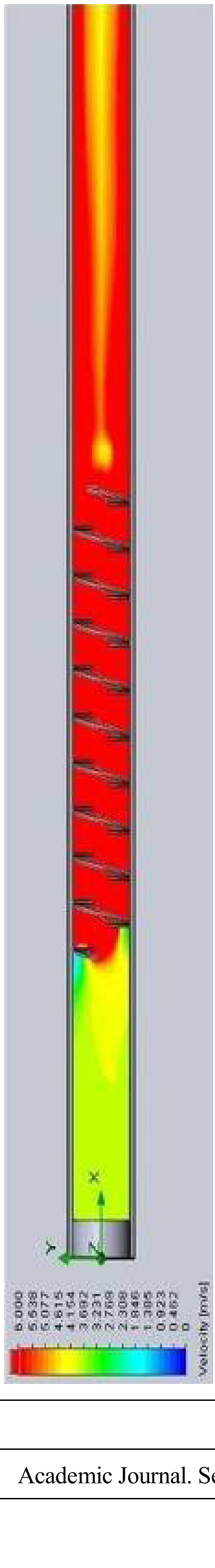
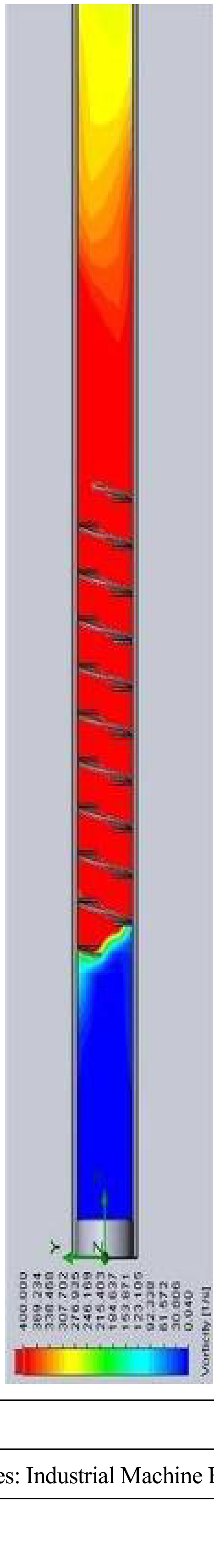
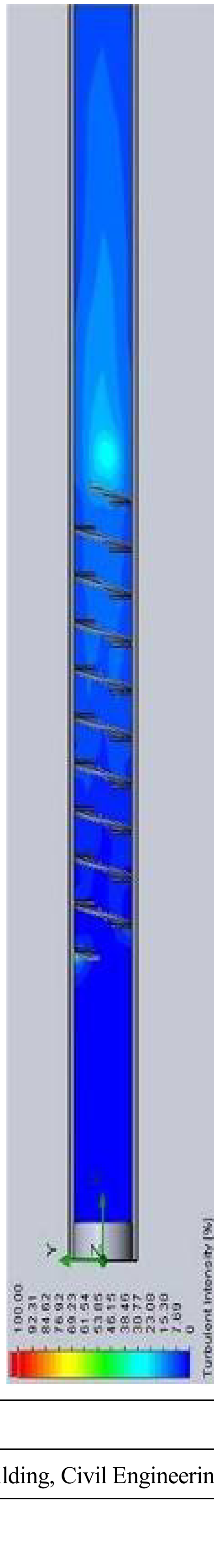
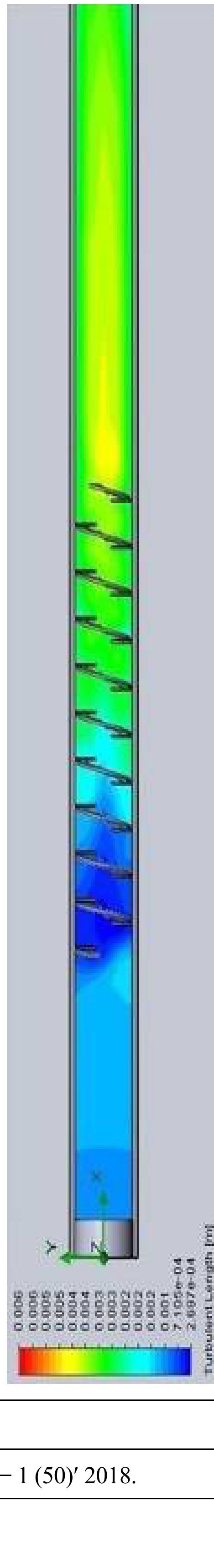
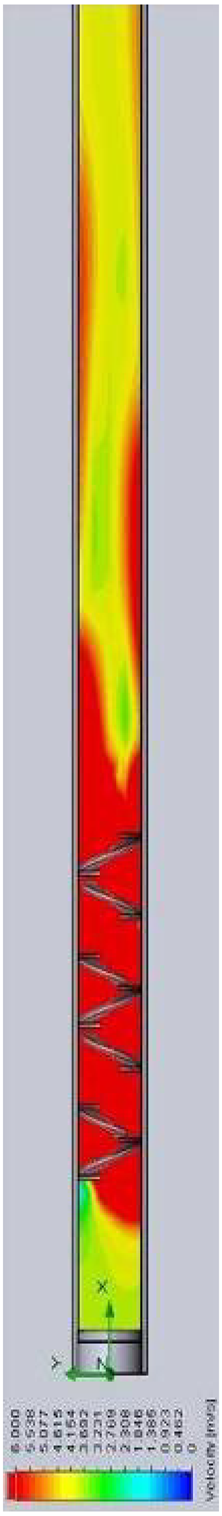
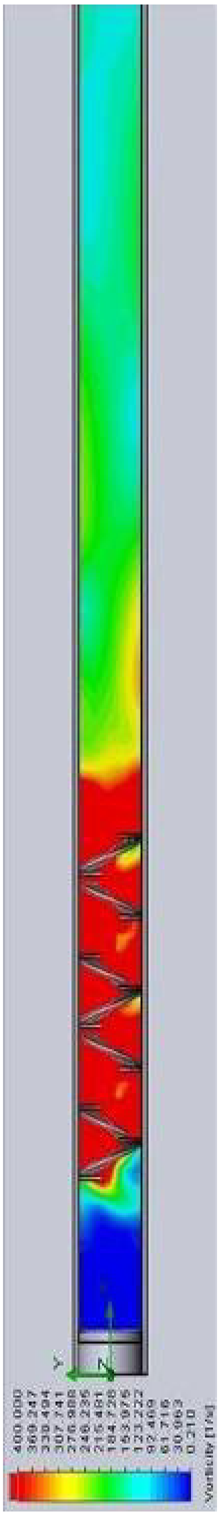
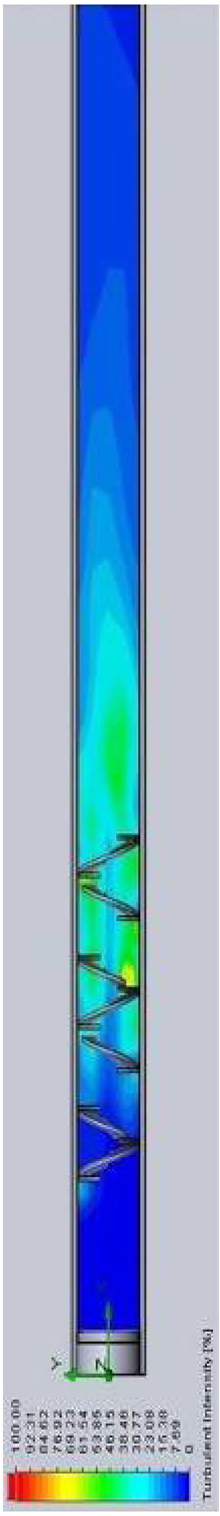
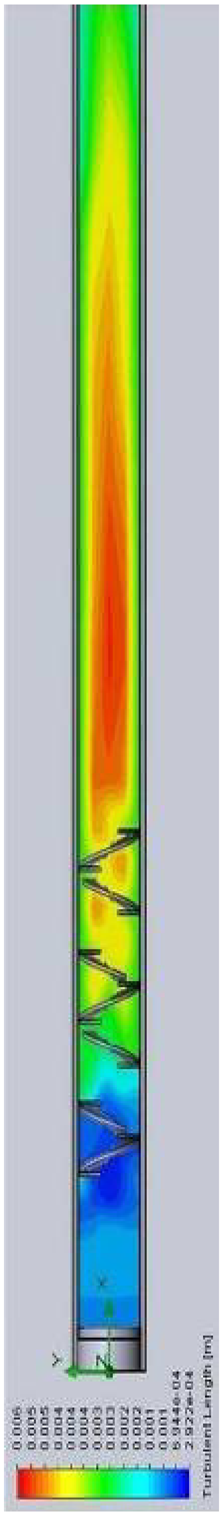
Experiment number	Experiment 3 – Pipe Ø114×9 mm with an additional static mixer № 2			
	Velocity field model	Vorticity field mode	Turbulence intensity model	Turbulence scale model
Model of parametric experiment				

Table 2 ctd

Experiment number	Experiment 4 – Pipe Ø114×9 mm with an additional static mixer № 3			
Model of parametric experiment	Velocity field model	Vorticity field mode	Turbulence intensity model	Turbulence scale model
				

The results describe the changes of mud turbulence along the pipeline in the site of static mixer installation and in the area of pipeline behind it. As you can see, turbulence evaluations are represented by curves *n (L)*, and *(L)* and l_t *(L)* which correspond each other (Figure 1 – 4). The average circular velocity of the fluid in the vortex flow *n*, the turbulence intensity *I* and turbulence scale l_m attain their maximum at the site of static mixer installation and then decrease along the pipe at the distance up to 10 pipe diameters (*10D*). Similar experimental data on the Re character were obtained in [11, 14]. However, each of the studied types of the static mixer affect individual characteristics of turbulence in different ways.

The maximum and minimum values of velocity *v*, vorticity *n*, turbulence intensity *I* and turbulence scale l_t are shown in Table 3.

The comparative analysis of the velocity curves *v (L)* shows that the mixers № 2 and № 3 differ favorably from the mixer № 1, and give close values of characteristics of the flow field in the pipe. The maximum flow velocity for them is at 15 – 16 m/s, minimum – 3.5 – 3.6 m/s, that is vastly larger than the indexes of the mixer № 1 (respectively 4.9 and 1.8 m/s) and 3.5 times more than the maximum flow velocity without mixer.

The analysis of vorticity curves *n (L)* shows that best data demonstrates mixer № 2 (maximum 786.5 s^{-1}), a little worse – mixer number 3 (706.4), next lower order are characteristics of mixer № 1 (108, 5). However, mixer № 2 delivers stable high vorticity in the pipeline on the section of 0.5 m long. For other mixers the curve *n (L)* has an unstable pulsating character.

The comparative analysis of turbulence intensity curves changes l_m *(L)* shows a significant advantage of mixer № 3 (maximum 43.53%) and the practical parity rate of results of mixer № 1 and 2 (respectively 28.13 and 27.01%).

The analysis of the changes of curves of the turbulent vorticity scale along the pipeline *n (L)* shows almost the same picture for all three mixers – the diameter of vortexes in turbulent flows naturally reduces at the site behind the mixer. The rate of turbulence scale changes is the same for mixers № 1, 2 and 3.

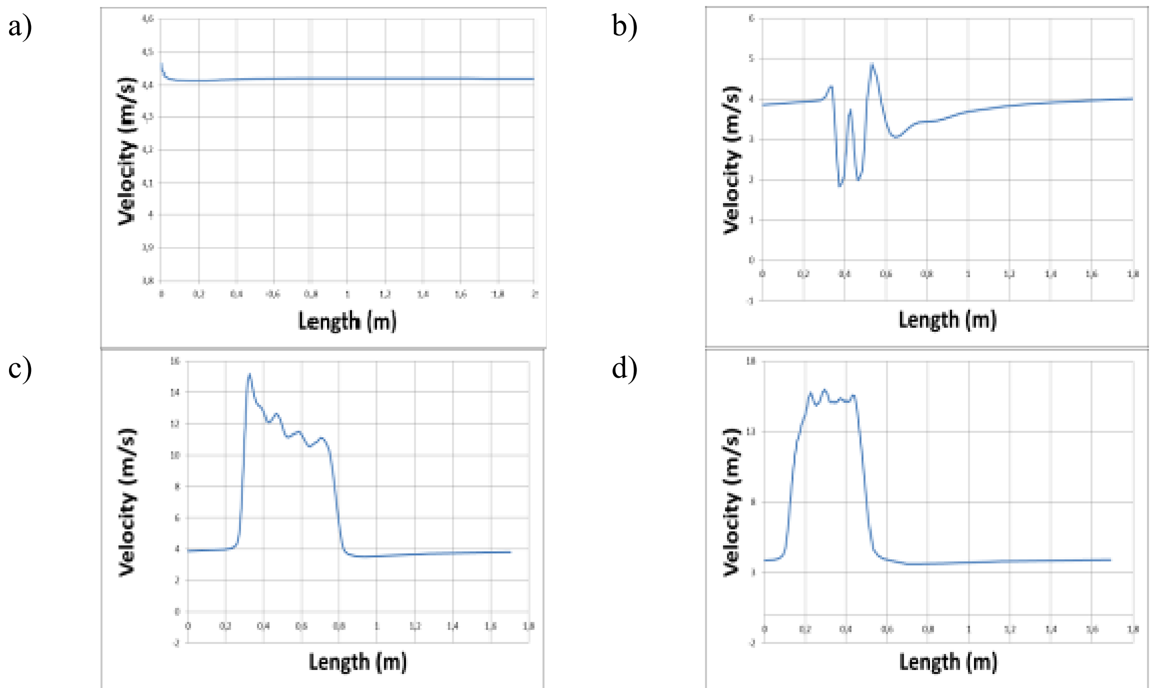


Figure 1 – The schedules of velocity changes *v* (m/s) relatively to the tube axis *L* (m) (curves *v(L)*):
a) experiment 1 (pipe without static mixer); b) experiment 2 (mixer № 1); c) experiment 3 (mixer № 2); d) experiment 4 (mixer № 3)

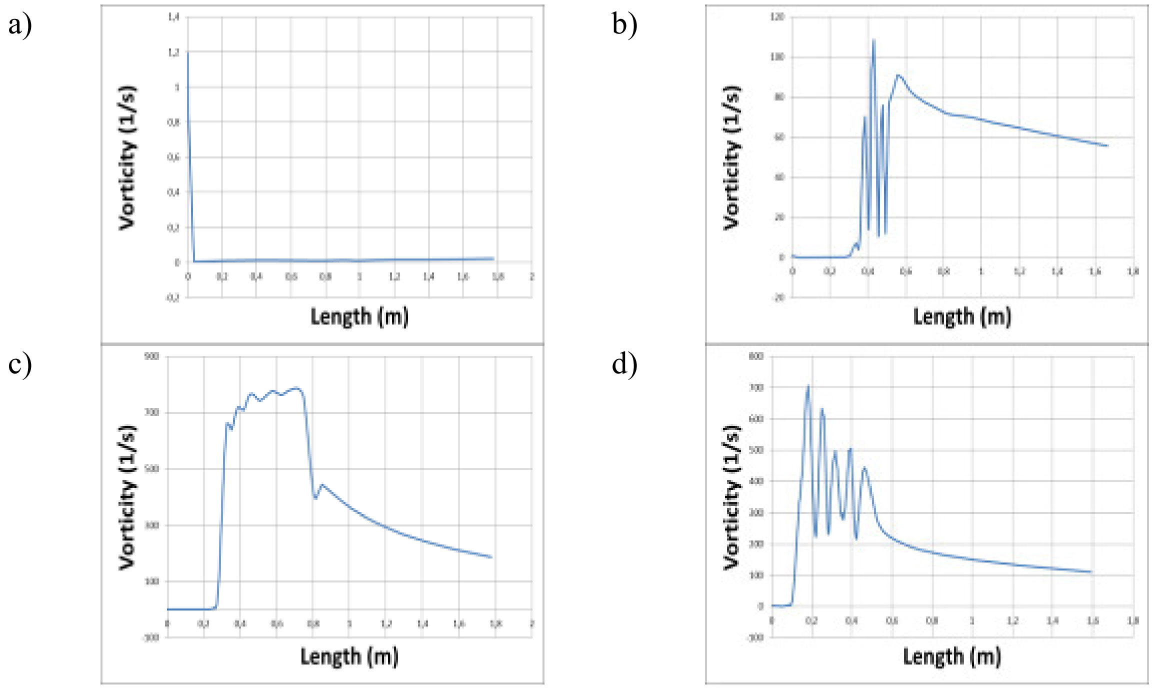


Figure 2 – The schedules of vorticity changes *n* (c^{-1}) relatively to the tube axis *L* (m), (curves *n(L)*):
a) experiment 1 (pipe without static mixer); b) experiment 2 (mixer № 1);
c) experiment 3 (mixer № 2); d) experiment 4 (mixer № 3)

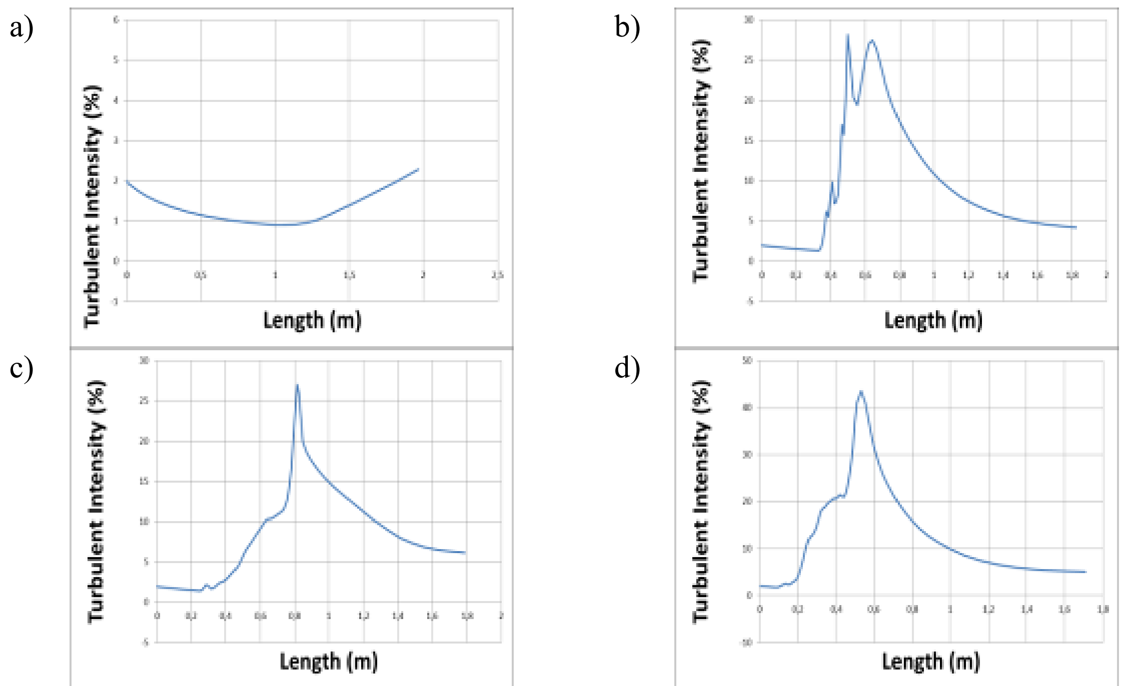


Figure 3 – The schedules of turbulenceintensity changes *I* (%) relatively to the tube axis *L* (m) (curves *I (L)*):
a) experiment 1 (pipe without static mixer); b) experiment 2 (mixer № 1);
c) experiment 3 (mixer № 2); d) experiment 4 (mixer № 3)

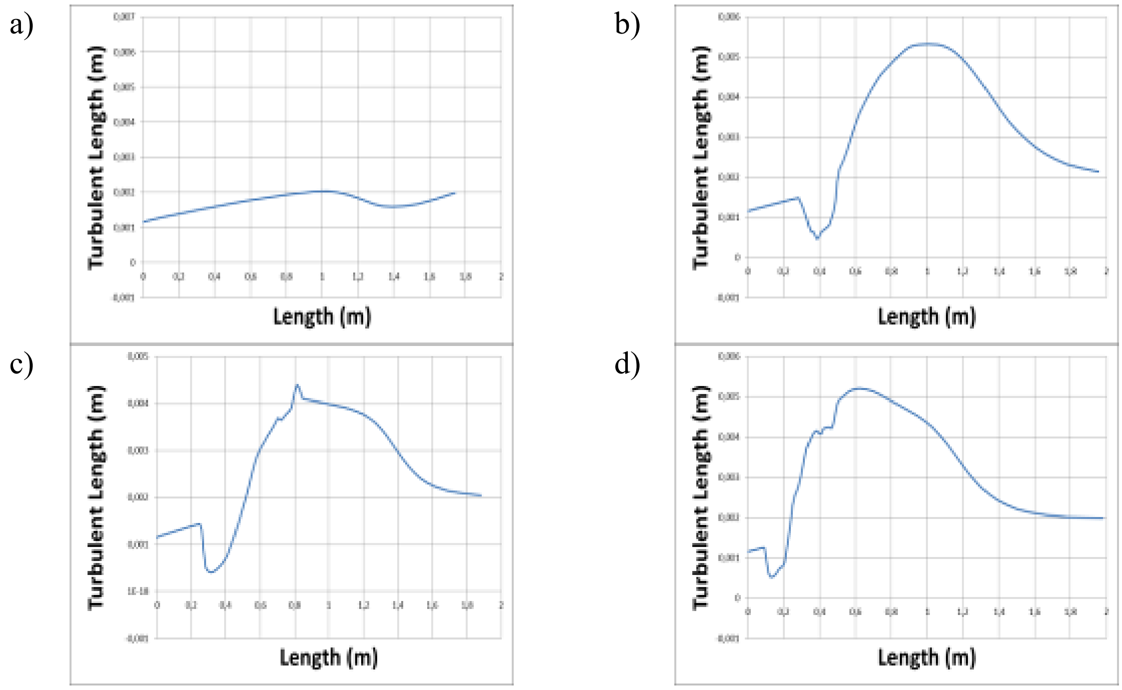


Figure 4 – The schedules of turbulence scale changes l_m (m) relatively to the tube axis *L* (m) (curves l_m (*L*)):
a) experiment 1 (pipe without static mixer); b) experiment 2 (mixer № 1);
c) experiment 3 (mixer № 2); d) experiment 4 (mixer № 3)

Table 3 – Maximum and minimum parameter values

Parameter	Experiment No	Design description	Max value	Min value
Velocity *v (m/s)*	1	Pipe without static mixer	4,43	4,41
	2	Pipe with mixer № 1	4,88	1,83
	3	Pipe with mixer № 2	15,19	3,53
	4	Pipe with mixer № 3	15,99	3,59
Vorticity *n*, c^{-1}	1	Pipe without static mixer	1,19	0,01
	2	Pipe with mixer № 1	108,5	0,01
	3	Pipe with mixer № 2	786,5	0,01
	4	Pipe with mixer № 3	706,4	0,97
Turbulence intensity *I* (%)	1	Pipe without static mixer	2,27	0,89
	2	Pipe with mixer № 1	28,13	1,28
	3	Pipe with mixer № 2	27,01	1,41
	4	Pipe with mixer № 3	43,53	1,73
Turbulence scale l_m, *m*	1	Pipe without static mixer	0,0020	0,0011
	2	Pipe with mixer № 1	0,0053	0,0005
	3	Pipe with mixer № 2	0,0044	0,0004
	4	Pipe with mixer № 3	0,0052	0,0005

Conclusions:

1. Static mixers are efficient devices that can increase the slurry turbulence and thus have several advantages, including a large number of possible options for design solutions,

such as the absence of moving parts, drives and power consumption, the possibility of combining with hydro and air-transport network.

2. The investigation of functioning of three static mixer designs of Non-Newtonian Fluid with the following mud parameters: density – 1250 kg/m^3, dynamic viscosity – 0.02 cPs with the use of Flow Simulation module of SolidWorks software environment made possible to obtain slurry parametric fields in the pipe work zone, including the mixer and a pipe section behind it up to 20 pipe diameters long: slurry velocity field *v (m/s)*; vorticity field *n* *(c^{-1})*and turbulence intensity field *I(%)*,turbulence scale field along the length mixing *l_m (m)*.

3. According to the obtained data, the best technological mixing specifications of mud are provided by mixers № 2 and № 3, which are recommended for carrying out in the system of mud preparation at the site of its mixing with the reagents. For them, the maximum pulp velocity is 15.2 – 16 m /s, vorticity is 786 – 706 c^{-1}, the turbulence intensity is 27 – 43.5%, the turbulence scale is 4.4 – 5.2 × 10^{-3} meters.

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MAGNETIC FIELD INFLUENCE TECHNOLOGIES ON HIGH-PARAFFIN CRUDE OIL IN THE PIPELINE WITH DIFFERENT CALIBER

Influence technology of constant magnetic field at high paraffin oil is enhanced to prevent the formation and wax deposition in the tubing and during oil transportation by pipelines of different calibres. Action mechanism determination of directed static magnetic field on high- viscosity paraffin oil wells in Boryslav deposits was investigated by conducting laboratory tests of samples. For Botyslav field conditions the construction of magnetic recovery equipment (MRE) regarding the influence of magnetic induction on heavy oil deposits is designed.

Key words: *paraffin, oil, magnetic induction, well, oil field.*

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ТЕХНОЛОГІЯ ВПЛИВУ МАГНІТНОГО ПОЛЯ НА ВИСОКОПАРАФІНИСТУ НАФТУ У ТРУБОПРОВОДАХ РІЗНОГО ДІАМЕТРУ

Удосконалено технологію впливу дії постійного магнітного поля на високо парафінової нафти з метою запобігання утворення і відкладення парафіну у насосно-компресорних трубах (НКТ) та при транспортуванні нафти нафтопроводами різних діаметрів. Визначино механізм дії спрямованого постійного магнітного поля на високов'язкі парафіністі нафти свердловин Бориславського родовища щляхом проведення лабораторних досліджень їх зразків. Спроектувано, для умов Бориславського родовища, конструкцію антипарафінового магнітного пристрою (МАУ) щодо впливу його магнітної індукції на поклади високов'язкої нафти.

Ключові слова: *парафін, нафта, магнітна індукція, свердловина, родовище.*

Introduction. Flow rate large losses during the operation of oil production wells derived from oil deposition of asphalt-resin-paraffin deposits (ARPD) in the tubing.

Asphalt-resin-paraffin deposits (ARPD) are known as paraffin - buttery or solid masses of dark color. Formation of paraffin deposits reduces oil production, decreases turnaround time of wells, increases labor, material costs and the cost of produced goods. The concept of the formation and control technology with ARPD in oil production is still relevant scientific, technical and practical problem, since this factor directly affects the final performance and well productivity.

Review of the last researches. Researches of scientists and experts have shown that pipeline crystallization of paraffin deposits is not constant criterion. The development of technology and equipment to control asphalt-resin-paraffin deposits has a long history.

However, it cannot be said that all the difficulties are associated with the problem preventing the formation of asphalt-resin-paraffin deposits (ARPD) in the tubing are resolved.

A.A. Abramson, Yu.V. Antipin, G.A. Babalyan, N.F. Bohdanov, D. M. Bohdanov, N.R. Ibragimov, N.N. Neprimerov, A.V. Zguba, A.N. Pereverzev, N.I. Tayusheva, V.P. Troniv, K.R. Urazakov, Z.A. Khabibullin and others had great contribution to the solution of theoretical and practical issues of paraffin deposits.

The researchers also established some regularities of paraffin deposits formation in wells that affect on paraffin deposition under the following conditions:

– reducing pressure in the bottom hole, and thus violation of the hydrodynamic gas-liquid equilibrium system;

– intensifying gas emission;

– decreasing of temperature in the reservoir and borehole;

– speed change of the gas-liquid mixture and its individual components;

– hydrocarbon composition in each phase mixture;

– volume ratio of phases.

Uncertain problems. Today the theoretical and practical issues of paraffin crystallization at various surfaces of pipelines, features and profiles of deposits in tubing while oil transportation by pipelines are not enough studied. The influence of magnetic field on the highly paraffin oil are generally considered selectively and without systematic approach.

Aim of research. Influence technology of constant magnetic field at high paraffin oil is enhanced to prevent the formation and wax deposition in the tubing and during oil transportation by pipelines of different diameters.

The main material presentation. Transportation of oil by pipelines of various diameters and variety of conditions for oil field exploration of high paraffin produced oil requires individual approaches to prevent technology of paraffin deposits in pipelines.

Today, as many decades ago, there are following dominated oil methods: thermal, chemical, mechanical, application of coatings, physical.

These methods with minor modifications, and sometimes without them are applied in wells with different operating. Therefore, in the description of technologies concerning particular way will include only significant differences characteristic of the operation present method.

The above methods are used in different production wells (free-flow, pumping and its varieties), as well as oil transportation by pipelines.

There are many factors that either hinder or contribute to the intensive formation of paraffin deposits.

To the most significant of them we can refer:

1. Flow rate. As the studies have shown, the intensity of the deposits increases at the beginning, with the increase in speed due to the growth in mass transfer, and then decreases, as tangential stresses which increase the bonding strength of paraffin to the equipment surface.

2. The gas factor and the process of gas evolution itself during the pressure decrease. With release and expansion of gas, temperature is lowered, and gas presence in the flow enhances mass transfer, as a result, the proportion of paraffin hydrocarbons crystallizing on the surface of the equipment increases substantially.

3. The presence of mechanical impurities, which are active centres.

4. Crystallization can lead to decrease in the intensity of paraffin deposition by reducing the state of oil super saturation and increase its share of crystallization in the volume.

5. The state of equipment (substrate) surface of has significant effect on deposits strength, in particular, substrate material polarity and surface quality (smoothness). The higher the value of the material polarity and its smoothness, glossiness (purity of treatment), the less is adhesion, and therefor at lower flow rates, paraffin formations will be broken from such surfaces.

6. Water cut of well production. It has twofold effect. First, with a small water amount in oil and other equal conditions, a slight increase in the intensity of paraffin deposits is observed, and then with increasing of water content in the flow, the intensity decreases both due to increase in the flow temperature (the heat capacity of water is 1.6 ... 1.8 times greater than oil), also due to the phase reversal, during which the contact of oil with the equipment surface deteriorates. 7. At the Boryslav field, the most intensive well waxing occurs at rate of 15 to 20 tons per day. With further increase in the production rate, we can observe sedimentation from the underground well equipment surface by gas-liquid mixture flows.

The oil flow sheet in the tubing cavity, at high water cut, for hydrophilic and hydrophobic surfaces is shown in Fig. 1.

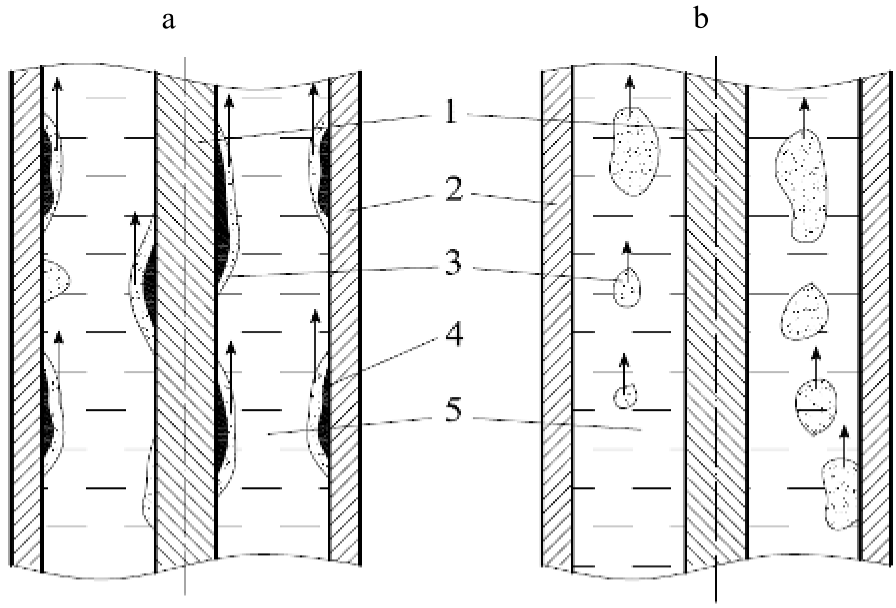

Figure 1 – Scheme of oil flow sheet in the tubing cavity with high water cut:
a) metal surface is hydrophobic; b) surface is hydrophilic;
1 – rod, 2 – tubing, 3 – oil, 4 – ARPD, 5 – water

Therefore, during exploitation of the reservoir, the changes occur in the physical chemical composition of the oil due to several reasons: decrease in reservoir pressure and gas release, hydrodynamic chromatographic section of the oil during its movement in the formation, dissolution of the components in the oil-flow past water and oxidation itemized to the formation with oxygen injected water. Al mentioned leads to change in the thermodynamic characteristics of the formation and the fluid chemical composition, which is the main reason for the formation of asphalt-resin-paraffin deposits in oilfield equipment.

Table 1 – The research results of APRD formation conditions

Depth, meters	Pressure, MPa	Total gradient temperature, °C/100 m	Temperature gradient caused by gas-oil mixture
1400	11,5	1,5	0,14
900	7,5	1,9	0,17
600	5,0	2,1	0,34
200	2,3	1,8	0,55

To prevent and reduce deposits growth using ASPR with the help of directly placed devices in the wells, in most cases devices with permanent magnets are used. These small devices have the best possible performance and virtually unlimited service life, they do not require significant maintenance costs, provide all the necessary range of magnetic treatment performance.

Magnetic devices that are used to prevent ARP deposition, have the most complex usually reversed field distribution, greater length of working channels, high gradient of magnetic field. Increasing of intensity and gradient are the main areas of such devices improvement.

The advantage of such anti paraffin equipment electromagnetic constructions is simplicity of increasing the duration of magnetic action, increasing the number of successively placed current windings and pole pieces. With corresponding increasing in the length of the device its consumption of power increases in proportion. However, despite the high specifications, this lack of general magnetic, closing axile surface of the pole pieces 3, leads to lower tensions of magnetic field and increased energy costs due to increased losses in the dispersed magnetic fluxes.

A scheme of development of a similar electromagnetic devices of Kharkiv National University of Engineering and Economics, with windings inside the pipe is shown in Fig. 2.

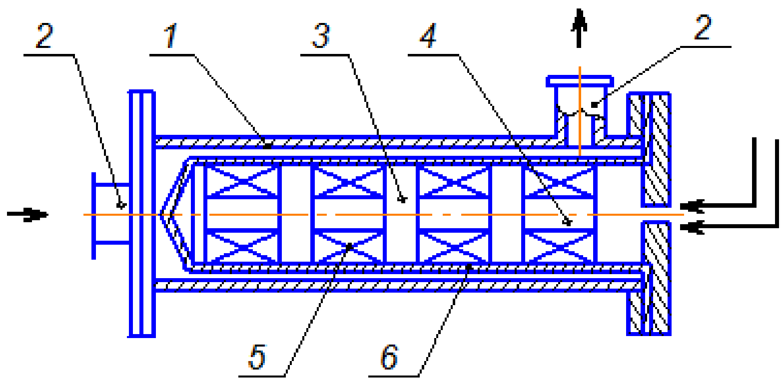

Figure 2 – Electromagnetic devices:
1 – the case; 2 – input and output flanges; 3 – pole pieces; 4 – total magnet core; 5 – current windings; 6 – shell of current windings

This construction has high permeability magnetic shielding outer body with input and output flanges for connection to fluid conduits but high permeability magnetic shielding pole pieces are connected with common axial cores where the current windings are. Near there are windings switched on towards each other on the treated fluid protected with nonmagnetic material covering. Annular gap between the outer surface of pole pieces and the inner surface of the body is working channel for fluid. System of current windings and pole pieces in the gap form reverse alternating magnetic field, which is used for magnetic treatment of liquid, and presence of the unifying magnetic pole piece reduces the total energy loss in scattering magnetic flux, thereby improving device energy parameters. Performance of various known devices ranges from 2 to 100 m3 / h, the maximum strength of magnetic field in the working channel 96 kA / m (magnetic induction 0.12 T), the channel length of the current magnetic field of 0.3 to 3 m. To receive reverse magnetic field to 6 current windings are used, where specific energy consumption is from 6 to 35 W / m3. If the length of the gap *g* between the outer surface of the pole pieces and the inner surface of the outer body-tube is less axial thickness of the pole pieces, high tension *H* of magnetic field in the working gap of such devices can be roughly determined from the equation:

$$H_{ñð} = \frac{I \cdot n}{2 \cdot g},$$

where *I* – amperage, and *n* - the number of turns in each winding.

The proposed magnetic anti paraffin equipment (MAE) does not have these disadvantages. Because of usage of pipeline cores figurine-shaped magnetic the system covers only small section of the pipeline. Number of magnets is small, magnetic treatment provide effective total fluid volume that passes through the device in the same conditions of high-gradient field with sufficient treatment duration.

Permanent cylindrical chain 1 mounted in the pipeline 2, has reversible axial magnetization created so that the outer poles have the same polarity and the opposite polarity formed in the middle of its length. The working channel for the fluid is the gap cross section between the outer surface of the pipe 2 and the outer pressure vessel 3. In order the fluid enter to the channel and out of it after treatment in the wall pipe 2, near the ends of the magnet 4 holes are made.

Fig. 3 shows schematic diagram of the proposed MAE.

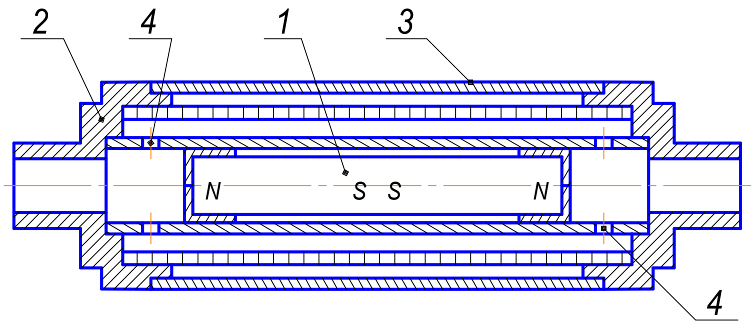

Figure 3 – Magnetic anti paraffin equipment (MAE) for liquid magnetizing:
1 – reversible magnetized permanent magnet; 2 – the pipeline;
3 – outer casing; 4 – holes for fluid input and output.

When using high permeability outer casing 3 in a device of this type, reverse magnetic field can be obtained (with a small reverse number) that predominantly is perpendicular to the direction of fluid flow force lines with high intensity and gradient. However, it is difficult to obtain large areas of high gradient length fields. Devices of this kind are easy to implement by means of the pipe 2 sequence of magnetization directions, alternating axially magnetized magnets. Significant increase number in field reverses number does not represent difficulties. As it can be seen further, on this principle very large number magnetizing devices are made. For all these devices the main purpose are magnetic water treatment and water systems. Obviously, the consistent deployment of any other above magnetic devices with permanent magnets, they can be converted to multiblock where many reverse magnetic fields can be received. Due to this fluid duration, treatment is significantly increased.

Conclusions. Therefore, influencing technology of magnetic field at high paraffin oil by magnetic anti paraffin equipment (MAE) proposes figurine-shaped magnetic cores use. Number of magnets is small, but it ensured the effectiveness of magnetic treatment of fluid entire volume passed through the device in the same conditions of high-gradient field with sufficient treatment duration.

The equipment (MAE) can work in pipelined of different diameter thus providing the necessary magnetic field strength and magnetic induction. It has small magnet mass, requires lower cost, comparatively to other similar devices.

The equipment (MAE) differs from other magnetic devices for fluids treatement that there is no need for any power supply to the unit, as it is based on permanent magnets.

Magnetic field influencing technology usage at high paraffin oil by magnetic anti paraffin equipment (MAE) shows the necessity to use such equipment either in free flow wells with high content of asphalt resign paraffin particles (especially in winter), or at the operation of deep- centered rod and diaphragm pumps, as well as oil transportation by pipelines of different diameters.

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CONDITIONS OF A SINGLE INTAKE DESCENDING OF INTERMEDIATE CASING STRINGS OF AN EXCESSIVE WEIGHT COMPARED TO A DERRICK LOAD CAPACITY

The way of the intermediate casing string descending is considered. It allows overcoming the limits of derrick load capacity through the weight transferring descending intermediate casing string on the cemented string that is based on the foundation. In particular, technology is developed where the weight of each additional string, as it builds-up, is equivalent to additional friction forces which arise in the enlarged area. Such a descent technology does not depend on the load capacity of the derrick load capacity, does not create loads on the basis of derricks as well as additional tensile loads in the string with an increase in its length. It is proved that if such a condition is met, all known restrictions for the descent of intermediate columns of any length in a single intake are removed. Authors determined descending casing string weight that is distributed over the previously installed and cemented string. It is suggested to distribute casing string weight on existing and cemented string.

***Keywords:** drill string, borehole, drilling fluid centralizer.*

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УМОВИ СПУСКУ В ОДИН ПРИЙОМ ПРОМІЖНИХ КОЛОН З ПЕРЕВИЩЕНОЮ ВАГОЮ ПОРІВНЯНО З ВАНТАЖОПІДЙОМНІСТЮ БУРОВИХ УСТАНОВОК

Розглянуто спосіб спуску проміжних колон, котрий дозволяє подолати обмеження за вантажопідйомностю бурових установок за рахунок перенесення ваги колони, яка спускається на зацементовану колону як на фундамент. Зокрема, розроблено технологію, коли кожна додаткова вага колони при її нарощуванні еквівалентна додатковим силам тертя, які виникають на нарощеній ділянці. З'ясовано, що така технологія спуску не залежить відвантажопідйомності бурової установки, не створює навантажень на основу веж, а також додаткових розтяжних навантажень у колоні при збільшенні її довжини. Доведено, що, якщо така умова виконується, то всі відомі обмеження для спуску проміжних колон будь-якої довжини в один прийом знімаються. Установлено вагу обсадної колони, що спускається, котра розподіляється на раніше встановлену і зацементовану колону.

***Ключові слова:** колона, свердловина, буровий розчин, центратор.*

Introduction. An intermediate casing string runs between the conductor and the drill string and serves for blinding the complicated zones or horizons overlying the design depth. Depending on the number of intermediate casing strings, the well is generally designed as a multi string. Most of the casing strings are descended into a well in one step. However, often at high depths, the string is lowered by parts (sections), respectively, into two or three steps. It is done in cases where the weight of the casing exceeds the lifting capacity of the drilling rig if, due to the insufficient strength of the casing, it is impossible to form a solid string for tension; if it is necessary to divide the string into parts due to the risk of complications occurrence. The length of the lower part (section) is chosen so that its tip would be above the shoe of the previous string at 50-100 m or 200 m higher than the possible zone of complications in the range of resistant rocks formations.

A characteristic feature of some gas deposits is the presence of abnormally high reservoir pressures, as well as massive gas deposits within thick gas-bearing formation. When constructing wells at gas fields it is necessary to consider its specific features:

1) the elasticity and compressibility of the gas that saturates the washing liquid during the drilling;

2) gas higher mobility and permeability compared to oil and water;

3) significantly higher pressure along the borehole from the wellhead to the bottom hole, compared to oil wells at uniform reservoir pressures;

4) high flow rates and gas velocities in the operation of gas wells, causing significant losses of formation energy. In order to prevent the disruption of a borehole in gas fields, the depth of the intermediate casing string descent should be deeper than that one of the oil fields.

Decreasing in a number of intermediate casing strings allows saving the time and resources in the process of a well completion. The necessity of descending each string severally is caused, basically, by an insufficient load-carrying capacity of derricks. Also, casing pipes use of domestic and sometimes foreign production cannot withstand the loads from a dead weight of long casing string. Nowadays the technology of descending the heavy casing strings is known. It uses heavy hydraulic jacks installed at a wellhead. But it solves the problem partially since the restriction of a maximum length of a casing string is still there.

To prevent the gryphon occurrence, before the perforation in gas-bearing or pressure formations, it is necessary to use the conductor or an intermediate casing string to cover the whole soil unit capable of absorbing the washing liquid and through which it is possible to for gas to leak into a surface. For gas fields with a thick gas formation and abnormally high reservoir pressure, the number of intermediate casing strings and the position of their liners (shoes) should provide a drilling without absorption of washing liquid and associated emissions and blowouts.

Analysis of recent sources of research and publications. In prosperous countries, the drilling is carried out by using the drilling rigs of high carrying capacity which allows the descending of heavy casing strings in a single intake. For example, at the well No. 1 at Bertha Rogers (USA), a 426 mm casing string with 6580 kN of weight was descended to the depth of 4202 m [1]. In case of an excess of the column weight over the load capacity of a rig, a special mobile equipment is used, it is equipped with the powerful hydraulic jacks with a high speed of its bearing supports. With this equipment at the well No. 1 in Medoip (USA) a 508-millimeter casing string with length of 3800 m and weight of 9500 kN was successfully descended [2]. The maximum depth and weight of the column, which is descended using this equipment, are determined by the load of threaded joints.

In our country, the methods of heavy casing columns descending have been developed and applied by the way of their weight decreasing, mainly by increasing the pushing out force. There are a number of technologies for descending heavy casing strings. The simplest way is the descent of the casing strings with the replacement of the drilling mud in it for a lightweight liquid with partial emptying. However, in this case, upon the restoration of the drilling mud circulation, there is a violation of the hydrostatic balance in the well. In the annular space, the density of the drilling fluid decreases, and in the pay zone, there are conditions created for the formation of the reservoir fluids inflow and its further extraction to the surface, and hence the conditions of water and gas oil manifestations and related complications. This inevitably causes the borehole stability deterioration.

Given the fact that the weight of the column increases by the weight of the volume of additional fluid, during the upward movement, the forces of resistance are added (10 - 15% for vertical wells). There are difficulties in undermining the column at the time of its extending with the next pipe. To go beyond the capacity of the drilling rig, it is necessary after the circulation to empty the column again to the previously achieved value, using a compressor and descent of a drill string [3]. The allowable amount of emptying during the descent of columns with impenetrability is limited to the condition of four-fold strength of pipes for crushing. The above limitations do not allow the heavy casing strings to be lowered in a single gear.

Identification of general problem parts unsolved before. An important role in the well drilling is the right choice of well design, which would ensure the creation of a durable and hermetical channel, stability of the borehole during the entire period of production, preventing the inner flow, and also allow performing well repair work. To reduce the weight of the casing string on the hook, the device was proposed to create an additional frictional force when descending the string into the well [4]. Friction governed by backfilling sand or other granular material that fills the annular space between the casing that goes down and the cylinder. The sand falls into the open upper cylinder funnel and pours off at the lower end through a series of narrow openings. The rate of sanding is regulated by compressed air. In the upper part of the cylinder, a hydraulically controlled telescopic device is provided for the possible reduction of the annular space intersection, if the speed of the column rise increases.

The creation of significant frictional forces provides the necessary clamping force [5]. When it is in accordance with the weight of the string, the device must be of a considerable length, which is difficult to achieve. Significant work on reducing the sections number of casing strings was carried out by the laboratory of techniques and technology for wells completion of the State Research Institute of Drilling Technology. Thus, a 340-millimeter intermediate casing string was descended at a depth of 4600 m by three sections instead of five according to the project. It is summarized and analyzed existing methods of facilitation of casing strings during their descent into wells, as well as possible variants of cementation. Nonstandard technologies of columns descent have been worked out, the realization of which is promising, but technically complex and has a number of limitations [6, 7]. Such technologies include the "pontoon" method of releasing heavy casing strings and the descent of the string by increasing the density of the fluid in the annular space.

In general, the goal is to develop the issue of reducing the sections number of casing strings. It has been developed in two directions:

- increasing of lifting capacity of drilling rigs, the creation of special jacks for descending the casing strings;
- increasing the lifting force acting on the string in a liquid mud.

The first way allows to execute the descent of the heavy casing strings and has only one limitation for the load in the threaded joints. When applying welded strings - is the limiting tensile stresses on the pipe body.

The second way is technically more complicated and requires additional casing installation, which adds additional restrictions. The first method of descending the casing in a single intake is real and limited by loads. In the second method, the sections number of the casing descending decreases by 30-60%.

Basic material and results. The descent of the string should be done after conducting geophysical surveying, preparatory works. Inspection and preparation of the casing elements were carried out primarily on the tube base. After a visual inspection, the pipes were subjected to instrumental control, with defectoscopic installations, calibers. The total length of the imported pipes by 5% exceeded the length of the casing. The reserve consists of the most robust pipes. Before the descent of the casing, the well was washed for 2-3 cycles. The rise of the casing pipes on the drill was carried out in the presence of protective rings for a thread. Each tube is numbered, measured and verified by external inspection. A template was passed through each tube.

The previous assembling of the productions string was carried out with the AKB-3M tool, smooth rotation of the casing from its capture to the complete stop, and then the quality of assembling was checked on the number of complete threads that remained above the coupling.

Fastening of the threaded joint was carried out with AKD-3M tool with maximum short-term torque. In the normal threaded connection, the last thread coincides with the end of the sleeve, a deviation of 1 thread is allowed. Intermediate washings were carried out from a depth of 1200 m every 300 m at least one cycle. In the process of flushing-out of well, pressure, circulation, and parameters of the drilling mud, level in the inflow containers should be controlled.

It is proposed another way to descend the intermediate casing strings, which allows overcoming the limitations of the drilling rigs lifting capacity by transferring the weight of the casing string to the cemented column as a foundation. Using this approach to the task of descending into of intermediate strings in a single operation, the weight of which exceeds the capacity of drilling rigs, there are no restrictions as to the carrying capacity of drilling rigs, as well as for the loads. Thus, the weight of the column on the hook can only be a part of its actual weight.

The implementation of such technology can be carried out by various technical means.

The essence of the technology lies in the fact that each additional weight of the string at its build-up is equivalent to the additional forces of friction that arise in the enlarged area. Such a descent technology does not depend on the load capacity of the drilling rig, does not create loads on the basis of the derricks, as well as additional stretching forces in the string at an increase in its length. If this condition is fulfilled, then all known restrictions for descending the intermediate casing strings of any length in one step are removed.

With this approach, the weight of the lowered casing is divided into a previously installed and cemented casing string, the action of the distressed loads moves down under the shoe of a previously installed string of a larger diameter. The diagram of tensile stresses in the string changes.

It is considered the technology of the descent of a heavy casing by an example of a hypothetical well.

In the well to a depth of 6800 m, it is necessary to lower the casing string with a diameter of 245 mm with a wall thickness of 12 mm of strength group M, which weighs 4760 kN. The string consists of casing pipes OTTM1 with a normal diameter of the coupling, a tolerable stress of 3870 kN and a limiting weight in the wedge holds of 4350 kN. The descent is carried out by a drilling machine with a carrying capacity of 2000 kN with a 324-millimeter column set in the well at a depth of 3600 m.

The descent of a 245-millimeter column weighing 700 kN is based on standard technology at a depth of, for example, 1000 m. Further descent is carried out on a rigid elastic centralizer (Fig. 1), each of them, as calculations show, in the gap of strings with diameters of

245 and 324 mm, can withstand a weight of 12 - 16 kN with a coefficient of friction «metal – metal» 0,17. When installing the centralizer while building-up the string, the rest of the weight can be unloaded to a 324-millimeter string.

To make it possible to break the column of wedges, a telescopic connection is established in a 245-millimeter string with a stroke of up to 0.5 m. The weight of the string under the telescope must be zero or less at the design depth and the casing string should completely hang on the centralizers.

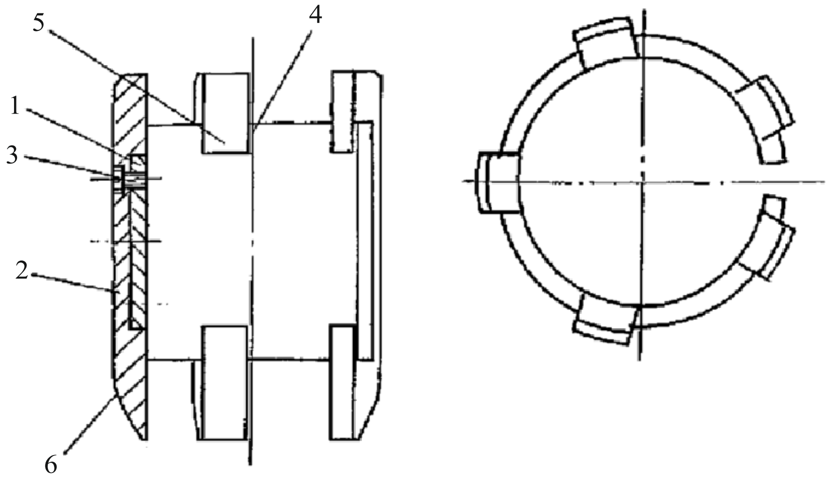

Figure 1 – Unloading centralizer for well pipe strings:
1 – spring element; 2 – guide lining; 3 – screw;
4 – slots; 5 – ledges; 6 – chamfers

It is allowed to not install telescopic string connection, but use the tension of the string to remove the wedges due to its lengthening. At the same time, only the part of the string whose weight is located on the hook will lengthen, and the lower part will be unloaded on the centralizers.

When the 245-millimeter string of 324-millimeter shaft leaves the open barrel, the weight of the 245-millimeter string will be increased by the weight of the column part which hangs on the centralizers. The centralizers will not work in a barrel with a nominal diameter of 295 mm.

By extending the string with the installation of the toruses center, it is possible to compensate the weight of the column in an open barrel length of 3200 m and leave the convenient weight to work with (approximately 200 - 300 kN) to hang on the hook.

It should be noted that the actual depth of the descent of a 245-millimeter column is equal to the load in an open barrel from a 324-millimeter column.

In this case, it is considered a 245-millimeter casing string with weight of 6390 kN (3870 kN from a boot of a 324-millimeter string plus 2520 kN in a 324-millimeter string) at a length of 9100 m in one step.

In addition, due to the lifting force of a drilling mud with the density of 1200 kg/m^3, the length of the string can be increased by 15,28%. If steel pipes of the grade R-110 were used in an open barrel, the weight of the string, lowered in one step by a drilling machine with a carrying capacity of 2000 kN, can reach 8710 kN, which is equivalent to 12443 m. In the Kola extra-deep well, it is possible to lower a 245-millimeter string in one step to a depth of 14344 m with a drilling density of 1200 kg/m^3.

The necessary and sufficient condition for the descent of the column in one step is the stability of the well bore during the descent.

Conclusions. The proposed method allows lowering heavy casing strings into a well in one step. The maximum weight of the string does not depend on the load capacity of the drilling rigs. The weight of the string is limited by the carrying capacity of the threaded joints of the string part, which is in the open barrel, and the weight of the string to the bottom of the previously lowered string.

The benefits of the lowering a casing string in one step include: no need in using the admission tool, which by its own weight limits the weight of the section; no need in equipping the string with connecting devices, as well as the use of disconnectors; reducing the wear of the columns in the docking areas where it reaches the highest values; the parallelism of the strings creates normal conditions for cementing; changing of the forces diagram in the string; more favorable conditions for the column sealing; no need to tension the string; shortening the borehole. An essential disadvantage is the lack of the strings ability to «wander».

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MATHEMATICAL MODELING OF THE THERMODYNAMIC PROCESS GAS-STEAM BUBBLES

A mathematical model that considers the inertial oscillations and thermodynamic components bubbles in liquid heat exchange processes, heat transfer on the boundary bubbles. Research of the dynamic characteristics of gas-steam bubbles in various size was conducted. After the calculations its temperature, velocity, pressure steam environment inside the bubble in time, graphs bubbles size change graphs were built . It is established that each bubble size has its oscillation frequency. Calculated speed phase transients and found that it is in its maximum during the bubble oscillation. For thermodynamic properties of the surface of contact liquid and gaseous phases defined amount of solid phase formed. The research results can be applied to optimize various of technological processes related to the boil, swelling materials, and the formation of gas hydrates in a fluid cavitation.

***Key words:** mathematical model, bubble, heat exchange, gas hydrate, pressure, temperature.*

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МАТЕМАТИЧНЕ МОДЕЛЮВАННЯ ТЕРМОДИНАМІЧНИХ ПРОЦЕСІВ ГАЗОПАРОВОЇ БУЛЬБАШКИ

Запропоновано математичну модель, яка враховує інерційну та термодинамічну складові осциляції бульбашок, теплообмінні процеси у рідині, теплообмін на границі бульбашки. Проведено дослідження динамічних характеристик газопарових бульбашок різних розмірів. Після виконаних розрахунків побудовано графіки зміни розміру бульбашки, її температури, швидкості руху, тиску парогазового середовища всередині бульбашки в часі. Установлено, що кожний розмір бульбашок має свою частоту осциляцій. Розраховано швидкість фазовоперехідних процесів і встановлено, що вона набуває максимальних значень саме під час осциляцій бульбашки. За термодинамічними характеристиками поверхні контакту рідкої та газоподібної фаз визначено кількість утвореної твердої фази. Результати досліджень можуть застосовуватися для оптимізації різноманітних технологічних процесів, пов'язаних з кипінням, спученням матеріалів, утворенням газових гідратів та кавітацією у рідині.

***Ключові слова:** математична модель, бульбашка, теплообмін, газовий гідрат, тиск, температура.*

Introduction. One aspect of the use of advanced technology gas storage is the possibility of building gas storage near large consumers (boiler, CHP). Structurally, a storage battery consists of gas tanks placed in the pit or in the hangar. In spring and summer filled with gas storage, which forms the clathrate structure, and in autumn and winter - give them gas at expansion by using low-grade heat source. Building of such storage facilities near CHP can significantly smooth out seasonal unevenness of gas consumption and is a real alternative to the construction of underground gas storage [1].

Technologically, the accumulation of gas occurs in a dissolution of gas bubbles in water under certain of thermobaric conditions [2]. The process of formation of solid phase takes place on the surface of the oscillating bubbles. During the oscillation is very quick change of thermodynamic parameters of the system «gas-steam bubble – liquid». The study of the dynamics of the process to determine the most influential factors to optimize the production process [3] and reduce the volume construction and installation works. Direct observation of phase transients on the surface of the bubbles is a complex engineering and technical challenge: high pressure (up to 20 MPa), small bubbles ($10^{-5}\div10^{-7}$ m) and the high rate of oscillation (10^{-5} s). Easier way is mathematical modeling of heat exchange and mass transfer processes on its surface.

Review of recent research sources and publications. To analyze the dynamics of growth of vapor bubbles commonly used equation Rayleigh-Plasseta [4-6]. To determine the vapor pressure inside the bubble is often used Clausius-Clapeyron equation [7, 8], or consider adiabatic process [9]. According to other researchers, the processes inside the oscillating bubbles are not limited to the phase transition or the lack of heat exchange at the surface bubbles. In [10] the mathematical formulation of the problem more fully. In addition to the equation of Rayleigh-Plasseta it contains the equation of van der Waals forces to determine the pressure within the gas-steam bubbles and allows you to calculate the temperature of gas inside the bubbles based on the first law of thermodynamics. Also added a mathematical model heat and mass transfer across the border bubbles. However, this mathematical model does not account for the impact of traffic on the wall near her heat exchange processes and phase transitions in liquid.

Selection not solved earlier of parts the general problem. For the modeling the transition processes phase on the surface of the bubbles should consider the possibility of dissolution of the gas bubbles in the liquid while liquid phase transition in the solid phase. This process is determined by the rate of heat and mass transfer processes at the surface of the bubbles, which in turn depends on the temperature and pressure inside the gas-steam mixture bubbles. In speed mode cavitation bubbles to change the size of some times can reach several hundred meters per second, which significantly affect the course of the heat - and mass transfer processes at its border. As a result of these processes, thermophysical characteristics of liquid on the boundary of the bubble can also significantly vary. Thus, for a correct formulation of the problem should adequately take into account the complex interrelated mechanical and thermodynamic processes that occur in a limited volume at high speed.

The problem statement. The aim of this work is to create a mathematical model of the dynamics of vapor bubbles that will get reliable information about its thermodynamic characteristics during growth or compression. In general, the mathematical model should include the following components: model the kinetics of gas bubbles in a viscous fluid; model thermodynamic processes inside the gas-steam bubbles; model of heat - and mass transfer processes in border vesicles; Modeling of phase transitions in liquid form with ice or other of solid phase; modeling heat transfer processes in the liquid surrounding the bubble.

Basic material and results. To develop mathematical models of gas-steam bubbles in the liquid is applied following simplifying assumptions: gas-steam bubble has a spherical shape; fluid is viscous and incompressible; gas-steam bubbles inside is a mixture of gas and vapor fluid whose mass may change as a result of mass transfer processes in border bubbles; Gas and vapor bubbles of fluid in the middle considered as real gas (including van der Waals forces).

The velocity of the fluid ($\dot{R}$) on the border of bubbles can be determined by integrating the known equation of Rayleigh-Plasseta [4]. At some times the pressure inside the bubbles can rise sharply and describing its thermodynamic state must take into account the difference of the parameters of state of ideal gas. To determine the partial pressures of the components of gas-vapor mixture, it is advisable to apply the equation of van der Waals forces [10, 11]. In general, the mathematical model of thermodynamic processes of gas-steam bubbles containing the following equation:

$$\frac{d\dot{R}}{d\tau} = \frac{P_{B(\tau)} - P_\infty}{\rho_r R} - \frac{1.5}{R}\dot{R}^2 - \frac{4\mu_r}{\rho_r \cdot R^2}\dot{R} - \frac{2\sigma_r}{\rho_r \cdot R^2}, \tag{1}$$

$$\frac{dR}{d\tau} = \dot{R}, \tag{2}$$

$$P_B = P_g + P_p, \tag{3}$$

$$P_g = \frac{R_\mu T}{\frac{\mu_g}{\rho_g} - b_g} - \rho_g^2 \frac{a_g}{\mu_g^2}, \qquad P_p = \frac{R_\mu T}{\frac{\mu_p}{\rho_p} - b_p} - \rho_p^2 \frac{a_p}{\mu_p^2}, \tag{4}$$

$$\frac{d\rho_g}{d\tau} = \frac{3}{R}\left(-\rho_g \frac{dR}{d\tau}\right), \qquad \frac{d\rho_p}{d\tau} = \frac{3}{R}\left(-\rho_p \frac{dR}{d\tau}\right), \tag{5}$$

$$\frac{d\left(m_g c_g + m_p c_p\right)T}{d\tau} = 4\pi R^2 q - P_B \frac{d\left(4/3\pi R^3\right)}{d\tau}, \tag{6}$$

$$\frac{dT}{d\tau} = \frac{3}{R\left(c_g\rho_g + c_p\rho_p\right)}\left[q - P_B\frac{dR}{d\tau}\right], \tag{7}$$

$$q = q_1 + q_2 - q_3 - q_4, \tag{8}$$

$$q_1 = 0.25\rho_p\left(u_{p(T)} - \dot{R}\right)c_p T_{(R,\tau)}, \tag{9}$$

$$q_2 = 0.25\rho_g\left(u_{g(T)} - \dot{R}\right)c_g T_{(R,\tau)}, \tag{10}$$

$$q_3 = 0.25\rho_p\left(u_{p(T)} - \dot{R}\right)c_p T, \tag{11}$$

$$q_4 = 0.25\rho_g\left(u_{g(T)} - \dot{R}\right)c_g T, \tag{12}$$

$$u_{p(T)} = \sqrt{8R_\mu T/\mu_p \pi} \text{ and } u_{g(T)} = \sqrt{8R_\mu T/\mu_g \pi}, \tag{13}$$

$$\frac{\partial\left(\rho_r c_r T_{(x,\tau)}\right)}{\partial\tau} + \dot{R}\frac{\partial\left(\rho_r c_r T_{(x,\tau)}\right)}{\partial x} = \frac{1}{x^2}\frac{\partial}{\partial x}\left(\lambda_r x^2 \frac{\partial T_{(x,\tau)}}{\partial x}\right) + q_{v(x,T)}, \tag{14}$$

$$-\frac{\partial\left(\lambda_r T\right)}{\partial x}(x = R, \tau) = -q, \tag{15}$$

$$T_{(x,\tau=0)} = T_0. \tag{16}$$

where $\dot{R}$ – speed liquid bubbles on the boundary, m/s;
τ – time, s;
$P_{B(\tau)}$ – pressure gas-vapor mixture inside the bubbles, Pa;
P_∞ – fluid pressure, Pa;

ρ_r – fluid density, kg/m^3;
μ_r – dynamic viscosity fluids, Pa·s;
σ_r – the surface tension on the boundary liquid gas, N/m;
P_g – the partial pressure of gas, Pa;
ρ_g, ρ_p – under the density of gas and steam, kg/m^3;
R_μ = 8314 – universal gas constant, J/(кmol·K);
μ_g – molecular weight gas, kg/кmol;
μ_p – molecular weight liquid vapor, kg/кmol;
T – the temperature of the gas mixture in the bubble, K;
a_g, a_p – constant Van der Waals forces respectively for gas and steam, (N·m^4)/mol^2;
b_g, b_p – constant Van der Waals forces respectively for gas and steam, m^3/mol;
m_g, m_p – weight respectively gas and pairs, kg;
c_g, c_p – mass heat capacity of the gas and water vapor, J/(kg°C);
q – specific heat flux at the surface bubbles, W/m^2;
$T_{(R,\tau)}$ – surface temperature bubbles, °C;
c_r – heat capacity of liquid, J/(kg°C);
λ_r – fluid conductivity, W/(m°C);
q_v – volumetric power sources or waste water heat, W/m^3.

As a result, phase of the transition processes on the boundary bubbles liquid can change its thermophysical characteristics (λ_r, ρ_r, c_r).Volumetric heat source considers phase transitions in liquid medium.

For the formation of the solid phase certain conditions must be fulfilled, the partial pressure of gas exceed the minimum pressure phase transition at a given temperature. The weight solids are determined by removal of the transition region phase transition heat. The intensity of the heat sources for volume phase transition are adopted linearly proportionally to the temperature difference between the surface and the phase transition temperature equilibrium solid phase. 1-16 system of equations can be solved by using digital techniques such as Runge-Kutta 4th order [12, 13].

In the proposed mathematical model computer program «RELEY41» was compiled and behavior of methane bubbles of different sizes under these initial conditions was investigate Methane bubbles from water vapor impurity in the water at the thermobaric conditions sufficient to hydrate formation were investigated. Size of bubbles effect on the rate phase-transition process was defined. To accomplish this fact calculations of thermodynamic properties of bubbles with radii 2 0,5 0,1 0,05 0,01 mm were done. The calculation results are shown in figure.1÷5.

For the initial bubbles radius of 0.1 mm. the results of calculation of the radius and mass of solid phase formed over time is shown in Figure 1. During initial isothermal conditions. Obviously, the most intensive formation of solid phase occurs during the oscillation process.

Analyzing Figure 2. there are three characteristic temperatures bubbles: warm region, the region damped oscillations and fixed area. Bubbles of small size are heated very quickly and this area of the graph we cannot see. Region warming is noticeable only for relatively large bubbles. Due to the intense of heat removal in this area there is the highest rate of phase the transition processes, but it has low duration and so the amount of solid phase formed is small.

In contrast to the cavitation process where the velocity of the bubbles wall reaches hundreds of meters per second, bubbles with gas have much lower velocity wall, Figure 3. Initial velocity is determined by the initial conditions, and eventually dies speed (due to the loss of energy to friction and heat exchange with the environment) more quickly than smaller bubbles.

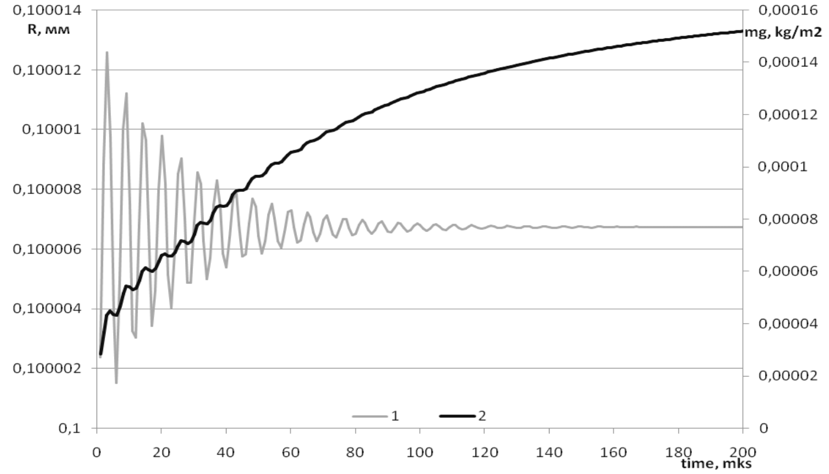

Figure 1 – Change of the radius of bubbles and the specific weight of solid phase:
1 – the radius of the bubbles (R), mm; 2 – the amount of solid phase (mg), kg/m^2

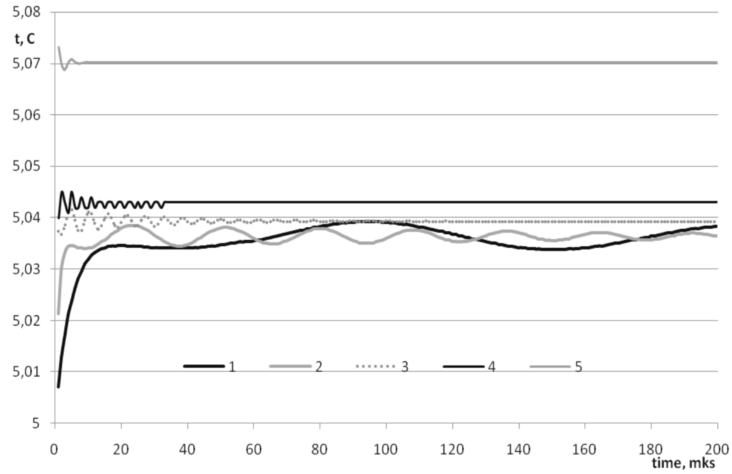

Figure 2 – Temperature of gas-steam bubbles environment:
1 – 2 mm, 2 – 0,5 mm, 3 – 0,1 mm, 4 – 0,05 mm, 5 – 0,01 mm

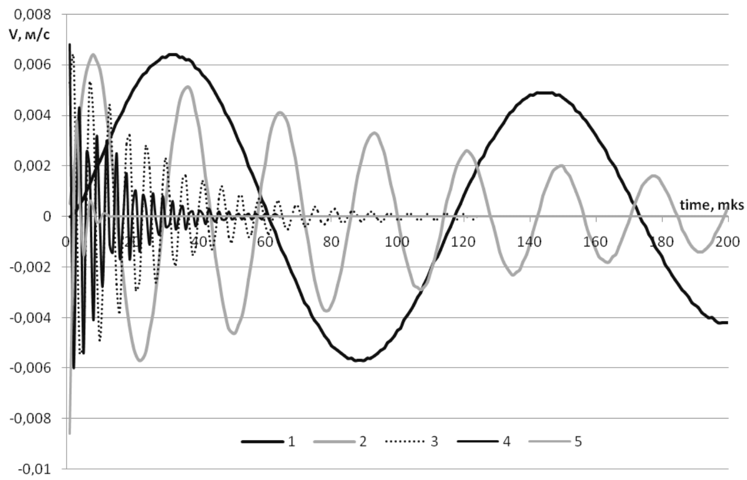

Figure 3 – The rate of change of radius of bubbles for bubble radii:
1 – 2 mm, 2 – 0,5 mm, 3 – 0,1 mm, 4 – 0,05 mm, 5 – 0,01mm

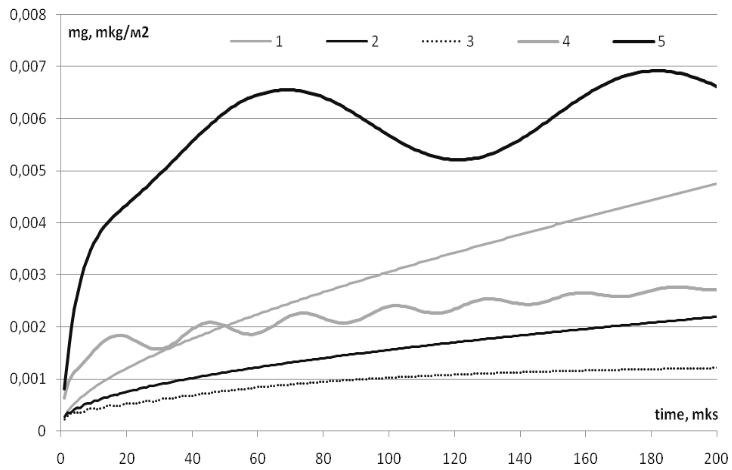

Figure 4 – The specific amount of solid phase that formed on the surface of bubbles of different sizes
1- 0,01 mm, 2- 0,05 mm, 3- 0,1 mm, 4- 0,5 mm, 5- 2 mm

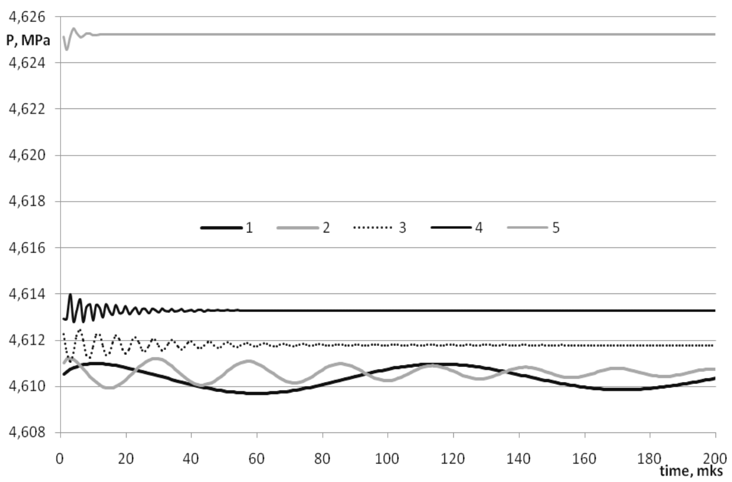

Figure 5 – Graphs of pressure changes bubbles of gas-steam environment
1 – 2 mm, 2 – 0,5 mm, 3 – 0,1 mm, 4 – 0,05 mm, 5 – 0,01mm

Analysis of the dependency indicates the significant influence of two factors on the rate of solid phase, the size of bubbles and its oscillations. Reducing the size of the bubbles leads to increase in their total surface area. However oscillations of small bubbles rapidly damped (range 0.01-0.1 mm) and do not create significant impact on the phase transition processes.

In large bubbles (radius 2 mm) dominant influence on the phase transition processes are oscillatory parameters. Their damped of oscillations continued relatively long time and during that time formed the main part of the solid phase. However, large bubbles have low total surface area of contact interface.

The pressure in the middle of the bubbles is another important factor for mass transfer, Fig. 5. At the same initial temperature conditions pressure low inside the bubbles is greater. This phenomenon is caused by surface tension, also it intensifies the process of phase transition.

Conclusions. It the investigation mathematical model for comprehensive consideration of the impact of various factors on the thermodynamic state oscillating gas-steam bubbles is achieved. At the beginning of solid phase oscillations formation (damped oscillations) bubbles are observed. The starting mechanism for these fluctuations is the temperature difference between the gas-steam bubbles and medium temperature phase transition, which is determined by the pressure medium. During solidification of the liquid phase, and then, through heat exchange and steam bubbles environment locally increasing liquid temperature occurs . Increased gas temperature leads to increased pressure bubbles and begins the process of increasing its diameter. Each size has its bubbles frequency oscillations. Gradually, viscous fluid damped oscillations and phase transition process is supported by heat removal in the outer layers of liquid. At the time of the oscillations top speed of solid phase formation was observed. Over time, the speed phases in the transition processes decreases gradually. The research results can be applied to optimize technological processes related to the boil, swelling materials, and the formation of gas hydrates in fluid cavitations.

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DIRECTIONS AND MECHANISMS OF INNOVATIVE HIGH-TECH AND ORGANIZATIONAL DEVELOPMENT OF REAL ECONOMY SECTOR

It is given the results of researching problems and prospects of real economy development and its leading sectors of economic activity – industry and construction. It is established the causes of development absence and inefficient functioning of these areas of management. On this basis the direction of innovation and high-tech industrial and organizational development and building complexes of the countr were determined. Modern mechanisms for the implementation of such strategic tasks at the expense creation and progress outrunning of domestic enterprises that are capable to use modern standards and strategic project management, the latest achievements of science, engineering and technology to develop and implement various investment projects and development programs were proposed. It is proved that research results should promote not only the progress of the real economy, industry and construction, but also to increase global living standards in Ukraine.

Key words: *innovation and high-tech development of the real economic sector, industry and construction, strategic and project management.*

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НАПРЯМИ І МЕХАНІЗМИ ІННОВАЦІЙНО-ВИСОКОТЕХНОЛОГІЧНОГО Й ОРГАНІЗАЦІЙНОГО РОЗВИТКУ РЕАЛЬНОГО СЕКТОРА ЕКОНОМІКИ

Наведено результати досліджень проблем і перспектив розвитку реального сектора економіки та його провідних галузей господарювання – промисловості й будівництва. Встановлено причини відсутності розвитку та неефективного функціонування зазначених сфер господарювання. На цій основі визначено напрями інноваційно-високотехнологічного й організаційного розвитку промислового та будівельного комплексів країни. Запропоновано сучасні механізми реалізації такого стратегічного завдання за рахунок формування й випереджаючого прогресу вітчизняних підприємств. Доведено, що результати досліджень мають сприяти не тільки прогресу реального сектору економіки, промисловості та будівництва, а й зростанню до світових стандартів рівня життя населення України.

Ключові слова: *інноваційний і високотехнологічний розвиток реального сектору економіки, промисловості та будівництва, стратегічний і проектний менеджмент.*

Introduction. Decreasing of scientific-technical, technological and economic potential of the national economy, absence of real reform and inconsistent of government policies in socio-economic sphere development – all this led to the current crisis occurrence. Today Ukraine takes 161 place in the world by economic freedom index. Another indicator of modern development that reflects the country's place in the global economy is its international competitiveness. By this indicator, the state takes 82 place among 133 countries of the world, including such indicators as «institutions» – 120 «development of finance market» – 106 «product market efficiency» – 109. In the field of higher education Ukraine takes 45th place in the field of innovation development – 52nd, by the level of advanced technologies use – 65th , in the sphere of intellectual properties protection – 114th.

The events of the last two years, signature the association agreement between Ukraine and the European Union (EU) showed that the society is determined move to global stable standards and innovative development of the country in all life spheres, management and business. In these circumstances, the task of forming a new organization and management model of the further real economy development, its leading economic sectors, industry and construction based on the standards of the strategic, multi-project and project management which successfully used in leading countries and world companies, primarily in US, EU, Japan, Singapore, South Korea, Canada and so on becomes relevant.

Analysis of the recent research and publications sources. Studing problems of modernisation industrial-economic systems and different spheres of economic activity, development of theoretical foundations and practical measures for the management of innovation processes and development of high-tech dedicated to work of domestic and foreign scientists such as S. Valdaytsev, Z. Varnalii, V. Heyets, N. Ekvilain, U. Zabrodin, A. Mihailichenko, V. Onyshchenko, A. Saruhanov, P. Zavlin, S. Illiashenko, V. Soloviov, V. Stivenson, L. Fedulova, R. Jacobs and others [1 – 7].

Analyzing their researching and development dynamics of the national economy real sector, industry and construction from 80 years of the twentieth century to the present day, we can determine that the main problems that lead to their inefficiency and low competitiveness today are:

a) declining the level of innovation-technological and economic potential of the national economy and its non-availability of progress for the last 25-30 years;

b) inconsequence and weakness of the socio-economic state policy and innovation development for ensuring sustainable growth of the national economy.

The main reasons of economy not inefficiency are:

• inconsequence of previous economic reforms;

• insufficience of legislative, regulatory, scientific and methodological basis for the national innovation system and the high-tech manufacturing formation;

• national policies and development programs non-availability, comprehensible state policy of socio-economical and innovativ development of economy, their implementation; mechanism;

• finance limitations and inefficiencies of other state science innovation support for processes, projects and development programs;

• lack of economic incentives for business entities to implement technological modernization by implementing innovative projects and development programs;

• low population innovation culture level and weak educational outlet on its growth;

• absence of developed large innovation and high technologies market in Ukraine;

• insufficient support for innovation, development of export-intended and high-tech manufacturers that are implementing innovative products;

• absence of public-private partnership and concerted action in the sphere of innovations for manufacturing and other economic sectors modernization;

• chaotic and unbalanced high-tech manufactures investment programs realization in Ukraine and lack of support from state body;

• low level of information-communication innovation processes service, including national science and economy integration into world processes;

• insufficiency of modern design, innovation, product-marketing and operation management of professional teams from the realization management standard of complex industrial systems development;

• lack of innovation consulting and development in the field of innovation-technological economy development;

• high imbalance in industrial inefficient load regions and lack of state policy proportional stable areas development.

Considering the mentioned facts? It is possible to confitm that Ukrainian industry innovative activity level in the field of engineering is almost 10 times lower of international standards (in Ukraine – it's 6-8%, and in the advanced countries and companies of the world – 70-90% [1, 9, 10]); fatigue of domestic manufacturer fixed assets in industry and construction exceed 80-85% of their technical-technological base (TTB); productivity and energy consumption per unit of production (or maintenance of buildings and structures) in accordance is 5-10 times lower and 3-8 times higher than in the USA, EU, Japan or South Korea.

Bold unsolved before parts of the general problem. According to the calculations of the famous Ukrainian specialist in the investment industry and construction G.O. Bardysha, to recover and to develop domestic economy to the European average it should be involved total investment amounting to 3,0-3,5 trillion dollars USA, including in the field of capital construction – 1,3-1,6 trillion dollars USA [5].

Considering these and other results of the conducted existing researches analysis and publications, it can be determined that existing approaches and mechanisms (strategy and policy) about organization and management of innovative and high-tech development of machine-building and other manufacturers in industry and construction in other areas of the national economy real sector need radical updates for real transition to an innovative model of development and management international standards. Still unexplored areas of specific and effective mechanisms (organizational, functional and production systems, procedures and tools) development of economy, industry and construction real sector. The basic material of this article is dedicated to implement these objectives

Formulation of the researching problems. The aim of this work is to illustrate new theoretical approaches and practical recommendations for the establishment of guidelines and mechanisms for innovative high-tech and organizational development of economy, industry and construction real sector, considering the best international and domestic experience, the latest achievements of science and technology, engineering, strategic, multi-project and project management [1 – 11]. Specific tasks that are solved in this article:

1) determination of strategic development directions of the real economy sector , industry and construction;

2) formation of industrial and organizational industry and construction and manufacture development mechanisms in these areas of management, providing their innovation progress and competitiveness.

Basic material and results. The authors` conducted researches allow to propose such results and recommendations.

Describing this work first tasks implementation and considering the global standards of social-economic management and innovative development of Ukraine, which are given in previous development and authors publications [4, 9-11], it is needed to determine the following specific features of real sector of the national economy, industry and construction development functioning and directions:

1. *The main objectives and priorities of modern public policy* innovation and high-tech development of the national economy should be:

• increasing in production scientifically based product with high added value volume;

• solving the socio-economic issues and saturation of the domestic market;

• optimization of regional and territorial development through the rational distribution of high-tech production;

• solving environmental problems and ensure safety, occupational safety, environment and areas of human activity;

• economic security and development of foreign trade.

In Table 1 there is given a detail of the above objectives and priorities.

2. *The main directions of state regulation* further development of industrial and construction facilities Ukraine can be defined:

• to set priority sectors and industries;

•development of state targeted (priority and complex) software innovation and technological sectors development and productions;

• state regulation of structural changes;

• the development of competition and restriction of monopoly;

• tax regulation of industry and construction;

• monetary policy for the development of industry and construction;

• depreciation policy;

• investment policy;

• scientific, technical and innovation policy;

• state policy of reforming property relations;

• price and tax policy;

• state order and targeted development program;

• integration of domestic industry and construction industry into the world economy.

• promoting to economic security of Ukraine.

3. Today the priority areas of innovation and high-tech development of the real economy sector, industry and construction in which there are used targeted programs and investment projects with use of standards of strategic and project management are:

3.1. Electrostations technic-technological modernization creation and development of new and renewable energy sources, newest resource saving and energysaving technologies, comprehensive progress of capital construction sphere. This complex includes:

• health safety means and safety technics at mining enterprises; modern equipment for coal mining complex conditions; equipment for production and preparation for methane consumption (in coal mining areas);

• energy-efficient motors and drives for basic industries; different electrical equipment; sources and energy efficient lighting systems; functional and power electronics in the energy field;

• power stations and networks modernization, telemetry systems, automation and protection in nuclear power stations (NPP); electrical networks of issuance NPP capacities; gassteam installation and combustion technology of low-grade solid, liquid and gaseous fuels, traditional fuels, etc.;

• oil and gas drilling equipment, including oil and gas installations for extraction on the territory of the sea shelf;

• highly energy efficient compressor equipment;

• high-tech manufactures development of construction industry and building organizations.

Table 1 – The goals system of innovation and high-tech development, industry and construction, real economy sector of modern state policy

The growth of manufacture scientifically based products with high added value	1. Specific weight share of high-tech products and competitive products with high value of added part in composition of GDP specific weight share growth. 2. Increasing the share of consumer goods in the composition of GDP. 3. Increasing GDP of industry. 4. Effective use of own potential and resources. 5. Ensuring strategic balance in national economy 6. Exit to foreign market, establishing of subsidiaries branches and joint manufactures.
Ensuring of competitiveness	1. Improving the products quality and transition to international standards of quality management and competitiveness (TQM, ISO). 2. Nomenclature development and improvement and assortmen of scientifically based technology and high-tech products with high added value. 3. Decreasing of product cost product, resource and energy intensity of production and products. 4. Promotion of domestic industrial products on domestic and world markets. 5. Creation and improvement of fundamentally new technical and technological base.
Solving of socio-economic issues	1. Increasing revenue to the European level (of advanced countries). 2. Improving the education and professional skills of all staff. 3. Comprehensive automatization and cybernation manufacture, maintenance and management. 4. Creation of comfortable and safe working conditions. 5. Social protection of industrial personnel.
Optimization of regional (territorial) development and location of high-logic manufacture	1. Creation of high-tech scientific production systems. 2. Strategic planning and placing new production, development available considering requirements and objectives of the Ukraine development programme strategy (its productive forces) under conditions of minimizing the total cost of establishing and implementing manufacture. 3. Elimination the uneven of trade and territorial production development. 4. Promoting the implementation of regional projects within the state industrial policy.
Solving of environmental problems and ensure of environmental protection	1. The transition to international standards of sustainable development, environmental safety and environmental protection. 2. Creation of the most favorable conditions for people activities 3. Decrease negative impact of the industry on the environment. 4. Saving and strengthening of natural resource state potential. 5. Ensuring rational and complex use of natural sources of Ukraine.
Realization of implementation-innovation and target development programs	1. Transition to program-budgeting and planning mechanism for the implementation of strategic objectives of innovation and development. 2. Simultaneous and mutually integrated development of the national innovation system and industrial sector of Ukraine based on management project and program development. 3. Ensuring establishment of economic growth and innovation development national centers.
Economic security	1. Facilitating to innovation accelerated development of high-tech producing by the way of tax, customs and other preferences by the state investment policy. 2. Removing structural imbalances both in the industry and in the country. 3. Opposition of the Ukrainian raw materials transformation to industrialized countries. 4. Creating favourable investment climate and increasing innovation activity of industrial sector. 5. Prevention capital outflows. 6. Decrease the dependence of industry on import (energy and critical technologies), compliance limit exports. 7. Minimization of the shadow sector and industry criminalization.

3.2. Mechanical engineering and instrumental engineering, which should provide high-tech development of all manufacture industries; development of modern high-quality steel, manufactures of enterprising newest building technique. This group includes:

• production of newest rocket-space types and aviation technique, and electric vehicles; system tools of technological projection, manufacturing and logistic support of working processes of new generation technique; newest tools of diagnostic types, machinery, equipment and parts for high-tech systems of various purpose;

• production of newest high-tech types and economic building technique and equipment for the building industry enterprise;

• systems of telemetry, ranging in different environments optoelectronic systems of dual purpose;

• equipment and materials for welding and performance of related processes, durable and dynamically stable welded constructions; equipment, materials and newest technologies for corrosion protection;

• equipment and special technologies of newest steel producing types; recycling technology of secondary raw materials nonferrous metals;

• household and municipal electronic technics and its technological processes of manufacturing based on innovative modern types of televisions, refrigerators and other consumer goods.

3.3. Nanotechnologies, microelectronics, information technology, telecommunications and computer systems. This complex applies to:

• information systems of control and management of basic technologies objects and various economic industries objects; intellectual computer systems and high productivity means; recognition software systems and processes object; digital broadband media distribution systems;

• laser techniques and equipment, technological processes of their application;

• electronic database of communication systems, computer and telecommunications technologies; optical fiber systems; light signal and information outfit.

3.4. Chemical technologies, new materials, biotechnology progress developing includes:

• modern catalysis, development and use of new catalysts and new catalytic processes;

• development of genetic-engineering technology;

• immunological medicines and biomixture materials;

• modern construction materials and technologies for their production and use;

• ceramic and supersolid materials;

• modern materials of chemical production;

• newest types of semiconductors and monocrystalline material.

3.5. Machines and mechanism, other industrial production for high-tech development of agriculture and processing industry

3.6. Transport systems; building, reconstruction, technical upgrading and modernization. This comprehensive direction includes:

• innovative technologies, machines and mechanisms for roads, bridges and transportation systems building and reconstruction;

• rationalizing modern system using satellite and ground-based equipment rationalizing ;

• ports reconstruction;

• gas, oil and ammonia transportation modernization.

3.7. Human health and economic sphere care includs:

• diagnostic and therapeutic software and hardware complexes;

• modern medicines;

• equipment and technology for alternative energy sources;

• energy-efficient, resource-saving, modular, environmentally safe equipment for water treatment systems, water purification, heating and control means.

3.8. Information-communication systems and technologies developing.

Along with determination and guarantee of realization domestic industry development and building sector priority directions, state structural policy modern mechanisms should provide measures for:

• investment flows and other resources in priority areas stimulation;

• support innovative and high-tech industries development that promotes and implements scientific and technical progress, high technology production and competitive production of world-class quality;

• protection and financial support of sectors that are in state of stagnation, diversification and optimization, which require industrial machinery radical overhaul;

• reorganization of production in depressed areas;

• developing national strategic, tactical and other plans, programs, national, sectoral and regional projects for the solution of structural adjustment programs as well as all categories of staff training and re-training, including new working power creation;

• solving problems related to capital focus and priority in capital-intensive areas of structural adjustment.

Considering these and other features of the real economy, industry and construction should take place at the national and regional levels of governance (strategic plans, programs, laws, regulations and other instruments) and in specific investment projects and targeted programs of enterprise and other complex systems development.

To solve second objective results of this study, strategic goals of innovation and hi-logical and economic growth of the country for the implementation of the ongoing progress of industrial enterprises and construction companies were identified. It is needed at first to implement the restructuring of their industrial and economic, organizational and functional systems of innovation investing mechanism production advanced forms and operations, based on common approach to project planning, development and management of domestic producers and providing these complex challenges of continuous improvement :

1. Ensuring manufactures responds to real conditions of the external environment by the way of:

• formation or clarification manufacture mission;

• business-activity strategy determing and development as long-term goals realisation means;

• specific projects and programs development and local integrated goals implementation;

• specific projects and programs development and integrated local and objectives implementation;

• these realization projects and programs processes organization managemen.

2. The ways for innovation-investment activities providing by creation new and improvement existing products, technologies and production capacity for customer needs satisfaction are:

• consumer demand and needs dynamics study, various segments and market sectors competition and prospects;

• creating of new company products creating and existing company products improvement;

• design new and improving existing technology and production capacity, improving it's production and economic mechanism and organizational and management systems and business activities.

3. Development of high-tech production that provides:

• production facilities and equipment, high technologies uninterrupted introduction optimization and rationalization;

• their efficient use ensure;

• new and rationalization existing processes design in space and time;

• industrial systems cybernation and automation, flexible manufactures and new lines creation;
• total quality management and competitiveness of products and manufacture ensuring;
• rationalization and improving management efficiency of enterprise manufacture

4. The ways to develop the system of providing production activities:

• comprehensive implementation and effective use of modern information and computer systems planning, organization, control and regulation of material-mechanical production that provides energy needs;
• procurement system and inventory management optimization;
• energy efficiency and enterprises energy saving improvement;
• equipment, buildings, structures and communications repair improvement;
• tool, transport and warehouse company facilities rationalization.

Comprehensive and effective solution of these problems provides use of modern project management and program innovation-investment development of high-tech industry approach of which is related to the:

• creating of new production systems (industrial companies and manufactures) for the production of competitive products;
• renovation and development of high-tech manufacture in existing industrial complexes and production facilities.

Conducted researches considering the world experience analsis results and successful management (and nowadays state of Ukrainian enterprises) allowed to *determine such main tasks as for business-processes progress keys:*

1. To monitor and analyse environment, market and customer needs to indicate global trends, identify new innovation, determination of specific requirements and conditions for further development and business activities.

2. To form and/or improve systematic imagery about the mission and total innovation-investment strategy of company development in total, individual elements and activities sphere etc. It is necessary to consider the requirements, conditions and achievements about development in environment, to assess the current state and potential opportunities and improving systems and all speres of the company activity, to develop them to the best world standards.

3. To develop and effectively implement innovative programs and projects by the creation and realization of new products, modern services and other innovations. To manage strategic process according to certain general and innovative investment strategies of the company. The concept and plans for product innovation should be developed, full range of research, design and other works for development, implementation and realization of innovation (projects and programs) should be conducted and their economic and other efficiency should be ensured.

4. It is necessary to create and develop marketing, development and other effective systems and fields of activities for the promotion, marketing and firm in-life service product innovation. This is especially important for unique vehicles, machinery, equipment, buildings and other facilities that have long-lasting operation period (several years) and require constant of service repair, further improvement.

5. To implement innovative technology development, technical-technological base and entire production system of the enterprise, including their features, sphere of material-technical and other support, organization and management, etc.

6. At same time with the development of industrial and construction companies, industrial and technically complex engineering systems, it is also advisable to improve the structure and functions of firms oriented on service. They need to develop logistics and effective resource flows and services for themselves and for all their customers (clients) to develop and implement new advanced qualification standards of personnel and services, develop a list of systems and mechanisms for the effective implementation of these services, ensuring their good quality.

7. To improve infrastructure, functions and direct relationships with consumers, customers and other contact audiences ensure maximum innovation and investment projects and programs «turnkey» with focus on long-term mutually beneficial cooperation with Ukrainian foreign audience, and in the field of foreign activity.

8. To improve system and standards of management human resource enterprises to apply modern project and program oriented methods of personnel management which implements innovative-investment projects and development programs. Thus, for the improvement of staff it is recommended to implement integrated and world achievements, best systems and personnel management standards, such as:

• standards ISO 9000, defining the total system and sphere of management;

• American, Japanese, European and other standards of project management and corporated business (PMBoK: 2004, P2M, IPMA, PRINCE 1 і 2 [7,8]);

• modern strategic management, which is based on the balanced system of indicators (The Balanced Scorecard). This system was proposed in the mid-1990s, by US scientists R. Kaplan and D. Norton;

• innovative form of gradual improvement management of enterprise personnel in the process of its development, which is based on the model of maturity project management (Project Management Maturity Model – PMMM).

9. To implement integrated development and information resources management of systems and sphere of innovation company, its projects and programs

10. Ukrainian enterprises have toimprove effectiveness of consumption (spending) material (especially) energy resources constantly, to provide real (at times) to reduce energy consumption and material production and production, to provide modern management and activity development in this sphere.

11. It is necessary constantly to improve all the financial and economic activity of business entities, especially in the field of studing, involvement and efficiency and return on investment involved in innovative projects and development programs. This strategic direction is advisable to use modern tools and procedures for project financing, effective management of development and implementation business plans, budgets, and other financial plans, standards of performance.

12. In the period of its activity development in any project or program, each manufacture must ensure the protection and system management in environmental area. In this field, the manufacture is more expedient to interact with the state bodies, with regional and local , public (environmental) organizations, with society etc.

13. Development of all types, directions and spheres of relationships with the environment, innovative manufacture that carries out improvement, going to be competitive. It is needed to improve the management of external relationships, develop effective PR-programs for organization and its products positive image.

14. As it was mentioned, each innovative manufacture must manage its development, systemic and long-term process improvement and change.To provie this it is recommended:

• to create (improve) and constantly measure complex of the organization activities performance;

• to conduct internal and external quality evaluation, to ensure its uninterrupted improvement and management based on totaly system quality management (TQM), international and domestic quality standards etc;

• to perform comparative analysis of activities, achievements, state and potential of the manufacture, to evaluate the performance of pre-designed policies, programs and projects;

• to make changes and adjustments to their policy measures to promote sustainable development and improvement of enterprise competitiveness, etc.

In complex and effectively realized certain strategic goals and objectives it should be based on the creation of innovative-investment model, strategy and program of company development at the application of this strategic and unique process project management standards.

Analysis of successful companies and world enterprises in innovation management, business-projects and development programs confirms advanced experience that is possible to determine the following basic rules of corporate and project management to solve improving business processes of various Ukrainian enterprises problems:

1. The integration of innovative tasks as fundamental basis for improving the competitiveness of the company – innovator is a single innovative program. This means that:

a) all staff understand adopted innovative program (strategy) development of innovative enterprise and support it;

b) all fields of the enterprise activity for effectively interaction and agreement develop according with this concept;

c) innovative problem (projects and programs, etc.) focus is to satisfy the demand on defined market segments;

e) innovative potential is concentrated (focused) in limited innovation-investment area (field).

2. Creating and stimulatingof innovation-investment climate in the innovations company provide:

a) development sense of career and readiness to risk both in managers and in employees;

b) development of extradepartmental and interdisciplinary thinking of developers;

c) development of critical attitude to achieved results in innovation and innovation business.

d) stimulating of innovation and investment activity on the enterprise;

e) development and deepening of cooperation with other organizations – innovators and business partners

3. Using of extraordinary organization solutions, which means:

a) organization of innovation-investment measures and transformations (innovation) as permanent basis of evolution and manufacture business-processes;

b) use of project form of innovation activity management;

c) the development of flexible, innovative adaptive structures.

4. *Development and application management methods of innovation projects:*

a) fundamental training of innovation, innovative measures and transformations;

b) quality project planning;

c) objective evaluation and economic projects assessment;

d) strict control by projects implementation.

5. Preparing of manufacture and innovative products promotion:

a) implementation researching and technical training of innovative products series manufacture under long-term demand of estimates for promising market segments;

b) preparing defined market segments for innovation product;

c) construction the system for sale innovative product;

d) preparation of system for service consumers maintenance.

6. Providing high efficiency and economy-innovation and investment projects and development programs:

a) reducing the duration and decreasing the cost for innovations and innovative-investment business;

b) ensuring maximum possible world class quality and satisfy the demand for innovative products;

c) implement of ahead innovative measures and changes in competitions;

d) permanent identification of the highest achievements and focus their innovations on their rational and efficient use

Conclusions. The main result of the gradual introduction and use of proposed mechanisms and practical recommendations for innovation and high-tech development enterprises and organizations is that it should be supported their progress and competitiveness in global and neoliberal space and also ensure Ukraine welfare to international life standards. This conclusion confirms the European model of sustainable development, which involves the state leadership and society.

The main areas for further researching in the field of innovation and high-tech development of real economy sector, industry and construction should be the search of financial effectiveness mechanisms, administrative and intellectual support of the above processes.

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WORLD EXPERIENCE OF BUSINESS PROCESSES ORGANIZATION BUILD-INVESTMENT PROJECTS NEW FORMS AND MANAGEMENT IN THE FIELD OF UKRAINE COMPLEX OBJECTS DEVELOPMENT

The scientific principles and practical recommendations on the organization and management of business processes and construction investment projects in the real sector of the Ukrainian economy are developed. These issues are considered on an example of the development of objects and processes in the oil and gas complex. The basis of its improvement is proposed to apply new forms and best international experience to ensure the modern progress and competitiveness of complex industrial and business systems. It is determined that the practical recommendations given in the article can be successfully applied in industry and other sectors of the real economy sector of Ukraine. Their use should ensure the acceleration of modern progress and the growth of the competitiveness of the domestic economy.

***Keywords:** innovative and high-tech development, project management, engineering, development.*

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НОВІ ФОРМИ І СВІТОВИЙ ДОСВІД ОРГАНІЗАЦІЇ Й УПРАВЛІННЯ БІЗНЕС-ПРОЦЕСАМИ ТА БУДІВЕЛЬНО-ІНВЕСТИЦІЙНИМИ ПРОЕКТАМИ В СФЕРІ РОЗВИТКУ СКЛАДНИХ ОБ'ЄКТІВ УКРАЇНИ

Розглянуто наукові основи та практичні рекомендації щодо організації та управління бізнес-процесами і будівельно-інвестиційними проектами в реальному секторі економіки на прикладі розвитку об'єктів і процесів у нафтогазовому комплексі України за рахунок упровадження нових форм та світового досвіду забезпечення сучасного прогресу й конкурентоспроможності складних виробничо-господарських систем. Запропоновано основні процедури та інструменти реалізації таких достатньо нових для України механізмів забезпечення інноваційного та високотехнологічного розвитку зазначених систем, як стратегічний, мультипроектний і проектний менеджмент, інжиніринг та девелопмент.

***Ключові слова:** інноваційний та високотехнологічний розвиток, проектний менеджмент, інжиніринг, девелопмент.*

Introduction. Nowadays the Ukrainian economy real sector state, its industrial, construction, oil and gas and other economy branches require the restructuring of the modern innovation and investment mechanism of high-tech and continuous development, based on the advanced forms application and world experience in this process organization and management, the latest advances in science, technique and technology in order to ensure their competitiveness in Ukraine and in the world.

Technical re-equipment and development of the state economy in the direction of world standards requires 3.0 – 3.5 trillion USA dollars in the various forms of investment. 1.2…1.5 trillion USA dollars should be mastered in the field of capital construction [1], that is, in the field of new construction and expansion of enterprises, industrial objects, oil and gas spheres, construction and other sectors of the national economy. This list should also include all types of construction investment and innovation projects and programs related to the reconstruction, modernization, overhaul and technical re-equipment of practically all available production facilities, equipment, buildings, constructions, various types of transport communications (highways and their infrastructure). The listed and other aspects of the present require world experience qualitative study and involvement in Ukraine and construction new forms organization and management in market economy for the modernization and development of all objects and processes in the economy real sector.

Analysis of recent research and publications. A number of domestic and foreign specialists developments are devoted to the study of issues related to the organization and management of business processes and construction and investment projects in the economy real sector, in the sector of capital construction, industry and objects of the oil and gas complex. Over the past decade, unconditional leaders in the field of innovation in the creation and practical implementation of the latest forms of organization and management of projects latest forms in the leading sectors of the economy are the leading construction and investment, development and engineering companies American academics and representatives. They created and developed the world-renowned worldwide standards of USA PMBoK project management: 2004 [2] that have been successfully applied and developed for several decades. Based on their principles, different sets of recommendations and norms of international organizations for project management are developed and are currently operating: the European standards of professional knowledge and skills on project management IPMA, the relevant requirements and recommendations of the World Bank and the European Bank for Reconstruction and Development, etc [6].

In addition to the listed and other standards of project management, which became the valid standards of investment business projects and programs design, organization and management for the development of complex high-tech, production, organizational-economic and transport systems in the leading economy branches, it is needed to submit the research of such famous foreign and domestic scientists as P. Martin and K. Tate [3], D. Gerd and A. Tovb [4], G. Kerzner [5], G. Tsines, Y. Zabrodin, A. Sarukhanov, S. Bushuev, I. Babayev, V Yakovenko S. Dziuba and others. In addition to the listed and other standards of project management, which became the valid standards of design, organization and management of investment business projects and programs for the complex high-tech, production, organizational-economic and transport systems development in the leading branches of the economy, the research of such famous foreign and domestic scientists as P. Martin and K. Tate [3], D. Gerd and A. Tovb [4], G. Kerzner [5], G. Tsines, Y. Zabrodin, A. Sarukhanov, S. Bushev, I. Babayev, V Yakovenko S. Dziuba and others. Analyzing existing publications, it should be noted that they pay great attention to the general procedures of projects organizational and economic management in a stable business environment, which are inherent in the advanced countries of Europe and the world. At the same time, due to the manifestation of crisis phenomena and the domestic economy development weakness, lack of experience in project management in Ukraine construction, industry, oil and gas complex in the market economy conditions, it can be determined that the existing recommendations of international organizations and project management scientists [2 – 9] for domestic enterprises today are not enough and they need to be refined accordingly.

Description of general problem unsolved aspects. Considering the above factors and theoretical developments and practical recommendations lack for the business processes and projects organization and management in the field of capital construction and industry, as well as at the objects of the oil and gas complex in management modern conditions, there was a need to expand the list of applied researches on studying and engagement to the new forms domestic enterprises activity and world experience in the processes organization and management. According to the authors, solving this problem is an actual task, which has some scientific novelty and practical value.

The purpose of the paper. The main objective of the article is the scientific foundations and practical recommendations formation for the business processes and construction organization and management and investment projects in the economy real sector, such as the objects and processes development in Ukraine oil and gas complex due to the introduction of new forms and world experience in the modern progress organization and management in economic systems.

Basic material and results. A key condition for the competitiveness of Ukraine oil and gas sector is the creation of a modern, closed-loop cycle for high value-added products extraction, transportation, deep processing and production. The main objects of the oil and gas complex include:

– preparation of territory and routes for the oil and gas wells, storage facilities, pipeline systems and oil and gas extraction, storage, transportation, products processing and sale processes other objects arrangement;

– linear objects (engineering structures, equipment and communications) of main pipelines;

– facilities for the extraction, storage and gas preparation for transportation;

– main pipelines compressor stations;

– gas distribution stations;

– facilities for oil and gas extraction, collection, storage and preparation for transportation;

– stations for the oil and gas transfer;

– marine oil pipelines;

– oil refining objects;

– gas processing objects;

– natural gas liquefaction plants;

– other objects.

Almost all enterprises and organizations of the oil and gas complex which are the subjects of economic activity, in order to ensure their functioning and continuous development in the current conditions of Ukraine, new functions and corporate standards of industrial and commercial and any other activities formation, must introduce company main business processes different innovations and world experience that can be grouped into such groups:

1. Market analysis, needs and requests of consumers (and customers) for products and enterprise services (marketing research).

2. Formation of strategic vision (market philosophy) and strategy (strategic plans and programs) among enterprise owners, leaders and personnel.

3. Innovations development and introduction (the latest products, services, technologies, production systems, etc.), including «turn-key basis» production and implementation.

4. Products and services effective commercialization and sale ensuring.

5. Production systems and processes formation (organization and constant progress) and providing them with everything necessary in the necessary level of products and enterprises competitiveness formation conditions.

6. Development of production service and enterprises oriented on production and commercial service, marketing and advertising (including engineering, development and life-style or «brand» service).

7. Organization and management of contractual work (contracting), cooperation with suppliers, consumers, clients and other stakeholders.

8. Personnel management of the enterprise.

9. Management of information systems, flows and enterprise resources.

10. Management of investments, financial resources and enterprises expenses in projects and programs, etc..

11. Creation and development of work comfortable and safe conditions on the objects and in all systems of the enterprise (ecological safety and environmental protection activities, technical safety and labor protection management).

12. Management of external communications and the environment of the enterprise.

13. Enterprise development programming and designing (its products, technologies, production and other systems and business processes). Their «turn-key basis» implementation ensuring. Development and competitiveness growth continuation.

The transition to world standards of management and ensuring the continuous progress of enterprises in the oil and gas complex requires not only the implementation of the above measures and processes, but also the work on the development and application of new mechanisms, procedures and tools for their organization and the successful implementation of a «turn-key basis». These (new for Ukraine) mechanisms should first of all include strategic, project and multi-project management, engineering, reengineering, development, corporate and life services, etc. Consider them.

Considering the global experience of integrated and rational use of procedures and tools for strategic, multi-project and project management, it can be noted that in addition to the world-renowned ISO 9000 standards in the management of projects and programs for the development of complex and unique systems (enterprises, facilities of the oil and gas complex, etc.), advanced countries and companies today also apply such complexes of standards:

– 3rd version (edition) of the PMBoK Guidebook: 2004, which today is the current American national (and world) standard of the project and program management system [2];

–developed on its basis in recent years and widely applied in practice, other modifications to this methodology of project management, such as:

– Japanese system of knowledge and skills in project management P2M;

– a complex of international requirements for the competence of project management specialists and the relevant European standards for professional activities in this area – IMPA Competence Baseline (ICB);

– project management methodology PRINCE 1 and PRINCE 2, applicable in the UK;

– other standards of project and multi-project management used in the management of investment projects and development programs.

Summarizing the world experience in applying new forms of organization and development of enterprises, corporate and project management, this process can offer a new comprehensive solution to existing problems in the state, forming the general contours of the standards system for managing investment projects and programs for the complex industrial and economic systems development in the oil and gas sector of the Ukrainian economy:

1. Ukraine ISO 9000 standards, legislation and regulatory system determining the general management system in the state, branch, corporate and program-design environment.

2. Project Management Standards, as outlined in PMBoK: 2004, P2M, ICB (IMPA), PRINCE 1 and PRINCE 2, which create a regulatory and methodological framework for building a project management system and programs. They are the focal point for the entire set of management standards for any projects (including construction, reconstruction and technical re-equipment of oil and gas complexes, enterprise development programs).

3. On the basis of the above listed standards, specific corporate standards are developed for today powerful (advanced) development and engineering companies that are able to implement «turn-key basis» for any projects and programs, including the attraction of necessary investments. Worldwide examples of such companies are Fluor Daniel Corporation, Technip-Coflexip, Bechtel, Parsons, Man, Petrofac, Foster Wheeler Inc., AMEC, ABB Group, World Super Engineering, etc. They implement projects all over the world, and therefore their corporate standards (among other things) regulate multi-project and project management in subdivisions (strategic business units – SBU and so-called «professional project management offices – PMO offices», participating in international and national unique projects, applying modern strategic multi-project and project management, engineering and development.

4. At the last stage, management standards are developed directly for specific projects and programs in the oil and gas sector. Their approximate list is shown in the Table 1.

One of the main tasks in the field of business projects organization and management and oil and gas complex development in Ukraine is to ensure its competitiveness in the conditions of the domestic economy globalization and neo-liberalization. Today, the concept of competitiveness should be considered as the property of economic entities and their products to the maximum society and individual consumers needs in comparison with similar companies and their products (including services), which are available (or offered) in the domestic and world markets. The key parameters that determine competitiveness today are the products, works and services quality, their price (cost), consumption and exploitation (service) costs, first of all, objects, structures and engineering communications in the oil and gas sector, including quality, price and other parameters of their «brand» and «life» service, modernization and further (continuous) development.

Today, engineering is recognized (in the leading countries and companies of the world) as a highly effective function of modern business and innovation and investment activity, which essence is to provide consumers with the latest products, high technologies and production systems, research, design, construction, calculation, analytical, production organizational structure advanced services, including the feasibility study and business plans, different types of project, work and other investment documents, professional supervision and project management of complex projects and programs. Today, the leading companies in the world operating in the oil and gas business use the following types of engineering: design, technological, cost, financial, industrial, integrated, which combines all the above types of engineering in various combinations. In leading countries and companies in the world, integrated engineering is called the Build-Own-Operate-Transfer system. This system assumes that the project's main executor (project team, «PMO office», customer, etc.) not only designs and implements a «turn-key basis» project, but also exploits it (for example, various facilities and engineering structures of the oil and gas complex) over a significant period of time (20…25 years), after which it can transfer it to the owner (for example, the state, or the operating organization).

Table 1 – Recommended list of project management standards at the oil-gas complex

Standard No.	Procedures which the standard relates to
1	Organizational structure of the enterprise
2	The main business processes of the enterprise
3	Corporate governance standards
4	Strategy and strategic plans for enterprise development
5	Organization of works on the pre-investment phase of the project
6	Preparation of proposals for the tender
7	Pre-project research, development, approval, examination and approval of feasibility study, business plans and other project documentation
8	The main aspects of project management (including the formation of its statute and the project management plan or program)
9	Organization of work on a project (program)
10	Development of project documentation
11	Use of project documentation
12	Identification and control on project documentation (program)
13	Project Control (Development Program)
14	Project quality plan
15	Memo (instruction) to project management (programs)
16	Shifts management
17	Discussion on the progress of the project implementation (programs)
18	Project planning (program-design activities)
19	Development and implementation of works breakdown structure and the organizational structure of the project (program) in the dynamics of its development
20	Project cost control (program)
21	Development of cost allocation structure
22	Project Reports (Program Activities)
23	Management on information, documentation and document circulation in the project (program)
24	Equipment and mechanisms
25	Project staff and personnel policy
26	Project Risk Management
27	Safety and creation of comfortable working conditions (labor protection, safety engineering, ecology, technogenic and other kinds of safety)
28	Procurement by project (program)
29	Subcontract agreements
30	Work instructions and regulatory framework
31	Materials control on the site
32	Check and test on the site. Adjustment works
33	Maintaining executive documentation
34	Preliminary development, testing and launching of objects (project, program)
35	Final report on the project (program)
36	Accumulation and use of project data (program)

The undeniable advantages of engineering should include: increasing the efficiency of any innovation and investment projects and programs for the development of enterprises and facilities of the oil and gas complex; reduction of design and program activities terms and cost with simultaneous growth of their quality and modernity; attractiveness for the owner, customer and investors; creation of prerequisites and real opportunities for the transition to world standards of management, corporate and project management; reducing investment and other risks, increasing competitiveness.

In addition, engineering, construction, development and other companies that have participated in or may be involved in projects and programs in the oil and gas sector, there are real opportunities and economic interest to provide a complex of «branded» and «life-long» services, or develop the objects of the oil and gas complex both in Ukraine and in the world.

Development is one of the most advanced and modern concepts of programmatic and design (system) management, when within the framework of a unique innovation-investment, construction, organizational or other project or program of enterprises and objects development of the oil and gas sector, not only an object is created of any complexity and uniqueness (enterprise, building or construction, industrial and other equipment, communications having a long-lasting decade-operation period), but also permanent and long get integrated service, modernization and development of this facility and professional management. It is a function of a legal owner who will carry out commercial or other exploitation of the whole object (equipment), and the potential co-owner, its developer and manufacturer. Such company development of the two mentioned entities always is considered in continuous facility improvement, increases technical and technological capacity, competitiveness, modernization of the brand service system, as well as in the mutually beneficial production and facility commercial exploitation in the prospect long-term. Thus, development is an implementation mutually beneficial form of various business projects and programs for innovation and investment enterprises – developers and producers of a unique product (including buildings, structures, communications, involved in the production of the economy oil and gas sector) and for the goods owners and buyers.

The engineer and developer-oriented advanced companies, construction companies and industrial enterprises world experience shows that the application in their business projects and development programs of the above-mentioned innovations from strategic and multi-project and project management, engineering and development allows these organizations to get:

– total duration of projects and programs reduction, by 12…20%, including their active-investment phase – 15…25%;
– work complexity reduction by 12…25%;
– operating costs reduction by 20-25%;
– the project total cost reduction by 10…15% or more [5].

At the same time, the quality and competitiveness of both the project (or program) and its developers and owners are grown up.

Considering the economy real enterprises and objects state in Ukraine, and especially in its oil and gas complex, as well as the need for their engineering, re-engineering (revolutionary, bifurcation renewal) and continuous development to world standards of management and business activities) at relevant enterprises it is necessary to accelerate the engineering and development subsystems and functions implementation in their innovation-investment models of activities continuous development and improvement, which in the end eventually lead to increased efficiency and competitiveness of domestic business entities and their products, reduce and protect against the negative impact of the crisis and provide integration into the global and European economy.

Conclusions. Summing up, it can be determined that the introduction of new forms and world experience in the organization and management of business processes and construction-investment projects for solving the problems of industrial objects and oil and gas complex capital construction in Ukraine, including the practical recommendations of the article authors, all considered promotes the progress of our country real economy sector, increases the population welfare, and ensures their dynamic transition to world standards of management, life and progress.

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WORKS EXECUTION ORGANIZATION AT BUILDING RECONSTRUCTION AND RENOVATION WITH USAGE OF SLABS LIFTING METHOD

The article considers works execution organization at reconstruction and renovation of residential and public buildings for embedded systems with usage of slabs lifting method at restrained urban conditions. The paper suggests usage features of the slabs lifting method at building reconstruction conditions. The technology of reconstruction with the usage of slabs lifting method allows to refuse practically the use of lifting cranes and to reduce significantly (up to 50%) the required building area compared with the lifting crane construction methods of embedded structures. It allows to perform the reconstruction without restriction of traffic on adjoining streets and to perform construction in restrained urban conditions.

Keywords: *organization, reconstruction, renovated buildings, embedded systems, floor slab panels, slabs lifting method.*

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ОРГАНІЗАЦІЯ ВИКОНАННЯ РОБІТ ПРИ РЕКОНСТРУКЦІЇ ТА ВІДНОВЛЕННІ БУДІВЕЛЬ ЗА ДОПОМОГОЮ МЕТОДУ ПІДЙОМУ ПЕРЕКРИТТІВ

Розглянуто організацію виконання робіт при реконструкції та відновленні житлових і громадських будівель шляхом зведення вбудованих систем методом підйому перекриттів в умовах щільної міської забудови. Наведено особливості використання методу підйому перекриттів в умовах реконструкції будівель. Виявлено, що технологія реконструкції з використанням методу підйому перекриттів дозволяє практично відмовитися від використання підйомних кранів та значно (до 50%) зменшити необхідну площу будівельного майданчика порівняно з крановими способами зведення вбудованих конструкцій. Це дозволяє виконувати реконструкцію без обмеження руху транспорту по прилеглих вулицях і здійснювати будівництво в умовах щільної міської забудови.

Ключові слова: *організація, реконструкція, відновлення будівлі, вбудовані системи, плити перекриття, метод підйому перекриттів.*

Introduction. Reconstruction of old urban development residential buildings, having, as a rule, historical significance and creating an architectural character of the city central part, is undoubtedly actual. The topicality is conditioned by the necessity of preserving the architectural integrity of historical development and, at the same time, the need for adaptation of planning concepts and buildings constructive concepts to the new operating conditions. The main reasons for the need of reconstruction are the discrepancy of existing planning concepts with modern conditions and the usage of wooden structures as ceiling structures. Due to long operating terms, wooden structures, as a rule, are generally at insufficient condition and require replacement or cannot withstand to the new increased operational loads. One of the such buildings reconstruction methods is the construction of embedded systems with the preservation of existing enclosure structures (Figure 1). To the features of works execution at the reconstruction of buildings at restrained urban conditions, especially in the central, historical parts of the cities it should be considered considerably limited size of the building area.

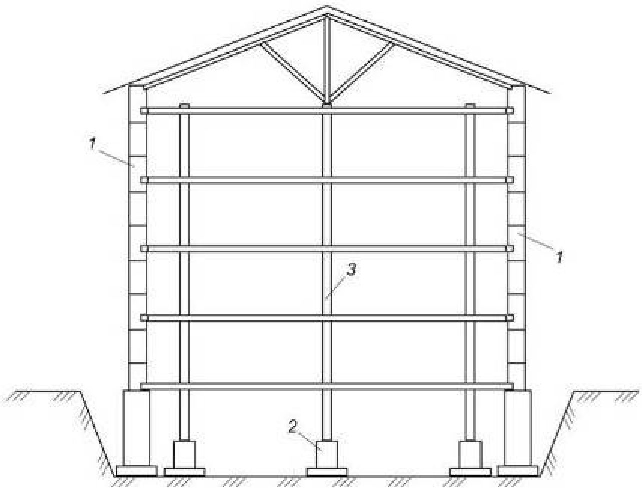

Figure 1 – Building reconstruction with the usage of enclosure structures:
1 – existing enclosure structures; 2 – foundation of the embedded system;
3 – embedded structures

This requirement essentially affects the possibilities of construction equipment and lifting cranes use. In some cases, the usage of lifting cranes requires the restriction or cessation of vehicles for a long time, and sometimes, the usage of lifting cranes is impossible at all. The foregoing leads to refuse of the usage prefabricated reinforced concrete structures and to use monolithic reinforced concrete structures as structures of the built-in frame. However, the usage of monolithic structures still requires the use of lifting cranes for the reinforcement cages and formwork supply. One of the methods allowing us to abandon the lifting cranes usage during the erection of built-in structures almost completely is the slabs lifting method [1].

Review of the latest research sources and publications. Recently, due to the actuality of the need for residential and public buildings of the old urban development reconstruction, a large number of works, was devoted to this question by both domestic [2 – 6] and foreign authors [7 – 15]. In works [2 – 6] the peculiarities of the foregoing buildings reconstruction, the problems of construction equipment use in the conditions of the limited size building area are considered, and the usage of prefabricated and monolithic reinforced concrete as a material of embedded structures is compared. It is concluded that the reconstruction of the old urban development buildings by replacing the ceiling structures and construction of built-in structures allows to preserve the architectural character of the cities historical part and at the same time to adapt buildings to the new conditions of exploitation. The works of foreign authors are often devoted to the issues of the concrete objects reconstruction [7 – 9], some parts of the works are devoted to the historic building structures strengthening [10 – 12]. Works [13 – 15] discuss the general issues of historical buildings reconstruction.

Definition of unsolved aspects of the problem. Despite the large number of works on the topic, until this time, the method of foregoing building reconstruction has not been proposed. There was no method that would allow to abandon usage of lifting cranes completely, to facilitate work organization under conditions of the building area limited size significantly and to refuse vehicles movement. The possibility of lifting slabs method usage for the reconstruction of buildings was not considered.

Problem statement. To analyze the peculiarities of lifting slabs method usage for the reconstruction of residential and public buildings by constructing embedded systems. To identify the benefits of these buildings reconstruction method use in comparison with other methods.

Basic material and results. As it has been shown [1], the construction of multistory frame buildings with a monolithic or prefabricated monolithic reinforced concrete frame by the lifting slabs method is to create the entire complex of floor slab panels on the ground surface or on a floor slab panel above the underground part. After floor slab panels installation and attainment it is required reinforced concrete strengthening, the floor slab panels lifting up to the design position on previously erected columns using lifts.

This method has the following sequence of work:

1) erection of building underground part by the traditional method, that anticipates the erection of column foundations, installation of the underground part columns, installation of building underground part envelope, floor slab panels, etc. (Figure 2);

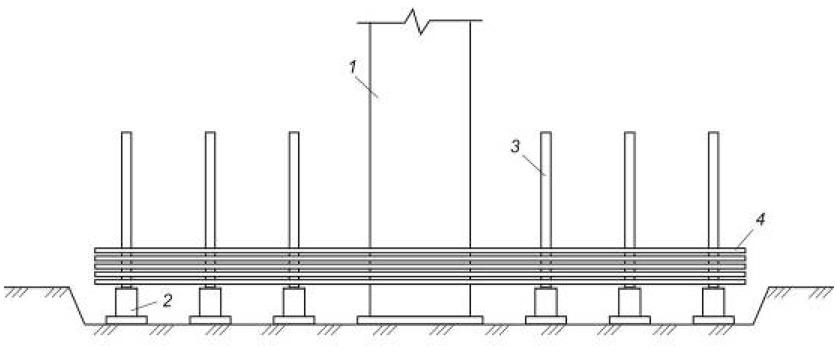

Figure 2 – Erection of the building with the slabs lifting method usage:
1 – stiffening core; 2 – foundations; 3 – columns; 4 – complex of the floor slab panels

2) the arrangement of the stiffening core, that ensures the durability of the building in the longitudinal and crosswise directions. As a rule, a stair enclosure with elevator shaft is placed inside the stiffening core;

3) installation, or placement of the ground floor monolithic columns;

4) installation of the floor slab panels complex round the building for the entire building area or section;

5) after the required reinforced concrete strength attainment of the floor slab panels, the lifting of slabs in an intermediate position with the usage of lifts fixed on the columns is accomplished;

6) installation or placement of the next level columns, moving the elevators and floor slab panels lifting;

7) after the lifting of all floor slab panels into the design position, the elevators are removed, next stage is the roof, enclosing structures arrangement, after that general construction works and finishing works are performed.

The advantages of such a method of building construction should include the fact that it can significantly reduce the usage of lifting cranes, and sometimes completely abandon their usage, significantly reduce the building area size; and in addition, due to lack of bearing walls inside the building; this method does not impose restrictions on the choice of design decisions.

This method was supposed to be used primarily for new construction, the usage of this method during the reconstruction was not considered.

From the analysis of this method advantages, it follows that its use during the reconstruction of buildings through the construction of embedded systems would be allowed for reconstruction without going beyond the constraints of restrained urban development conditions imposed on the construction. The method of floor slab panels lifting due to the refusal of cranes usage would allow reducing the building area size to almost the area size of the reconstructed building; refuse to restrict traffic and pedestrians and the operation of nearby objects.

The structure of the building reconstruction works by the proposed method will be as follows:

1) disassembling of the internal structures of the building is carried out. At the same time, disassembly should be conducted with strict sequence observance of the structures disassembling adopted in the technological plans. If necessary, the reinforcement of the enclosing structures is done;

2) the soil excavation under the foundations of the embedded system, the installation of monolithic foundations under the embedded structures are arranged. If necessary, there can be concrete mixture preparation at the object and feeding it with a light-weight concrete pump, located in the dimensions of the building. After the attainment required concrete strength, the waterproofing of the foundations and soil backing with compaction are carried out. If necessary, a reinforced concrete floor of the underground building part is arranged;

3) erection of first level monolithic columns is done. Formwork and reinforcement frames are installed from scaffold; the delivery of concrete mixture is carried out by a light-weight concrete pump;

4) installation of monolithic floor slab panel above the underground part of the building is carried out. Concrete is delivered centrally to the formwork by a concrete pump, located outside the building;

5) after the attainment required concrete strength of floor slab panel above the underground part, on its surface one after another the entire complex of floor slab panels (Figure 3) is arranged. Panels are separated by a separating layer;

6) after the attainment required concrete strength, the structures of the scaffold, formwork, prefabricated reinforced frameworks and a light-weight concrete pump are loaded onto the last plate for concreting the next level of columns;

7) elevators for lifting floor slab panels are installed on the headings of the first level columns [1]. Panels are lifted to an intermediate position and fixed (Figure 4);

8) concreting of the second level columns is carried out;

9) the lifting of the floor slab panels is continued after the attainment required concrete strength of the columns. The last two paragraphs are repeated until the concreting of the last level columns end and the lifting of all floor slab panels in the design position. After that, the joints between columns and slabs, between plates and enclosing structures are packed; the light-weight concrete pump is removed from the surface of the slab by a lifting crane. Installation of the building roofing, internal general construction works, and finishing works are operated by traditional methods.

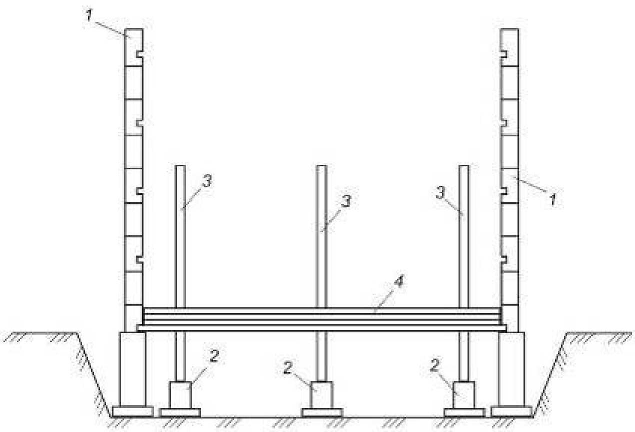

Figure 3 – Installation of embedded frame with usage a lifting slab panels method (stage of floor slab panels concreting):
1 – existing walls; 2 – foundations under embedded framework; 3 – columns;
4 – floor slab panels complex

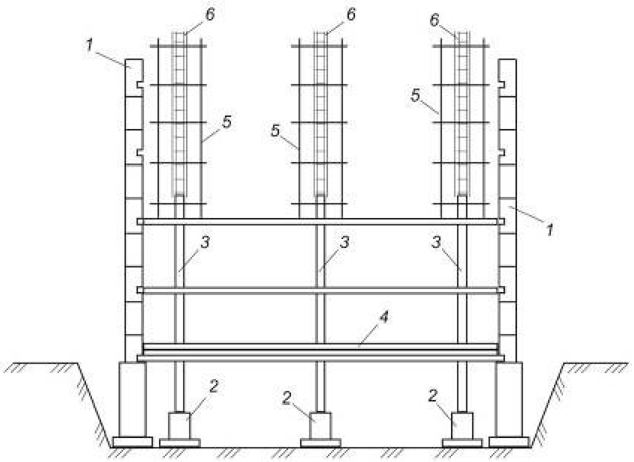

Figure 4 – Installation of embedded frame with usage a lifting slab panels method (stage of second level columns concreting):
1 – existing walls; 2 – foundations under embedded framework; 3 – columns;
4 – floor slab panels complex; 5 – scaffold; 6 – column formwork

As it has been shown in the given works, the proposed technology of buildings reconstruction allows to refuse the lifting cranes use during the works execution practically. It allows to increase the building general plan coefficient of compactness to 60 – 80%. With the usage of the traditional construction methods, this ratio is about 10 – 40%. The proposed method obviates the floor slab panel formwork installation and supports structures under it, as the formwork is pre-concreted slabs. This method allows to reduce the work complexity on the floor slabs installation significantly.

However, the usage of the lifting slabs method at the construction of embedded systems involves the manual assembly of column formwork, installation of reinforcing cages into the formwork, assembly and disassembly of scaffold, installation and removal of lifts.

The most effective method of construction organization is the streaming construction method and this method usage allows for definite significant advantages such as regularity and low intensity of resource consumption, reduction of construction time [7]. However, in the consideration case, the usage of this construction method is complicated by certain conditions. One of these conditions is a relatively small building area size that complicates or makes it impossible to separate the building to the required number of work zones. As a rule, in most cases, such types of buildings could be divided up to a maximum of two work zones that is not enough for the streaming organization of the monolithic structures construction. However, such case does not mean that it is impossible to effectively organize the process of installing embedded systems by usage of lifting slabs method. At the same time, the greatest attention should be paid to the organization of work on the installation of floor slabs, since their share in the total construction complexity of embedded systems is 60 – 80%. Figure 5 shows an work indicative schedule on the floor slabs package installation.

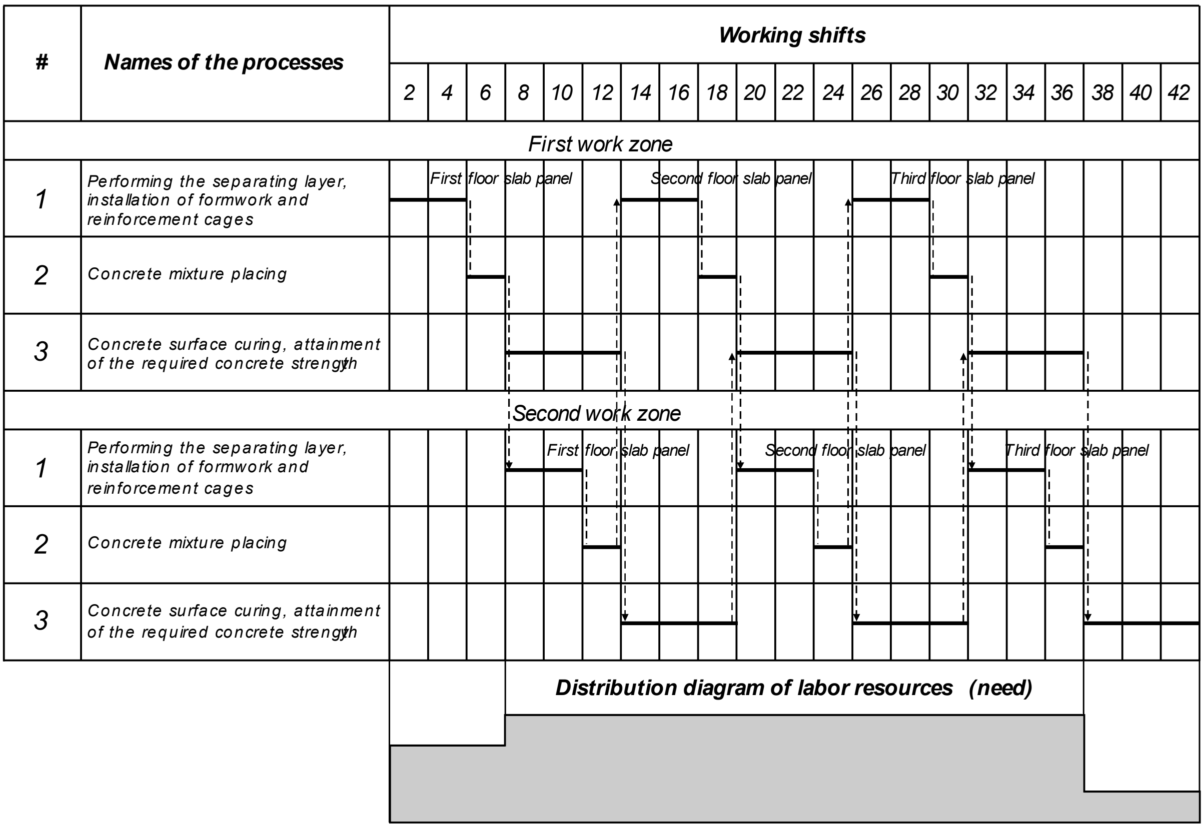

Figure 5 – Work schedule of floor slabs package installation by dividing the building into two work zones: arrowed lines show the workers transfer

As it has been seen from the chart shown in Figure 5, even when dividing the building into two working zones, the continuous work of the building team is organized. Due to this condition, the number of workers, duration of the distribution layer between the slabs installation work, the installation of reinforced frames and the concrete mixture placing should be the same with the duration of the concrete surface curing and attainment of the required concrete strength for the work start on the next slab. One of the conditions for such work organization is the use of integrated workers brigade combining the installation of reinforced frameworks and the concrete mixture placing. At the same time, as it is seen from the distribution diagram shown in Figure 5, the irregularity coefficient of labor resources usage is close to one.

Conclusions. The proposed buildings reconstruction technology can be used (after conducting an economic comparison with other possible methods of reconstruction in these conditions) for the reconstruction of residential and public buildings by installing embedded systems of monolithic reinforced concrete frame type.

The technology of reconstruction with the usage of slabs lifting method allows to refuse lifting cranes use and to reduce significantly (up to 50%) the required building area compared with the lifting crane construction methods of embedded structures significantly. It allows to perform the reconstruction without restriction of traffic on adjoining streets and to perform construction in restrained urban conditions.

One of the criteria for such technology use is the presence of buildings stiffening core and stair enclosure walls cannot be disassembled.

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INDICATORS OF FLOOR SLABS REINFORCEMENT TECHNICAL AND ECONOMIC INVESTIGATION BY DIFFERENT TECHNOLOGIES

The paper presents the results of technical and economic indicators study of reinforcing monolithic reinforced concrete slab various methods, namely: the supply of metal beams with the installation of additional supports, external reinforcement of stretched zones using MAPEI technology, adhesion of metal plates and carbon fiber to the developed technology. There is established that the highest indicators of the materials cost, labor intensity and wages for the execution of works relate to the option of reinforcing the floor slab by supplying metal structures, and the lowest indicators have options for reinforcing the floor slabs using MAPEI technology and bonding carbon fiber under the developed technology. The cost of materials for reinforcing the plate in the developed technology with the adhesion of steel plates is the lowest, but complexity, wages and the duration of the work on this technology are much higher than other studied technologies of external reinforcement.

Key words: *reinforcement of floor slab, external reinforcement, carbon fiber, technical and economic indicators.*

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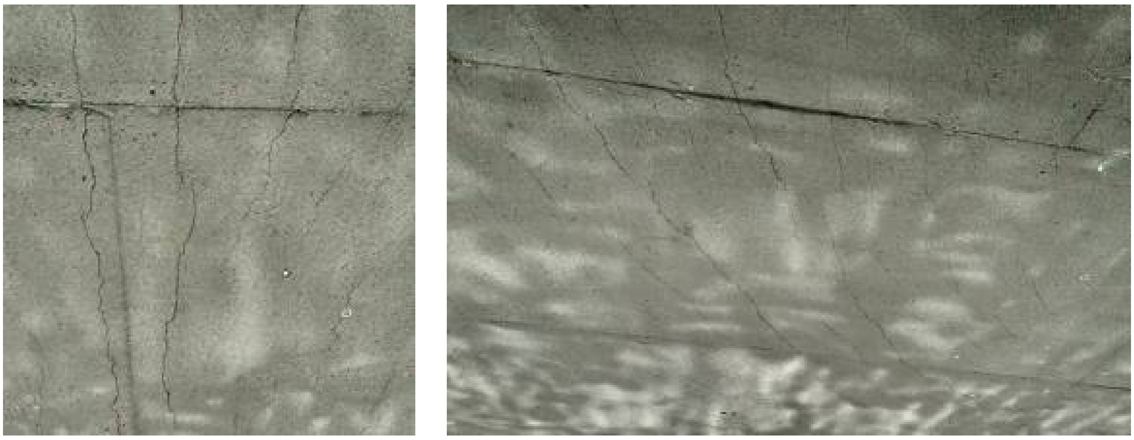
ДОСЛІДЖЕННЯ ТЕХНІКО-ЕКОНОМІЧНИХ ПОКАЗНИКІВ ПІДСИЛЕННЯ ПЛИТ ПЕРЕКРИТТЯ ЗА РІЗНИМИ ТЕХНОЛОГІЯМИ

Наведено результати дослідження техніко-економічних показників різних методів підсилення монолітної залізобетонної плити перекриття, а саме: підведенням металевих балок з установленням додаткових опор, зовнішнім армуванням розтягнутих зон за технологією «МАПЕІ», приклеюванням металевих пластин і вуглецевого волокна за розробленою технологією. Установлено, що найвищі показники вартості матеріалів, трудомісткості та заробітної плати на виконання робіт має варіант підсилення плити перекриття підведенням металевих конструкцій, а найнижчі – варіанти підсилення плити перекриття за технологією «МАПЕІ» й приклеюванням вуглецевого волокна за розробленою технологією. З'ясовано, що вартість матеріалів на підсилення плити в розробленій технології з приклеюванням сталевих пластин найнижча, але трудомісткість, заробітна плата та тривалість виконання робіт за такою технологією значно вищі за інші досліджувані технології підсилення зовнішнім армуванням.

Ключові слова: *підсилення плити перекриття, зовнішнє армування, вуглецеве волокно, техніко-економічні показники.*

Introduction. During the inspection of a building in Kyiv, a number of damages and defects in inter-floor overlapping structures were detected. The building is 5-storey, frame-monolithic. For floor slabs reinforced concrete was used. On their surface numerous damages were found in the form of cracks, located in stretched areas (Fig. 1). It was found that damage was caused by the influence of constructive, technological and operational factors, in particular: insufficient reinforcement of stretched zones; overload of constructions; discrepancy of the protective layer thickness with the design decision and regulatory requirements; strength of concrete is lower than specified in the project; the thickness of the floor slab, in some places, is less than the design; probable early unpacking.



a) b)

Figure 1 − Longitudinal (a) and bending (b) cracks on the surface of the floor slab

As a result of the visual-instrumental inspection and checking calculations of the overlap pings it was established that the technical condition of the structures does not provide sufficient load bearing capacity for their normal and safe operation according to their intended purpose. Therefore, it is recommended to perform their enhancement.

Analysis of recent research and publications. The analysis of normative and technical literature has established that in practice, in order to reinforce the slabs, the following structural and technological solutions are used: increasing the cross-section due to the build-up; change of the static scheme of work due to the installation of protrusions, slippers, racks, etc. [1 − 3]. However, in this case, the installation of additional reinforcement structures will reduce the inter-floor space, or change the structural and planning decisions of the interior space of the building. In addition, the load on the supports and foundations is significantly increased, or even the need arises for the installation of additional foundations for new supports.

Selection of previously unsettled items of the general problem. Considering the above mentioned, one of the excellent methods of reinforcing structures is external reinforcement (gluing using special adhesives on the surface of high-strength canvases, plates or strips (lamellae) structures. Materials and technologies of foreign production are now used in domestic building practice [4 − 6]. For the purpose of structures enhancement by external reinforcement, metal and composite materials based on carbon fibers, fiberglass and plastics can be used [7 − 10]. On the basis of SE «Research institute of building production» (Kyiv) a number of experimental studies were carried out. Based on the results of these studies constructive and technological solutions for amplification of beam structures with external reinforcement using domestic materials were developed [11 − 13]. However, before providing recommendations for using specific amplification technology in construction, its techno-economic feasibility should be established.

Thus, the **aim** of this publication is the study of technical and economic performance (TEP) of various technologies for reinforcement of overlap slabs.

Main material and results. In order to assess the economic feasibility of a certain method of slabs enhancement, it was decided to compare the following methods of amplification: supply of metal beams with the installation of additional supports, external amplification under the «MAPEI» technology and external reinforcement under the previously researched and proposed technologies. Provided research and comparison of technical and economic indicators for overlapping with area of 250 m², where outer loop is based on the piers and columns support it in the middle (Fig. 2, 3).

The main technical and economic performance indicators (TEP) selected for comparison: cost of materials for amplification, complexity, wages and performance duration.

By the *first method*, the reinforcement of the floor slab is provided by the supply of metal beams with the installation of additional supports (columns) (Fig. 2). Since, when performing amplification by the indicated method, the height of the room is significantly reduced, but in this case it is not acceptable, therefore this method is considered only for the purpose of the TEP comparison.

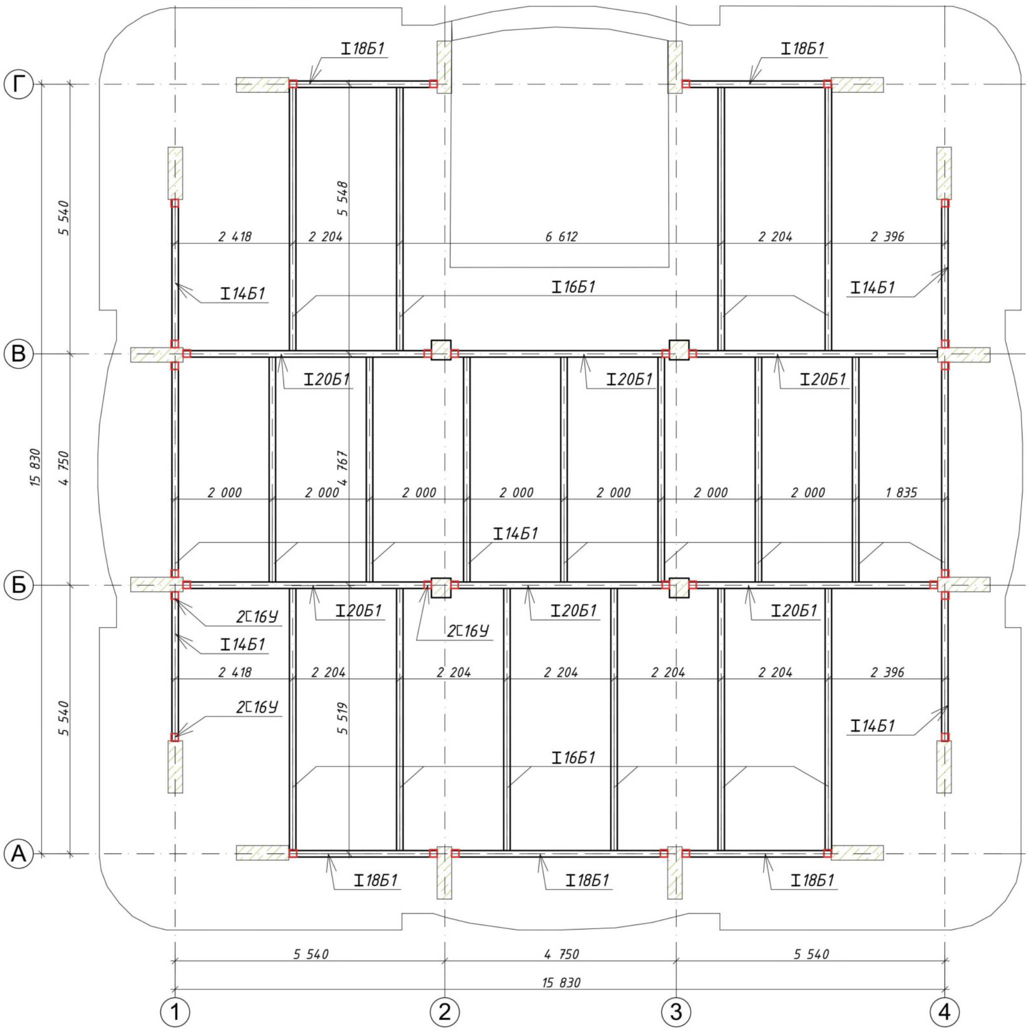


Figure 2 − Scheme of reinforcing elements of the floor slab according to the method of supplying steel beams with the installation of additional columns

The project provides for the installation of rigidly fixed metal columns (two 16U wires welded in a box) to existing reinforced concrete vertical structures in the areas shown in Fig. 2. On the columns longitudinal double T-beams 18B1 and 20B1 on the lower shelves are laid and welded; through their inserts (if necessary) transverse double T-beams 14B1 and 16B1 are laid and welded.

The following methods provide external reinforcement of slabs made of reinforced concrete, such as bonding metal plates or carbon fiber on the stretched zones. It allows increasing bearing capacity of the plate and reducing deflection. The areas for adhesion of strips for external reinforcement are shown in the schematic of the reinforcement elements (Fig. 3).

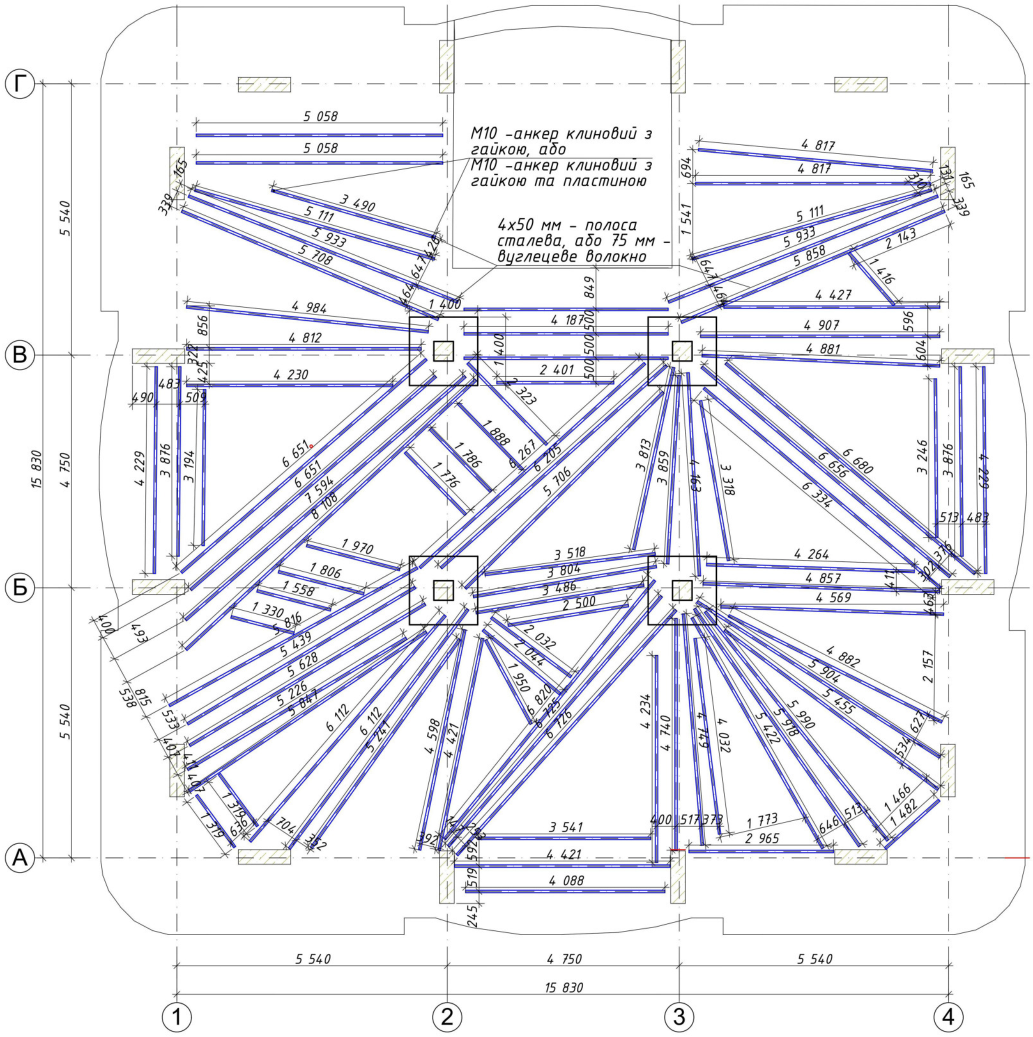


Figure 3 − The layout of the floor slab reinforcement elements with the method of external amplification (MAPEI technology, metal plates, or carbon fiber under developed technology)

By the *second method*, the reinforcement of the floor slab is performed according to the technology of the «MAPEI» company. Prior to the beginning of the reinforcement work, future carbon fiber bonding sites should be cleaned from the «glossy» surface using a sandblasting device, or a grinding machine with a special disk to clear concrete surfaces. This technology provides for priming the concrete surface to strengthen the foundations with composition MapeWrap Primer 1, bonding unidirectional carbon fiber MapeWrap C UNI-AX with adhesive MapeWrap 21. The ends of the carbon fiber strips are mechanically fixed with metal plates and bolts M10.

In the *third method* is scheduled to perform enhancement of the plate by sticking metal plates to its stretched areas. Firstly, the cleaning of the places for gluing plates is carried out according to the technology described above. Subsequently, the place of gluing the plates and 10 mm on each side outside each plate is impregnated with the composite grounding foundation «Consolid 1» manufactured by LLC «COMPOSIT". At least 24 hours later bonding of metal plates (4 x 50 mm) is executed on an epoxy based adhesive «EDMOK» produced by LLC «COMPOSIT». At the ends the plates are fixed with wedge anchors M10. Plates are pre-wiped to shine and degreased.

By the *fourth method*, the reinforcement is performed by gluing of of unidirectional carbon fiber stripes on the stretched zones. Before the beginning of the gluing work, place intended for bonding the fiber is prepared, namely, the surface is cleaned and «Consolid 1» foundation is applied as described in the previous method. After 24 hours «EDMOK» adhesive is applied on the prepared surface and strips of carbon fiber are «drown» in it with a spatula. In 5-10 minutes, another layer of «EDMOC» adhesive is applied to the surface of the fiber so that it completely percolates the fiber and remains on the surface. The ends of carbon fiber strips are mechanically fixed with metal plates and bolts M10.

First of all, there should be established the cost of the materials necessary for the work on reinforcing the floor slab for each described technology. The total cost of materials and equipment is determined by the formula (1):

$$P_M = \sum_{i=1}^{n} (\rho_{мп} \cdot Q_{мп}),$$

where $\rho_{мп}$ – total value of the products of each individual material;

$Q_{мп}$ – the volume of each individual material..

For the *first method* is defined the overall length of each type of steel elements and multiplied by its density, double T-beams 14B1 = 571,41 kg, 16B1= 698,5 kg, 18B1 = 256,41 kg, 20B1 = 659,2 kg; Channel for columns 16U = 2905,32 kg.

Consequently, the total cost of metal elements is:

$$P_M = 23{,}2\ ^{\text{UAH}}/_{\text{kg}}\ (571{,}4\text{kg} + 698{,}5\text{kg} + 256{,}4\text{kg} + 659{,}2\text{kg} + 2905{,}3\text{kg}) = 118\ 106{,}5\ \text{UAH},$$

where 23,2 is the cost of one kilogram of steel constructions.

For the *second method* it is established that in order to strengthen the stretched zones of the slab floor, 376 rm of carbon fiber MapeWrap C UNI-AX 300/10 is required, which, with a strip width of 10 cm has a density of 300 g/m. The cost of such carbon fiber is 154,7 $^{\text{UAH}}/_{\text{rm}}$.

Thus, the cost of fiber needed to strengthen the entire plate is:

$$P_M = 154{,}7\ ^{\text{UAH}}/_{\text{rm}} \times 376{,}0\ \text{rm} = 58\ 167{,}2\ \text{UAH}.$$

The MapeWrap Primer 1 priming fluid consumption is 11.28 kg. With the price of 1 kg of grounding 740,76 $^{\text{UAH}}/_{\text{kg}}$, its total cost is:

$$P_M = 740{,}7\ ^{\text{UAH}}/_{\text{kg}} \times 11{,}28\ \text{kg} = 8\ 355{,}7\ \text{UAH}.$$

For adhesion of carbon fiber, adhesive MapeWrap 21 is used with a total consumption for an overlapping amounting 37,6 kg. The price of glue is 553,97 $^{\text{UAH}}/_{\text{kg}}$, so its total cost is:

$$P_M = 553{,}9\ ^{\text{UAH}}/_{\text{kg}} \times 37{,}6\ \text{kg} = 20\ 829{,}2\ \text{UAH}.$$

For the *third method* of amplification, analyzed in this article, 581.0 kg of steel stripe 4x50 mm are needed at its price of 23,2 $^{\text{UAH}}/_{\text{kg}}$. Consequently, the total cost of the steel strip for reinforcing the plate is:

$$P_M = 23{,}2\ ^{\text{UAH}}/_{\text{kg}} \times 581{,}0\ \text{kg} = 13\ 479{,}2\ \text{UAH}.$$

The total cost of the grounding «Consolid 1» is:

$$P_M = 227\ ^{\text{UAH}}/_{\text{l}} \times 9{,}5\text{l} = 2\ 156{,}5\ \text{UAH}$$

with its need amounting 9.5 liters and a price of 227 UAH/liter.

The total cost of adhesive «EDMOC», which is used when amplifying the plate according to this technology, is:

$$P_M = 288{,}0\ ^{\text{UAH}}/_{\text{kg}} \times 28{,}2\ \text{kg} = 8\ 121{,}6\ \text{UAH}$$

at its price 288 UAH/kg and need 28.2 kg.

For the *fourth method* of amplification 376.0 rm of unidirectional carbon fiber with density of 300 g/rm and a width of a strip of 10 cm a required, as indicated above, at a price of 125.0 UAH/m. Under these conditions, the total cost of fiber for the plate reinforcement is:

$$P_M = 125{,}0\ ^{\text{UAH}}/_{\text{rm}} \times 376{,}0\ \text{rm} = 47\ 000{,}0\ \text{UAH}.$$

The total cost of the grounding «Consolid 1» is:

$$P_M = 227{,}0\ ^{\text{UAH}}/_{\text{l}} \times 15{,}7\ \text{l} = 3\ 564{,}0\ \text{UAH}$$

with its need for 15.7 liters and a price of 227.0 UAH/liter.

The total cost of adhesive «EDMOC», used when amplifying the plate according to this technology, is:

$$P_M = 288{,}0\ ^{\text{UAH}}/_{\text{kg}} \times 75{,}2\ \text{kg} = 21\ 657{,}6\ \text{UAH}$$

As the need for glue «EDMOC» is 75.2 kg, and its price – 288 $^{\text{UAH}}/_{\text{kg}}$.

The calculation of the cost of materials needed to reinforce the floor slab is given in Table 1 and in Fig. 4.

Table 1 – Cost of materials for reinforcement of floor slab by different methods

Name of material	Cost of materials, UAH			
	supply of metal beams	«MAPEI» technology	gluing metal plates	gluing carbon fiber
Metal beams and columns	118106,5	-	-	-
MapeWrap C UNI-AX 300/10	-	58167,2	-	-
MapeWrap Primer 1	-	8355,7	-	-
MapeWrap 21	-	20829,2	-	-
Steel strip 50×4	-	-	13479,2	-
Consolid	-	-	2156,5	3564,0
EDMOK	-	-	8121,6	21657,6
Carbon fiber 300/10	-	-	-	47000,0
Anchor M10	-	516	516	516
Steel strip for mechanical fastening	-	345	-	345
Total cost	**118106,5**	**88213,1**	**23757,3**	**73082,6**

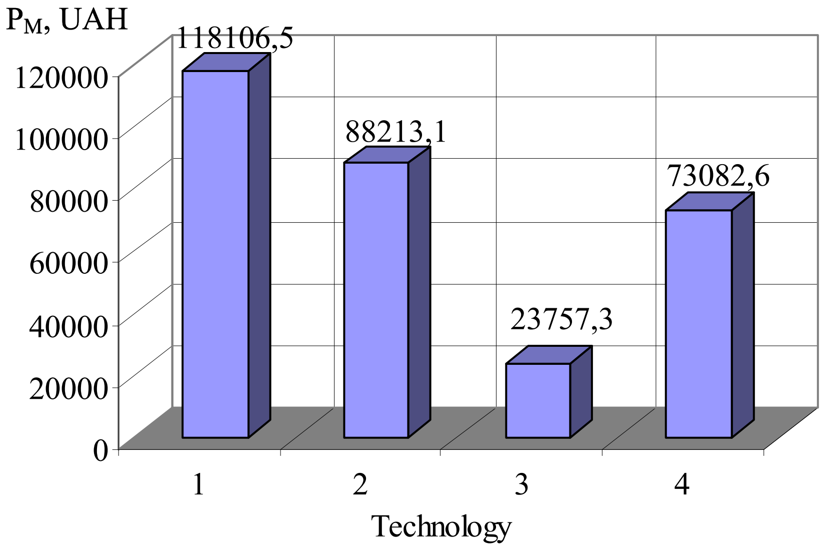


Figure 4 – Cost of materials for reinforcement of floor slab:
1 – supply of metal beams; 2 – «MAPEI» technology;
3 – gluing of metal plates; 4 – gluing carbon fiber

The second phase of the study TEP provides establishment of complexity and amount of wages for performed work at strengthening plates with different technologies. The complexity is determined by the normative values in accordance with the DBN, SGS, DSTU and Unified Norms and Prices, as well as by the values obtained by the results of their own timing. All labor complexity values are summarized in Table 2.

Table 2 – Calculation of labor complexity of reinforcing overlapping slabs by different methods and wages for these works

Working operation	The complexity and wages to enhance the floor slab according to the appropriate technology							
	supply of metal beams		«MAPEI» technology		gluing metal plates		gluing carbon fiber	
	Complexity, man/hour	Wages, UAH	Complexity, man/hour	Wages, UAH	Complexity, man/hour	Wages, UAH	Complexity, man/hour	Wages, UAH
Painting of metal elements	28,5	584,2	-	-	-	-	-	-
Cutting, assembly and welding	478,2	9803,1	-	-	-	-	-	-
Cleaning the surface	-	-	8,8	180,4	8,6	176,3	8,8	180,4
Priming the surface	-	-	10,2	209,1	9,6	196,8	10,2	209,1
Technological break	–				24 hours			
Gluing Carbon Fiber / Metal Plates	-	-	55,4	1135,7	336,3	6894,1	55,4	1135,7
Fixing ends of stripes with anchors	-	-	19,2	393,6	17,6	360,8	19,2	393,6
Total	**506,7**	**10387,3**	**93,6**	**1918,8**	**372,1**	**7628,0**	**93,6**	**1918,8**

Wages, salaries of workers performing amplification are defined in UAH based on the data book «Pricing in construction». The indicated costs are 20.5 UAH/hour for the worker with a grade of 3.8. The calculation of labor and wages is shown in Table 2. The histogram of labor intensity (Fig. 5) and wages (Fig. 6) on the reinforcement of the plate by different technologies is constructed according to the data of Table 2.

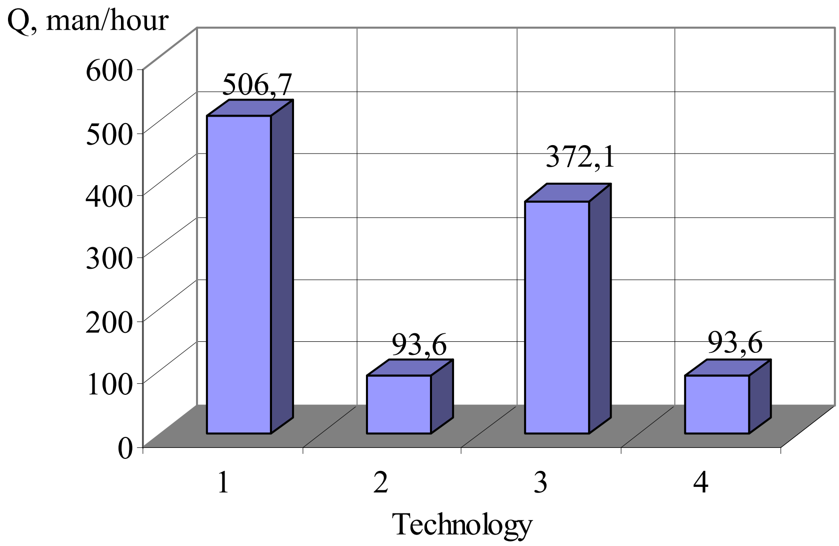


Figure 5 – The work complexity on reinforcement of the floor slab:
1 – supply of metal beams; 2 – «MAPEI» technology;
3 – gluing of metal plates; 4 – gluing carbon fiber

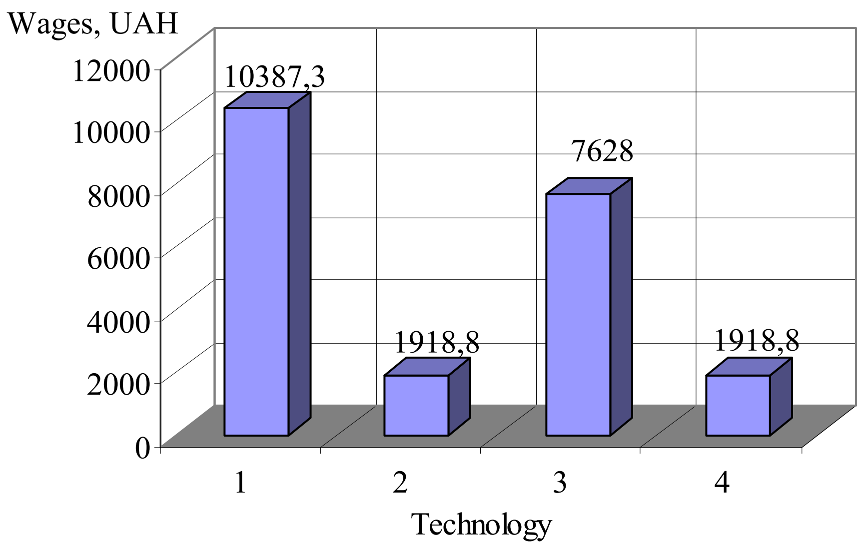


Figure 6 – Wages for strengthening slabs:
1 – supply of metal beams; 2 – «MAPEI» technology;
3 – gluing of metal plates; 4 – gluing carbon fiber

The duration of the floor slab reinforcing process (Fig. 7) is determined by adding the duration of the work operations with the length of the technological breaks between them according to the Table 2. It is considered that the painting of metal constructions and surface cleaning is carried out by one worker, the assembly and welding work is carried out by three workers, and all other processes are performed by two workers.

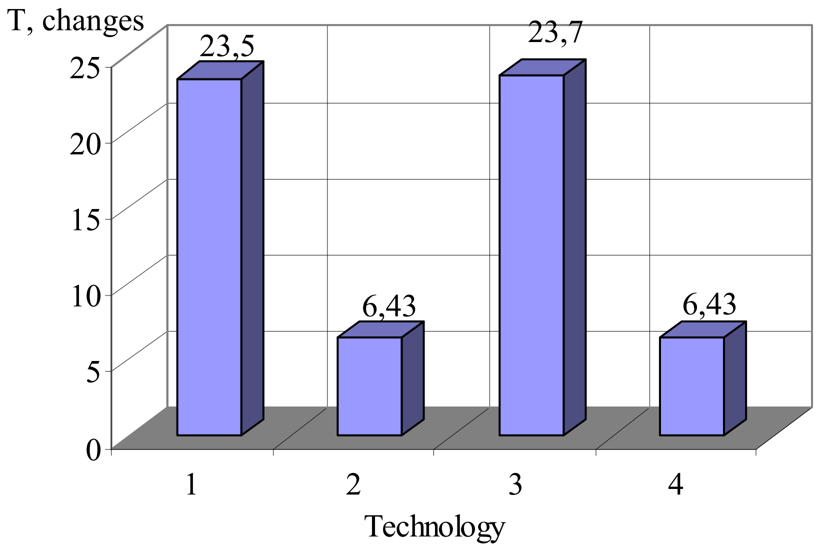


Figure 7 – Work duration on floor slabs reinforcement:
1 – supply of metal beams; 2 – «MAPEI» technology;
3 – gluing of metal plates; 4 – gluing carbon fiber

Conclusions. According to the study of technical and economic performance of different amplification technologies applied to the same floor slab, it was found that the cost of materials, complexity and wages for performing such works on reinforcing floor slab with the amplification method of metal structures supply is the highest. At the same time, labor intensity, wages and the duration of work execution are the lowest with the reinforcement of the floor slabs by the technology of the company MAPEI and the bonding of carbon fiber by the developed technology. The cost of materials for reinforcing the slab by the developed technology, namely, bonding of steel plates is the lowest, but the complexity, wages and length of work under such technology is much higher than other investigated technologies of reinforcement by external enhancement.

The research results are a strong argument in choosing a particular way of structures strengthening, one of the decisions of feasible study

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METHODOLOGICAL PRINCIPLES OF CALENDAR PLANNING BY PROBABILISTIC CHARACTER OF CONSTRUCTION

It is defined that the tasks of reducing the cost of construction products should not be solved by saving the direct costs, but due to objective reduction of costs by rationality of the production work organization. It is established that construction works are fulfield in prone to changes of the environment conditions, so not only these factors should be taken into account in the process of construction, but also those that may only be predicted with a certain probability. It was developed the dependence of the work duration calculation both with the usage of a set of construction machine and without its usage by applying the coefficient of the construction project sensitivity to the factors of a changing environment. The proposed calculation allows to reduce the failure risk of the commissioning project terms and provides an opportunity to accelerate the construction process in the given specific conditions. It was carried out the estimation of the coefficient of sensitivity in different local conditions depending on the influence of various factors: weather conditions; provision of material resources; number and qualifications of workers.

***Keywords:** scheduling, duration of construction, coefficient of sensitivity, probabilistic nature of construction.*

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МЕТОДИЧНІ ПРИНЦИПИ КАЛЕНДАРНОГО ПЛАНУВАННЯ ПРИ ЙМОВІРНІСНОМУ ХАРАКТЕРІ БУДІВНИЦТВА

Відомо, що завдання зниження вартості будівельної продукції повинні вирішуватися не шляхом економії на прямих витратах, а за рахунок об'єктивного зниження витрат шляхом раціонально організованого виробництва робіт. Установлено, що будівельне виробництво функціонує в умовах схильного до змін середовища, причому на процес будівництва впливають не тільки заздалегідь ураховані чинники, але й такі, що можуть бути передбачені лише з певною ймовірністю. Наведено залежності з розрахунку тривалості проведення робіт як при використанні комплекту будівельних машин, так і без нього із застосуванням коефіцієнта сенситивності будівельного проекту до чинників мінливого середовища. Запропонована методика розрахунку дозволяє зменшити ризик зриву термінів здавання об'єкта та надає можливість прискорити процес будівництва в конкретних умовах. Виконано оцінювання коефіцієнта сенситивності в різних місцевих умовах залежно від впливу різноманітних чинників: погодні умови; забезпечення матеріальними ресурсами; кількість і кваліфікація робітників.

***Ключові слова:** календарне планування, тривалість будівництва, коефіцієнт сенситивності, ймовірнісний характер будівництва.*

Introduction. Scientific studies of calendar planning models and methods of control, project management and forecasting of changes in schedules in the course of works are relevant for today and give the significant impact on the results of work of construction organizations [1, 2].

All activities of an organization include risks. Organizations manage risk through its identification, analysis and subsequent evaluation, whether the risk will be changed through processing to accord the risk criteria. Throughout this process they exchange information and consult with stakeholders, carry out risk monitoring and analysis, and fulfill the management actions to counteract risks, to reduce the impact of risks on the construction process.

The main factors that determine the efficiency of construction are the price and quality. The tasks of the costs reducing of the construction products should be solved not by saving the direct costs in the form of buying cheap materials or using «cheap» labor, but by means of the objective costs reduction by optimizing the planning and organizing the construction process [3].

In the field of construction there are numerous problems that should be solved by improving the methods of scheduling. Among these problems are the problems of construction time and costs exaggeration; low productivity; low quality products; downtime of workers and equipment; poor working conditions; the wrong ratio of workers of different professions (adjusters, welders, carpenters, etc.) in each area of work (due to the lack of sufficient numbers of workers of some professions, workers of the other professions will work with low productivity) [4].

So, it is normal that within the equal position the competitive advantage is given to the construction company with higher labor productivity and fewer downtime.

The costs of the construction product depend not only on the costs of used materials, machines/mechanisms, wages of workers, but also depends on how rationally and productively the construction works are organized [3].

Review of the latest research sources and publications. Some of the tasks of the construction production organizing are control, management and optimization of the scheduling in the process of their implementation, taking into account the variability of the working environment and the influence of various groups of factors on the planned actions with the aim to complete the project in the most favorable terms.

In these conditions it is extremely important to ensure the coordination of the work of all construction participants, to subordinate their activities to the general rhythm of construction, to take into account the influence of numerous random factors that cause deviations of the entire system from the planned course of work.

The construction industry operates in conditions prone to environmental changes (supply disruptions, accidental breakdowns of machines and mechanisms, weather changes, etc.), and the construction process is affected not only by factors that can be taken into consideration in advance, but also a number of such influences which can be provided only with a certain probability [5].

The internal risks are such risks that different parties within a company can undertake. Subcontractors, stakeholders, designers etc. fall under internal risks. External risks are those that are out of the project organization range to control, for example weather or introduction of new laws. Project risks represent those risks that could occur during a project and are associated with time scheduling, cost, quality etc.

Risk management does not only involve time and cost factors in a project but it is also the way of understanding the problems that might emerge before it is too late, and by doing so the processes can be more easily controlled. The quantification done in the risk management is good to have as a reference to highlight the different areas that are in need of further investigation, clarification or design [8].

The most frequent causes of project implementation failures are: a lack of resources, insufficiently skilled workforce, misallocation of funds and unrealistic terms, which is a consequence of poor planning quality [1].

It is very important to observe the schedule of construction and installation works, as the disruption of the schedule may introduce disorganization into the production process.

Definition of unsolved aspects of the problem. The creation of resource and time buffers is a typical technique to combat the disruption of the work schedule [2]. The specificity of the construction industry does not allow applying the theory of constraints in its classical form for creating resource and time buffers.

Risk assessments in the organizational and technological models of construction production can be carried out using fuzzy logic [7–9].

Determination of the construction facilities duration by average indicators can be carried out according to normative documents [10], but it should be noted that the schedule is unique for each construction organization.

Problem statement. It is improving the methodological principles of scheduling for the probabilistic nature of construction by calculating the duration of construction and installation work, taking into account their sensitivity to various external and internal factors that cause deviations from the planned schedule of work.

Main material and results. The sequence of work, conditioned by the technology of construction processes, is the first to influence the duration and cost of housing, loading of resources, etc.

Despite the fact that there are general guidelines, they are not intended to provide the risk identical management in all organizations. When creating and applying calendar plans and risk management structures, it is necessary to take into account the different needs of a particular organization, its specification, objectives, context, structure, activities, processes, functions, projects, products, services or assets and specific practical methods used.

It should be noted that the tasks of scheduling are usually considered in the traditional formulation according to the accepted technology of conducting works, starting from technological routes and standards. At the same time, studying of technology options or, at least, using of various technological routes in the analysis will allow to identify the optimal options for loading resources, respectively, to increase productivity and reduce costs.

Moreover, when considering options, it is not necessary to engage in simple search, it will be correctly guided by logical dependencies and experience, which will allow to identify the most promising ways and, having considered them, choose the optimal variant of the schedule.

The chosen schedule will also allow to minimize losses, such as: excess stocks of raw materials and materials; reducing or eliminating the intersection of workspaces with simultaneous carrying out of various works; reducing the probability of spoilage or loss of materials; improving quality (no need to redo the work); eliminating excessive movement of people and equipment through the construction site.

The implantation of risk management and ensuring its continued effectiveness requires the adoption of a strict and permanent commitment on the part of the management of the organization, as well as strategic and detailed planning to fulfill obligations at all levels.

Also, we note that the methods use an imperfect regulatory framework, based, as a rule, on averaged indicators.

The plan of combating risks must be developed throughout the organization, ensuring the application of risk management policies and the inclusion of risk organization in all procedures and processes of the company. The risk management plan can be integrated into other plans of the organization, for example, into a strategic plan.

On Fig. 1 shows the scheme for accounting for the probabilistic nature of construction production in the scheduling (construction and installation work).

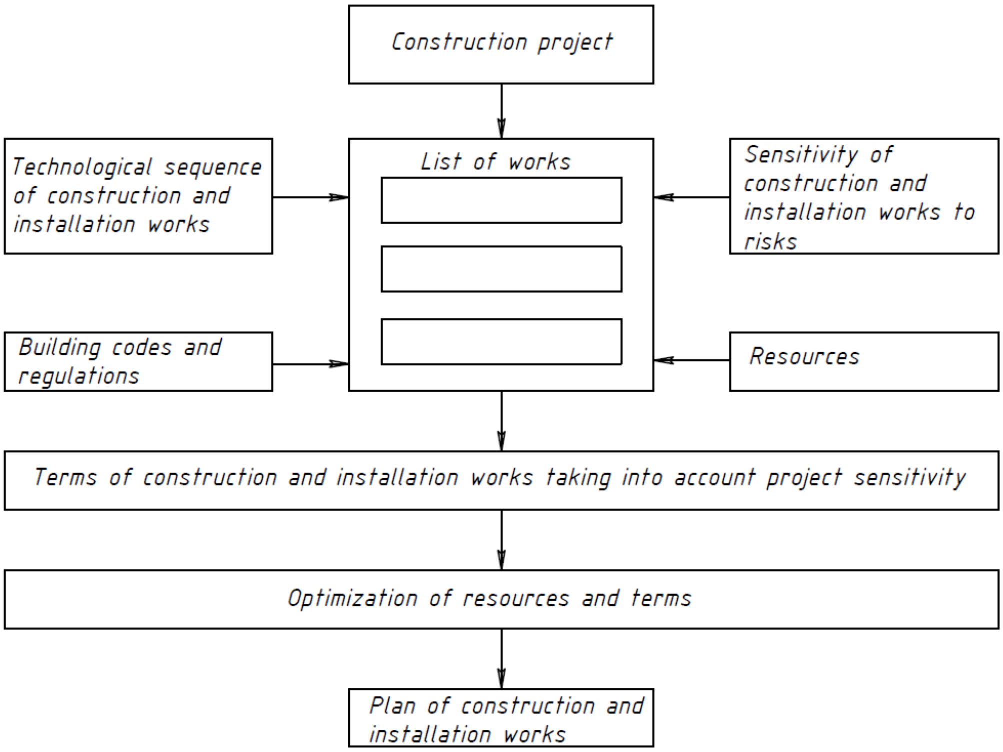

Figure 1 – Scheme of accounting for the probabilistic nature of construction production in the calendar planning

The way expressing the consequences and the probability of their occurrence, and the way they are combined to determine the level of risk, should reflect the type of risk, the available information and the purpose for which the result of the risk assessment should be used. All these principles must comply with the risk criteria. It is also important to consider the interdependence of various risks and their sources.

It is proposed to use the sensitivity coefficient in calculating the duration of construction to take into account the probabilistic nature of the process.

The duration of work (in working days) is determined by the formula (1):

$$T = \frac{Q \cdot K_c}{n \cdot Z}, \tag{1}$$

where Q – labor intensity of work, person-days;

K_C – the sensitivity coefficient;

n – the number of workers who can occupy the front of work;

Z – shifts.

The following formula (2) can be used to estimate the duration of the work performed with the use of a set of machines under the conditions of a probabilistic nature of the construction industry:

$$T_{p\partial} = \frac{W \cdot K_{CM}}{M_z \cdot Z} + T_P \cdot K_{CP}, \tag{2}$$

where W – scope of the work;

K_{CM} – the sensitivity coefficient for the main work performed;

M_Z – capacity of a set of machines in shift;

Z – shifts;

T_P –duration of preparation of cars for performance of works (installation, sampling);

K_{CP} – the sensitivity coefficient for preparatory work.

Organization and management of processes is always present, but the proposed calculation can reduce the risk of commissioning time disruption and gives the opportunity to speed up the construction process, as in these specific construction conditions, specialists are aware of the strengths and weaknesses of the project. Therefore, during calculating or adjusting the duration of work, a time reserve (insurance) is laid in the form of possible failures (for example, equipment failure, material shortage, theft, staff diseases, etc.). and for the purposes of project acceleration if appropriate conditions are created for it.

As for the impact of risk factors that have little effect on the implementation schedule, such a minor change in the schedule through the management influence of directorate makes it possible to catch up and enter the schedule.

The evaluation of the sensitivity coefficient can be determined in different local conditions, as shown in Table 1 – 4. It is also possible to assess the sensitivity coefficient for other risk factors.

Table 1 – Selecting the sensitivity coefficient to the provision of material resources

№	Assessment of the state of material resources	Coefficient of sensitivity
1	Disruptions in the supply of material resources (unreliable suppliers, logistics violation, other reasons) and / or insufficient quality (many defects)	>1
2	The quantity and quality of material resources do not contribute to effective work (not enough tools and equipment, poor quality of overalls, fuel and lubricants, building materials, etc.)	1
3	The amount of resources is sufficient to carry out the work, their quality is high, which contributes to higher labor productivity and a shorter work execution time	<1

Table 2 – Selecting the sensitivity coefficient taking into account the number of workers

№	Assessment of the number of workers	Coefficient of sensitivity
1	The number of workers is less than required for various reasons (employed at other sites, diseases, absenteeism, etc.)	>1
2	The number of workers is sufficient in quantity, but the wrong ratio between workers of different professions (welders, installers, carpenters, etc.), which does not contribute to effective work	1
3	The number of workers is sufficient in quantity with the correct ratio between workers of different professions (welders, installers, carpenters, etc.), which contributes to higher labor productivity and shorter work execution	<1

Table 3 – Selecting the sensitivity coefficient when taking into account the qualifications of workers and engineers

№	Qualification assessment	Coefficient of sensitivity
1	Workers have insufficient experience in carrying out various jobs, low qualification of engineers and technical workers	>1
2	The qualification of workers and engineers is sufficient to carry out the work, but it is not sufficient to exceed the norms of time	1
3	The qualification of workers and engineers is high, there is a material interest in overfulfilment of norms	<1

Table 4 – Selecting the sensitivity coefficient when taking weather conditions into account

№	Assessment of weather conditions	Coefficient of sensitivity
1	Winter period	>1
2	Autumn and spring periods	1
3	Summer period	<1

If the sensitivity coefficient factor is shown K> 1, this can be, for example, K = 1.1; K = 1.2, if, in the opinion of experts, the scope of work will be performed faster, respectively, by 10%, by 20%. The value of the sensitivity coefficient is K<1, it can be, for example, K = 0,9; K = 0.8, if, according to experts, the scope of work will be executed more slowly, respectively, by 10%, 20%.

Conclusions.

1. Structural chart of accounting for the probabilistic nature of construction production in the calendar planning.

2. There were developed the calculation dependencies of the work duration, both with the use of a set of construction machines, and without their use.

During the research, it was revealed:

The duration of construction is recommended to be determined taking into account the probabilistic nature of the construction process.

Risk is an important factor in the construction industry and its influence and impact only keeps increasing with infrastructure projects becoming ever more complex and therefore more exposed to high risks. Depending on the severity of the problem, the consequences can result in a heavy cost due to a failure or oversight in the risk evaluation. An assessment of the risk rating should provide an understanding of which risks are epymost affected for the particular organization of construction.

It has been analyzed the influence on the sensitivity coefficient of various factors, such as material resources, the number and qualification of workers, weather conditions.

It is planned to consider the determination of the duration for other various works taking into account the probabilistic nature of the construction industry in further studies.

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PROBABILITY OF THE UTILITY NETWORKS DOUBLE-RING STRUCTURES' CONNECTIVITY FOR SITES WITH VARIOUS RELIABILITY

The analytical description study results on probability of connectivity for the structures used to model the reliability of various complicated systems are presented. Expressions are formed to calculate the connectivity probability of systems that have structural redundancy. The characteristic components of the formulas are distinguished and they are systematized according to their increasing complexity and the number of elements. The features of the equations' structure permitting to conveniently formulate the probability of the structures connectivity in the process of their construction and transformations are determined. The examples show the formation of formulas and their structural parts at various levels of complexity. The use of the ratio value of the network structure element's unreliability and its reliability is justified, thus reducing the awkwardness of exact expressions for the connectivity probability of network structures and substantially improves the compactness and convenience of using the equations.

***Keywords:** structural modeling, connectivity probability, reliability of a system with different reliability of elements*

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ЙМОВІРНІСТЬ ЗВ'ЯЗНОСТІ ДВОКІЛЬЦЕВИХ СТРУКТУР ІНЖЕНЕРНИХ МЕРЕЖ З РІЗНОЮ НАДІЙНІСТЮ ДІЛЯНОК

Наведено результати дослідження аналітичного опису ймовірності зв'язності структур, що використовуються для моделювання надійності різних складних систем. Побудовано вирази обчислення значення ймовірності зв'язності систем, що мають структурний резерв. Виділено характерні складові формул, систематизовано їх у порядку збільшення складності та кількості елементів. Установлено особливості будови рівнянь, які дозволяють зручно складати вирази ймовірності зв'язності структур у процесі їх конструювання й перетворень. На прикладах показано утворення формул і їх структурних частин різних рівнів складності. Обґрунтовано використання величини співвідношення значення ненадійності елемента структури мереж та його надійності, що скорочує громіздкість точних виразів імовірності зв'язності структур мереж і суттєво підвищує компактність та зручність використання рівнянь.

***Ключові слова:** структурне моделювання, ймовірність зв'язності, надійність системи з різною надійністю елементів.*

Introduction. Development of science and technology widely uses different technical systems (TS), including engineering networks in all branches of industry and management. Qualitative characteristics of compound systems are steadily increasing. However, the problem of security remains the major one. Functions performed by the present day engineering systems are complicated enough, and their tasks are extremely responsible.

The need to integrate different scientific fields in the problem of systems' reliability is explained by the fact that the problem is complicated. Theoretical studies indicate [1 – 7] that it is impossible to achieve a significant increase in the systems' reliability level taking separate isolated measures. Integrated solution of tasks enhances the efficiency of each method and helps obtain qualitatively new results.

Determining the probability of connectivity is an important task in designing and reconstruction of utility networks: water, gas, mobile operators' networks, etc. [1].

Analysis of recent research sources and publications. Recent studies [5] have analyzed the reliability of the network structures with elements that have the same reliability values. In the operating objects, the properties of reliability and efficiency are in the mutual conflict [1, 5]. To raise the reliability of different systems additional expenditures are necessary, thus reducing their efficiency. The study [8] represents the optimum network design with series-parallel structures. In [10], the method of rational valuation of reliability in systems with parallel and serial-parallel structures is suggested.

In [11], the main reasons of inadequate development on redundant utilities' reliability problems are highlighted. The concept of interval and boundaries of the efficient improvement of the redundant utilities' reliability is suggested. The attention is focused on the approaches to the identification of the basic patterns in technical systems' failures, the probability model of the technical systems' non-failure is suggested. Comparison of the probability time of technical systems' non-failure operation with different configurations is performed, the influence of structure forms on the technical systems' uptime is shown.

Determination of still unsolved aspects of the general problem. In mathematical modeling of reliability it is necessary to accurately determine the probability of connectivity for structures with varying reliability of elements.

Problem statement (formulation of the aim and methods to research the problem). Probability expressions should be suggested for connectivity of network systems with areas of varying reliability. At a trial designing or reconstruction of structurally complicated redundant systems, this model will be useful for better decision making.

A method should be suggested for constructing analytical expressions of the exact connectivity probability value for structures with varying reliability elements having two cycles, and a random number of sites in cycles in a compact recording format that is convenient for computer simulation.

It is necessary to display dependence expressions on the connectivity probability for structures that are transformed by means of removing or adding new sites.

Basic material and results. The form of network structure significantly effects its reliability. Important is the connectivity of the structure that is a topological feature of the network. To determine the structural reliability the concept [5] operating condition is applied, when all components of the structure are connected to each other. The system may have a structural reserve that is formed by closure of connections between the elements in the form of cycles. Accordingly, non-operating condition arises when connection is lost with the only node of the structure. The ultimate operating condition [5] is formed through the connection of all the nodes of the structure by sites of wooden cover. In this case, the system's structure is extensive and has no connection closures between the nodes.

Probability of the network structure connectivity is the probability of the compatible events sum:

$$R = \sum_{i=1}^{T} P(B_i) - \sum_{i \neq j} P(B_i \cap B_j) + \ldots + (-1)^{T-1} P(B_1 \cap B_2 \cap \ldots \cap B_T), \tag{1}$$

where $P(B_i)$ – is the formation probability of the wooden cover B_i that connects all the nodes of the structure;

$P(B_i \cap B_j)$ – is the probability of the simultaneous existence of the wooden covers B_i and B_j in the system's structure.

Let us consider modeling of the structure's reliability in more general terms. Let us assume, that sites are having different reliability: $r_1^d \neq r_2^d \neq \ldots \neq r_m^d$. Limitation: the nodes' reliability is considered absolute: $r_1^v = r_2^v = \ldots = r_m^v = 1$.

Let us take two sites and connect them so as not to close the structure. Then the reliability of the system, with two sites connected in series, will be: $R=r_1 \cdot r_2$, with three sites: $R=r_1\ r_2\ r_3$, with n sites: $R=r_1\ r_2,\ldots,r_n$. The structures of systems with elements connected in series can have a branched (extensive) form. They have no structural reserve, because they have not a high level of reliability. The reliability value (probability of connectivity) varies within the range of rational numbers (0, 1). This indicates that with increasing of the number of sites connected in series, the level of the system's structural reliability is reducing nonlinearly.

Let us close the structure into the cycle for the two sites to have common ends. The simplest structural reserve of the system is formed, when there are not one, but two connections between the nodes. The reliability equation for the redundant structure consisting of two sites with different reliability $r_1 \neq r_2$, in the form of parallel connection between the two nodes v_1 та v_2 looks as follows [1]

$$R = 1 - \prod_{i=1}^{2} (1 - r_i), \tag{2}$$

or:

$$R = -r_1 r_2 + r_1 + r_2. \tag{3}$$

Let's consider the formation of more complicated structures with a single cycle. Let us add to the structure one site with reliability r_3 into the cycle C^2 and the new node $C_1^2 \cup d = C_1^3$. Then another structure will be obtained in the form of a triangle with the following equation of the connectivity probability:

$$R = -2r_1 r_2 r_3 + r_1 r_2 + r_1 r_3 + r_2 r_3. \tag{4}$$

With the increasing number of sites and nodes in the structure S with a single cycle $C_1^i \cup d = C_1^{i+1}$ its connectivity probability expression changes. For the first summand, the coefficient value increases in magnitude per unit, its value being equal to the number of sites smaller by one: $(p-1)$. The first summand is the product of the coefficient and the reliability values of all sites:

$$-(p-1)\prod_{i=1}^{p} r_i.$$

The number of succeeding additive components is increasing by one, too. The total number of these components is $-p$, i.e. it is equal to the number of sites. Each summand is the product of the reliability values r_i i.e. it equals to the number of sites, but for the one having the relevant number in ascending order:

$$\sum_{j\neq i}^{p}\frac{\prod_{k=1}^{p} r_k}{r_k}.$$

Table 1 presents graphical models of structures with one cycle C^p with a number of sites p < 6 and the respective reliability expressions. The general equation for the reliability of redundant structures, having a single cycle $S \subset C_i^p$, $i = 1$ with p sites is written:

$$R = -(p-1)\prod_{i=1}^{p} r_i + \sum_{j\neq i}^{p}\frac{\prod_{k=1}^{p} r_k}{r_k}, \tag{5}$$

where r_i –is reliability of the i-site.

With the increasing complexity of the structure, the study performed using this approach is getting quite cumbersome. So, let us apply systematization and structuring of the research.

Let's start with the simplest redundant structure, the structure consisting of the 2 sites, connecting two nodes (Fig. 1). The total of possible unique operating conditions of the redundant structure consisting of 2 sites will be three. Analytically this is expressed as:

$$R = -r_1 \cdot r_2 + r_1 + r_2. \tag{6}$$

Let us assume the concordance that the likely failures of sites are marked with counting numbers in a series of natural numbers. This will help determine patterns of constructing a compound network structure. Then the structure connectivity probability equation is the product of all sites' reliability multiplied by the sum of one and the ratio of unreliable to reliable sites:

$$R = M(1+\sum_{i=1}^{2} e_i) \tag{7}$$

where $e_i = \frac{1-r_i}{r_i}$, $M = \prod_{i=1}^{n} r_i$ $i = 1,2,...,n$

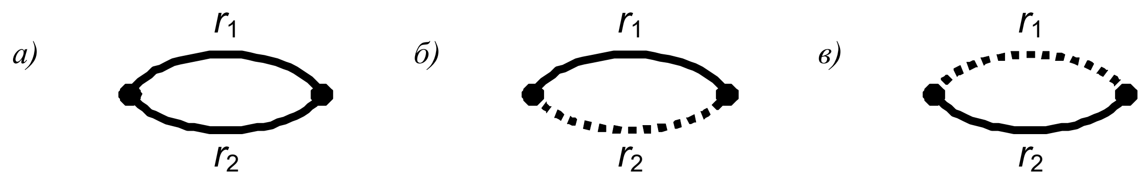


Figure 1 – Operating conditions of the 2-sites redundant structure:
a – all sites are operating; *b* – failure of site 2; *c* – failure of site 1

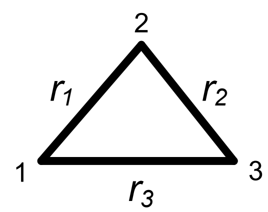


Figure 2 – Operating condition of the elementary structure without failures of sites

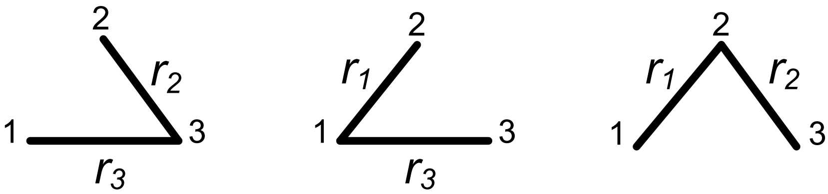
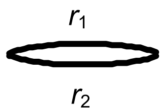
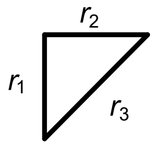
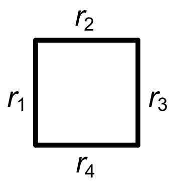
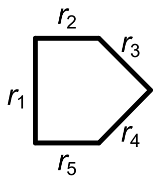


Figure 3 – Operating conditions of the elementary structure when one of the sites fails

Some results of the consistent input of sites into the single-ring structure and defining their connectivity probability expressions are presented in Table 1.

Table 1 – Connectivity probability of single-ring structures comprising 2-5 sites and nodes $S \subset C_i^p,\ i = 1,\ p<6$

Numerical character	Graphic symbol of a structure	Connectivity probability expression
$m=2$ $p=2$ $n=1$		$R=M(1+\sum_{i=1}^{2} e_i)$
$m=3$ $p=3$		$R=M(1+\sum_{i=1}^{3} e_i)$
$m=4$ $p=4$		$R=M(1+\sum_{i=1}^{4} e_i)$
$m=5$ $p=5$		$R=M(1+\sum_{i=1}^{5} e_i)$

Thus, reliability of the structure $S^p \subset C_v^l,\ v=1$ equals:

$$R = M F_d, \tag{8}$$

where $M = \sum_{i=1}^{p} r_i \quad i = 1,2,...,p$, $F_d = 1 + \sum_{i=1}^{p} e_i$, $e_i = \frac{1-r_i}{r_i}$,
r_i – reliability value of the i-site of the structure S.

Property 1. When entering i-site r_i and the top into structure S with a single cycle $v = 1$, as a part of the second summand of the equation (8), there appears the value $e_i = \frac{1-r_i}{r_i}$ of this site, which is the ratio of its values of unreliability and reliability. Accordingly, with removal of i-site the value e_i disappears from the expression (8).

Let us raise the complexity of structure S and, maintaining the structural reserve, let us enter the second cycle into the structure. The reliability equation for the three parallel sites r_1, r_2, r_3 is:

$$R = 1 - \prod_{i=1}^{3}(1 - r_i) \tag{9}$$

or, having performed the transformation,

$$R = M\left(1 + e_5 + \sum_{i=1}^{4} e_i(1 + e_5) + \sum_{i=1}^{2} e_i \sum_{i=3}^{4} e_i\right). \tag{10}$$

Let us enter one site and one top into one of the cycles. Accordingly, we'll obtain a change in the connectivity probability of the structure, which is expressed in Table 2.

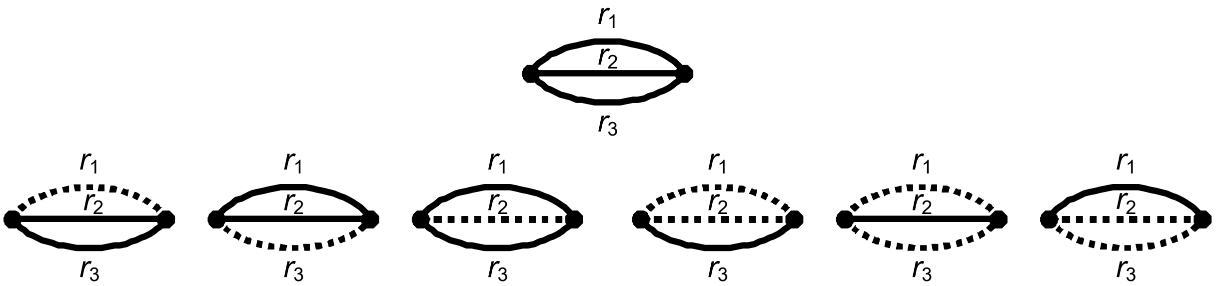


Figure 4 – Seven operating conditions of the redundant structure consisting of 3 sites

Analytically, the connectivity probability of the structure equals:

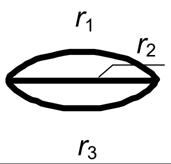
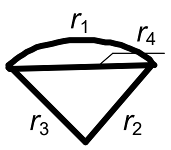
$$R = r_1 \cdot r_2 \cdot r_3 - r_1 \cdot r_2 - r_1 \cdot r_3 - r_2 \cdot r_3 + r_1 + r_2 + r_3, \tag{11}$$

or:

$$R = \prod_{i=1}^{3} r_i - \sum_{j=1}^{3} \frac{\prod_{j=1}^{3} r_j}{r_j} + \sum_{k=1}^{3} r_k . \tag{12}$$

Assuming that variable $r_i = \frac{1}{1 + e_i}$, $M = \prod_{i=1}^{3} r_i$, $i = 1,2,3$, after transformations, we'll obtain the equations included into Table 2.

Table 2 – Connectivity probability for double-ring structures with three sites $S^p \subset C_v^l$, $v = 2$, $p<4$

№	Network structure	Connectivity probability formula
1		$R = M\left(1 + e_3 + \sum_{i=1}^{2} e_i(0 + e_3) + \sum_{i=1}^{1} e_i \sum_{i=2}^{2} e_i\right)$
2		$R = M\left(1 + e_4 + \sum_{i=1}^{3} e_i(1 + e_4) + \sum_{i=1}^{1} e_i \sum_{i=2}^{3} e_i\right)$

Property 2. At entering another cycle with *h*-sites into the single-cycle structure, the structural connectivity is formed, which is described by the additive component:

$$F_C = e_h + \sum_{i=1}^{h-1} e_i(1+e_h) + \sum_{i=1}^{j} e_i \sum_{i=j+1}^{h} e_i \ .$$

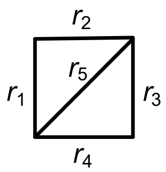
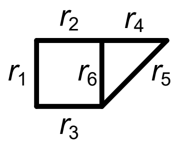
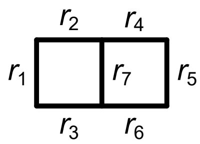
In the connectivity probability equation:

$$R = M\,(1+\sum_{i=1}^{p} e_i + F_C)\,, \tag{13}$$

where $e_i = \dfrac{1-r_i}{r_i}$, $M = \prod_{i=1}^{p} r_i$, $i = 1, 2, ..., p$, r_i – reliability values of *i*-site of structure *S*, *j* – is the number of sites in the first cycle C_1^l.

With the increasing number of sites in the cycles the additive component of the connectivity probability equation varies. The connectivity probability of structures with two cycles and the number of sites from five to seven are presented in Table 3.

Table 3 – Connectivity probability of structures with two cycles and the number of sites from five to seven $S^p \subset C_v^l,\ v = 2$, p<6

	Networks structure	Connectivity probability formula
m=4 *p=5*		$R=M\left(1+e_5+\sum_{i=1}^{4}e_i(1+e_5)+\sum_{i=1}^{2}e_i\sum_{i=3}^{4}e_i\right)$
m=5 *p=6*		$R=M\left(1+e_6+\sum_{i=1}^{5}e_i(1+e_6)+\sum_{i=1}^{3}e_i\sum_{i=4}^{5}e_i\right)$
m=6 *p=7*		$R = M\left(1+e_7+\sum_{i=1}^{6}e_i(1+e_7)+\sum_{i=1}^{3}e_i\sum_{i=4}^{6}e_i)\right)$

Thus, reliability of the structure $S^p \subset C_v^l,\ v = 2$ equals:

$$\begin{aligned} F_C &= e_h + \sum_{i=1}^{h-1} e_i(1+e_h) + \sum_{i=1}^{j} e_i \sum_{i=j+1}^{h} e_i \\ R &= M\left(1+e_p+\sum_{i=1}^{p-1} e_i(1+e_p) + \sum_{i=1}^{j} e_i \sum_{i=j+1}^{p-1} e_i\,)\right) \end{aligned} \tag{14}$$

where $e_i = \dfrac{1-r_i}{r_i}$, $M = \prod_{i=1}^{p} r_i$, $i = 1, 2, ..., p$,

r_i – reliability values of *i*-site of structure S^p,

p –is the number of sites in the structure,

j – is the number of sites in the first cycle C_1^l,

v – is the number of cycles,

l – is the number of sites in the *i*-cycle.

Conclusions. The equation for the exact value of the connectivity probability of structures with varying reliability of elements, having two cycles and a random number of sites in the cycles, is obtained.

A compact form to record reliability equations is obtained, their structure being convenient for computer simulation.

The structure of the suggested dependences permits obtaining new expressions of connectivity probability for structures, transformed by removing or adding new sites. This approach helps apply them in the computer modeling of connectivity probability of network structures in various fields of engineering.

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MOISTURE EFFECTS ON THE BUILDINGS ENCLOSING STRUCTURES

Analysis of the most common damages due to moisture effects on brick, reinforced-concrete and wooden buildings enclosing structures has been carried out. Causes of their occurrence and prevention ways have been analyzed. The most dangerous moisture types such as constructional, soil, atmospheric, operational, hygroscopic and condensed have been outlined. Measures to ensure protection against water-saturation of building elements such as eaves size, walls waterproofing, available airways, protective painting, hydrophobic impregnation, sufficient ventilation, heating, water supply systems and draining timely repairs have been recommended.

Key words: *damage, moisture effects, brick, reinforced-concrete, wooden buildings enclosing structures*

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ВПЛИВ ВОЛОГИ НА ОГОРОДЖУВАЛЬНІ КОНСТРУКЦІЇ БУДІВЕЛЬ

Виконано аналіз найбільш поширених пошкоджень від впливу вологи цегляних, залізобетонних і дерев'яних огороджувальних конструкцій будівель, проаналізовано причини їх виникнення та шляхи запобігання. Виділено найбільш небезпечні впливи від таких видів вологи, як будівельна, ґрунтова, атмосферна, експлуатаційна, гігроскопічна, конденсат. Рекомендовано для забезпечення незамокання елементів будівель достатні розміри карнизних звисів, наявність гідроізоляції стін, продухів, захисного фарбування, гідрофобного просочування, достатньої вентиляції, опалення, своєчасних ремонтів систем водопостачання та водовідведення.

Ключові слова: *пошкодження, вплив вологи, цегляні, залізобетонні, дерев'яні огороджувальні конструкції будівель.*

Introduction. Buildings enclosing structures moisture conditions is connected with their thermal behavior [5, 6] and should be complied with requirements [1]. Therefore, it is necessary to pay sufficient attention to buildings enclosing structures protection from hydration with construction (basic), soil, and atmosphere, operational, hygroscopic and condensed moisture. Constructional materials moisture has a negative impact on hygiene and operational performance of buildings enclosing structures, and results in premature failure of constructions under the temperature effects influence [2].

Recent studies and publications analysis. Moisture effects on roofing system enclosing structures at heat insulation, repair and reconstruction of flat roof are outlined in [3, 4]. Moisture effects on enclosing structures of foam mortar [7] structural ceramics [8] facade systems with external plaster layer [9] and ventilated interlayers [10] have been researched. The issues of buildings enclosing structures protection from moisture effects, frost destruction were researched by various specialists, such as [11, 12]. Due to thermal modernization of existing housing facilities, the need to assess buildings enclosing structures technical condition has appeared; the damage from moisture effects to brick, reinforced-concrete and wooden buildings enclosing structures has been systematized.

Purpose of the paper is to review the most common damages of moisture effects on building brick, reinforced-concrete and wooden buildings enclosing structures as well as to analyze causes of their occurrence and their prevention ways.

Main material and results. A special effect on buildings bricks enclosing structures has ground, atmospheric, operational, hygroscopic and condensed moisture. Operational moisture is available in premises with wet processes (showers, baths, car washes, etc.), increased moisture level inside the premises leads to soaking of brickwork that increases its thermal conductivity. Moisten brickwork has intense sorption processes of brickwork moisture extension vertically and horizontally. Traditionally, such processes are clearly visible on building face (see Fig. 1), they result in mortar ablation, active brickwork frost destruction, the brickwork fragments fall, formation of buildings enclosing structures brickwork local emergency sections.

Figure 1 – Mortar ablation, brickwork frost destruction, the brickwork fragments fall, buildings enclosing structures brickwork local emergency sections due to the operational moisture impact

Significant atmospheric moisture effects on the brickwork of buildings enclosing structures arises in case of eaves insufficient width, drainage absence, installment of horizontal pipes underneath the drainage cunette (see Fig. 2, a). Lack of drainage and incorrect installment of the blind area around the building leads to splashing of rain water that leads to water-saturation of the brickwork of buildings enclosing structures basement part (see Fig. 2, b). Brickwork of eaves and parapet, that are not installed or partially destroyed flashing of parapet cover (see Fig. 3, a), buildings enclosing structures architectural elements without water channel at drip nose or absence thereof (see Fig. 3, b) is subjected to substantial water-saturation due to atmospheric moisture. Water-saturation of the buildings enclosing structures brickwork due to atmospheric moisture leads to acceleration of sorption processes, bricks frost destruction, mortar weathering. Excessive water-saturation of the brickwork leads to formation of biological pollution (i.e. moss, grass, bushes, trees) on buildings enclosing structures (see Fig. 3, c).

Soil moisture effects on the buildings enclosing structures brickwork as a result of destruction or lack of horizontal waterproofing, in case of banked up building sidewalks level above the level of horizontal waterproofing (see Fig. 4).

a)

b)

Figure 2 – Atmospheric moisture effects to brickwork of buildings enclosing structures:
a) mortar weathering, frost destruction of the brickwork;
b) water-saturation of the sole plate as a result of spraying water,
fall of the brickwork fragments

a)

b)

c)

Figure 3 – Atmospheric moisture effect on the buildings enclosing structures brickwork:
a) destruction of the parapet cover in the absence of flashing; b) architectural elements destruction in the absence of drip nose; c) brickwork biological pollution

Figure 4. – Soil moisture effect as a result of destruction or lack of horizontal waterproofing, in case of banked up building sidewalks level above the horizontal waterproofing level

Construction moisture in the buildings enclosing structures brickwork leads to thermal conductivity increase within first 2–3 years of operation, as well as it can lead to formation of mold on construction surface, contractible non-uniform deformation of the building. In order to reduce the influence of initial moisture effect on the buildings enclosing structures, it is necessary to provide the building heating and ventilation system normal functioning.

The main types of moisture effects on reinforced-concrete constructions are atmospheric and operational moistures. Other types of moisture effects are less extended. The nature of atmospheric and operational moisture effects on buildings reinforced-concrete enclosing structures is identical. The danger is caused by defects arising during construction as well as with poor performance of works. The main reason of reinforced-concrete constructions defect is insufficient thickness of the concrete protective layer, porosity of the reinforced-concrete element because of the large water-cement ratio, which leads to reinforcement corrosion. At the time of inside reinforcement corrosion of reinforced-concrete constructions with increase of corrosion products volume by 2–2,5 times. There is concrete protective layer emission– formation of «blowings» – that leads to intense actions of capillary moisture to concrete protective layer is been destroyed in due course time. Such damages are typical for buildings reinforced-concrete enclosing structures that contact with atmospheric and operational moisture, for example, bridges in brick walls (see Fig. 5, a), balcony plate (see Fig. 5, b) external wall panel (see Fig. 5, c). For constructions made of lightweight concrete any moisture effect is dangerous: because of increased porosity, the concrete rapidly gets soaked, that leads to increase of thermal conductivity coefficient, which leads to condensate formation and even more water saturation.

a) b) c)

Figure 5 – Destruction of protective concrete layer of reinforced-concrete lintel (a), balcony plate (b), wall panel (c)

For buildings wooden enclosing structures, the most dangerous is the atmospheric moisture effect. Thus, for loft wooden constructions, moisture effect arises due to defects in roof (atmospheric moisture) as well as incorrect design of ventilation framing scheme (condensed moisture) (see Fig. 6). In order to exclude atmospheric moisture effect, it is necessary to repair the damaged roof timely, there shall be eaves gutter and pectinal airway and roof windows designed on the loft in order to avoid formation of condensed moisture, their area should be 1/300 – 1/500 of the loft space. Typical for wooden floor construction is operational and condensed moisture effect. For normal operation of wooden floor constructions, it is necessary to make airways under the floor that should be opened during the spring, summer and autumn. It is also necessary to carry out fire retardant and antiseptic treatment of wooden constructions

Figure 6 – Damage to wooden construction covering due to atmospheric and condensed moisture effect

Conclusions

1. Atmospheric, operational and soil are the most dangerous types of moisture for buildings enclosing structures.

2. Moisture effects on the buildings enclosing structures load-bearing capacity made of wood, brick and reinforced-concrete the most.

3. Measures to ensure protection against water-saturation of building elements are constructive (eaves size, walls waterproofing, available airways), technological (protective painting, hydrophobic impregnation, compliance with regulatory requirements for walls thermal resistance) and operational (sufficient ventilation, heating, timely repairs of water supply systems and draining) nature.

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COMPREHENSIVE STUDY OF ATMOSPHERIC POLLUTION IN CITIES

Assessment of air pollution level of Poltava is performed by using statistical data of enterprises inventory, stationary sites and results of calculations by program EOL-2000. Assessment and correlation the results by new researches (bioindication, air pollution indexes) according to the standards of Ukraine and the world ones are performed. The possibility of performance the further project for an impact of technology-related air pollution on the environment and human health is proved. Implementation of GIS for monitoring the air pollution of Poltava is proposed. This system can be used as a model of such GIS for other similar cities. The process of occurrence of the heat-island affect over cities and its influence on the atmospheric component is described. The importance and efficiency of the comprehensive study of atmospheric air pollution in cities are proved.

***Keywords:** atmosphere, methods of analysis, EOL 2000, pollution index, GIS technology, heat-island effect.*

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КОМПЛЕКСНЕ ДОСЛІДЖЕННЯ ЗАБРУДНЕННЯ АТМОСФЕРИ В МІСТАХ

Виконано оцінювання рівня забруднення атмосферного повітря міста Полтави за допомогою статистичних даних інвентаризацій підприємств, стаціонарних постів спостереження та результатів розрахунків програми ЕОЛ-2000. Здійснено оцінювання й кореляцію отриманих результатів новими методами (біоіндикація, індекси забруднення атмосфери) згідно зі стандартами України та світу. Доведено можливість виконання подальшого прогнозування впливу техногенного забруднення атмосфери на навколишнє середовище й здоров'я людей. Запропоновано впровадження географічної інформаційної системи спостереження (ГІС) за забрудненням атмосферного повітря міста Полтави, модель якої може бути використана для моделювання подібних ГІС і для інших міст. Описано процес виникнення острова тепла над містами та його вплив на атмосферну складову. Доведено важливість та ефективність комплексного дослідження стану забруднення атмосферного повітря міст.

***Ключові слова:** атмосфера, методи аналізу, ЕОЛ-2000, індекси забруднення, ГІС-технології, острів тепла.*

Introduction. The topic of the atmospheric air quality of always remains relevant since the environment general condition and, most importantly, the health of the inhabitants depends on the city air state. Its importance is determined in the national policy of Ukraine and enshrined in the Law of Ukraine «On the main principles (strategy) of the state environmental policy of Ukraine for the period up to 2020» [1].

Review of the latest research sources and publications. For the formation of the atmospheric air pollution state integrated assessment theory reliability were studied and used as own scientific worked [2, 3], and the works of other scientists. Specifically, publications [4] and [5] it is the application of GIS technology in research environment and atmosphere in particular. Resource [6] and [7] provide information on bioindicating the environment state. In source [8] air quality indexes for many pollutants have been established. One of such methods combination goal in a comprehensive assessment is verifying the compliance of the obtained results to primarily acts and directives on air quality of the city, the main of which can be found in the source [9].

Definition of unsolved aspects of the problem. Despite a sufficient number of methods for studying the environment there are some unresolved issues. One of them is choosing the number of research methods that can be used to give a full picture of the air state while ensuring financial, resource and time efficiency.

Problem statement. The aim of the project is the research of air pollution level in the cities with a population of about 250 – 350 thousand people by using complex assessment and with the possibility of further forecasting of the technology-related air pollution influence on environment and human health.

Main goals are: 1) analysis of atmospheric pollution from stationary and mobile sources; 2) experimental research by bioindication methods; 3) calculation of atmospheric pollution indices; 4) development of GIS for analysis of previous data and further forecasting; 5) improving the city ventilation and the urban heat-island effect.

Basic material and results. In modern conditions, systematic monitoring of air pollution level in Poltava is carried out 24 hours a day at 4 stationary posts of surveillance «POST-2A». Sampling for contamination with harmful impurities is carried out four times a day with 10 ingredients, except for dust, soluble sulfates, and carbon monoxide. Characteristics of atmospheric air pollution in the city according to stationary posts of surveillance are given in Table 1. Some indexes of atmosphere pollution by individual contaminants in the areas of Poltava were calculated basing on the data of statistical reporting 2-TP Air and materials of inventory sources of pollutant emissions [2].

Table 1 – Characteristics of air pollution in Poltava (mg/m^3)

Impurities	MAC	Average concentration		Maximum concentration	
		2015	2016	2015	2016
Dust	0,15	0,2	1,3	0,9	1,8
Sulfur dioxide	0,05	0,004	0,08	0,019	0,05
Carbon dioxide	3,0	2	0,7	9	1,8
Nitrogen dioxide	0,04	0,035	0,98	0,19	1,25
Nitrogen oxide	0,06	0,025	0,46	0,11	0,3
Hydrofluoric acid	0,005	0,002	0,3	0,017	0,6
Hydrochloric acid	0,2	0,02	0,09	0,11	0,45
Hydrogen nitride	0,4	0,01	0,3	0,08	0,35
Formaldehyde	0,003	0,003	1,14	0,068	1,5

According to preliminary calculations of the atmospheric air pollution index by the individual pollutants for the Poltava arias, it can be proved that the atmospheric air of each district and the city as a whole is conditionally clean. Since the integrated air pollution index set by calculation is ranged from 1.3 to 2.61, which is much less than the norm, that is 5. In this case, the class of the ecological state of the atmosphere is defined as normal, this is a low level of pollution, which affects at the overall level of city air pollution not much and, as a result, does not significantly affect at the inhabitant health and their working capacity.

Within the framework of the topic, the analysis of the pollution level is carried out on the basis of pollutants dispersion calculations in the atmosphere surface layer under the program EOL-2000 [h] for 195 enterprises of the city, which represents 3,686 emission sources. The total number of identified pollutants is 196 [3]. On the basis of the calculation obtained results, the program EOL-2000 [h] compiled maps of pollutants dispersion in the surface layer of the atmosphere separately for administrative areas of the city (Fig. 1).

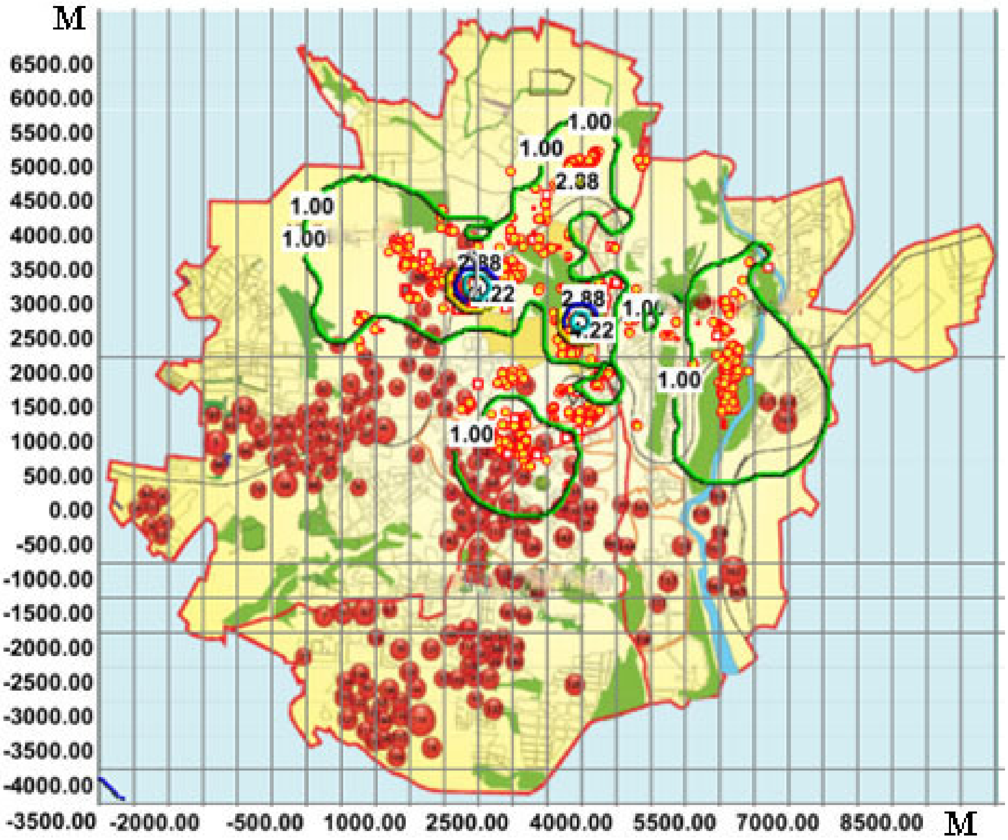

Figure 1 – Dispersion map of nitrogen oxide (IV) in Poltava

Another method of the comprehensive assessment of Poltava air state in the future is bioindication use. Environmental factors affect certain plant species very strictly [10]. Thus, there can be used the inverse pattern and considered the impact of the physical environment on plants. For example, lichenindication (determination by lichen), briodinduction (using moss) and mycoindication (using fungi) can be used to determine air pollution level.

The city sustainable development ecological dimension indexwhich is based on three categories: «Ecological systems and natural resources», «Anthropogenic pressure on the environment», «Municipal ecological management», is an important indicator of such comprehensive assessment of the city air environment. [11]. It contains 28 indicators of city sustainable development and 89 indexes.

For this research, it is planned to use two indicators: «air quality», which consists of seven parameters (quantitative indexes of characteristic pollutants), and «emissions to atmospheric air», which contains two parameters (emissions of pollutants into the atmospheric air per 1 km^2 and per person).

The protection of the atmosphere cannot be successful if it applies only measures directed against certain pollution sources. The best results can only be obtained through objective approach to determining the causes of air pollution, the contribution of individual enterprises, sources and determining the real possibilities of limiting these emissions. Therefore, the final stage of the conducted assessment is the creation of geo-information technology for the atmosphere protection. Its task is interwoven with the government information and analytical system of Ukraine emergency situations, developed by the order of the Ministry for Emergencies in Ukraine by specialists and scientists of the Distributed Information and Analytical Center of INTEK-Ukraine, the Institute of Cybernetics of the National Academy of Sciences of Ukraine, the Institute of Geochemistry of the National Academy of Sciences and the Ministry of Emergencies of Ukraine, NDC technologies of sustainable development of the Taurian National Vernadskuy University and CJSC «ECOMM Co.», which aims are to provide interagency information interaction and analytical support for decision-making on the basis of modern spatial analysis methods, simulation of emergency situations development and forecasting their consequences [12].

Thus, using the GIS toolkit, a program package (an example of their use is shown in Figure 2) allows calculating and visualizing the results of modeling the pollutants emission into the atmosphere, considering all inventory data of enterprises, with the possibility of their correction and forecasting under changing different indicators.

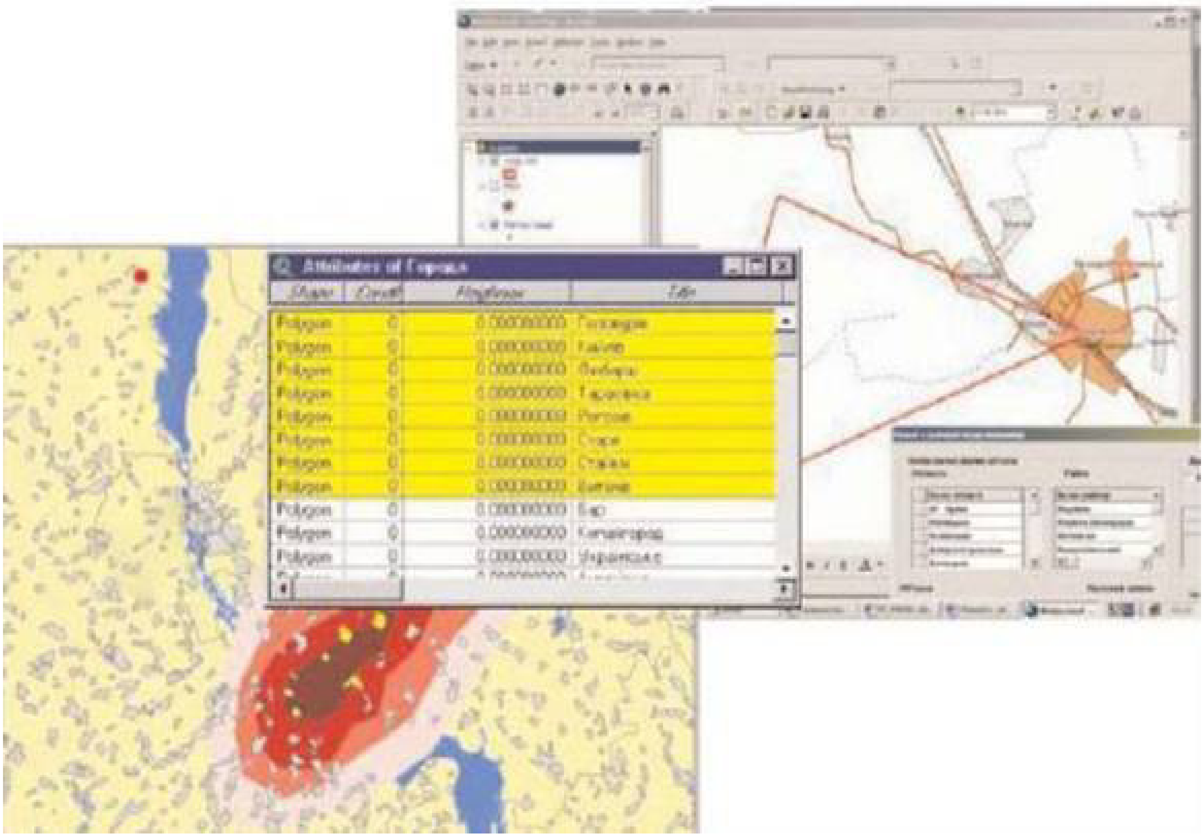

Figure 2 – Modeling results visualization of the pollutants emission in the atmosphere using the GIS toolkit

The air temperature is most strongly influenced by the territory urbanization and is one of the most measurable meteorological parameters. The temperature differs between the urbanized area and the surrounding undeveloped or poorly developed landscapes depending on a number of factors. Among them there are the city size, its territory construction density and the synoptic conditions, the weather conditions at a given time.

One of the most significant features of the urban climate is the appearance in the city of the so-called «heat-island effect», which is characterized by higher air temperatures than in the countryside (Fig. 3). This phenomenon is the result of several reasons.

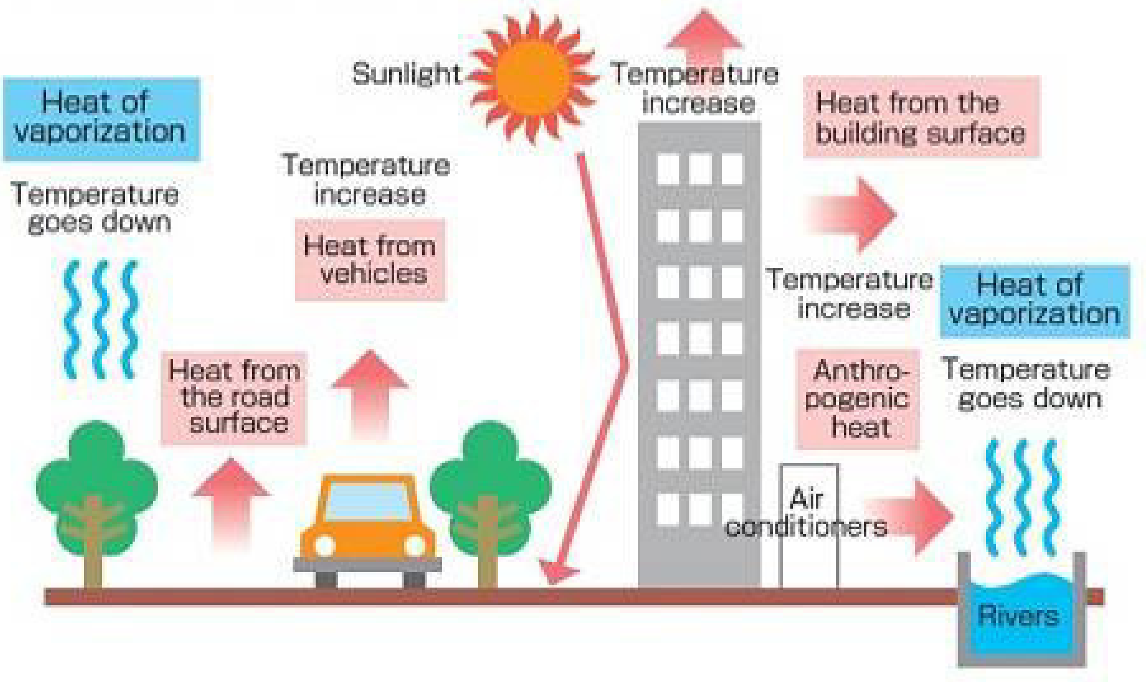

Figure 3 – How the «heat-island effect» occurs

First, in cities, the albedo of the underlying surface is reduced due to the appearance of its buildings, structures, artificial coverings. Reduction of the albedo as a result of the territory development leads to more intensive absorption of solar radiation in comparison with unoccupied areas, the accumulation of the heat absorbed by the day in the construction of buildings and structures, and its release into the atmosphere in the evening and night hours. In addition, in urbanized areas, the heat consumption for evaporation is sharply reduced by reducing areas with open soil cover and occupied by green plantations, and the rapid removal of atmospheric precipitation by rainwater systems does not allow the creation of moisture reserve in soils and surface water bodies. Urban development also leads to the formation of air stagnation zones, at low wind speeds, prevents turbulent mixing of the atmosphere surface layer and hears transfer to its overlying layers. Consequently, the building heat transfers due to the conditions deterioration of turbulent mixing in the surface layer decrease in comparison with the undeveloped territories and the heat builds up inside the building causing it to overheat.

Secondly, the formation of the «heat-island effect» in the city territory is facilitated by change in the atmosphere transparency. Various impurities from enterprises and transport coming into the atmosphere lead to a significant decrease in the total solar radiation. But, even more, they reduce the counter infrared radiation from the earth surface, which in combination with the heat transfer of buildings and industrial facilities leads to the appearance of a local greenhouse effect and the development of temperature anomalies in the city.

Most strikingly, the contrast of the city-suburbs temperature appears when it is clear and not windy and disappears when it is windy and cloudy. In the evening and at the first hours after sunset, due to the peculiarities of the heat-island effect formation, the temperature contrast is sharper than at noon, and in summer it appears better than in winter with similar synoptic situations.

The average air temperature in the big city is usually above the temperature of the surrounding areas at 1 – 2°C; however at night, with a slight wind, the temperature difference can reach 6 – 8°C. Over the centers of large cities, the «heat-island effect» rises to 100 – 150 m, and in cities of smaller sizes – 30 – 40 m (Fig. 4).

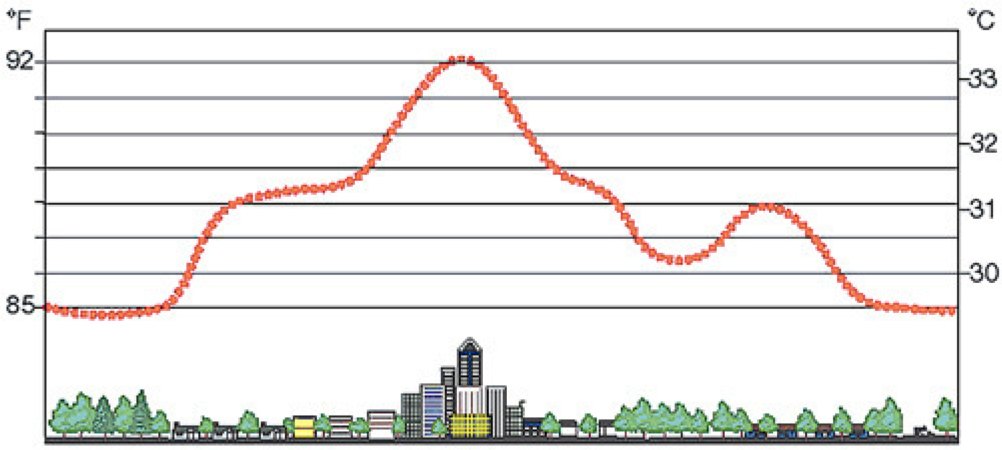

Figure 4 – Section of the «heat-island effect» over the city

The formation of a «heat-island effect» in built-up areas has a number of direct or indirect environmental and bioclimatic effects, which can have both positive and negative character.

Ecological consequences of the «heat-island effect»:

– «displacement of the city» by its climatic characteristics in the southern direction: the frost-free and snowless periods in the city territory increase, the earlier onset of the growing season;

– increase in the number of days with thaws. In the cold half-year, the transition of air temperature through 0 ° C creates problems not only for economic and road maintenance services of the city but also for the state of its natural environment components, primarily green vegetation [13].

Therefore, it is very important for the comprehensive assessment of atmospheric air to consider these factors.

Conclusions. The performed analysis and assessment the atmosphere state allow for objective forecasting, planning and further development of the atmospheric air protection program in Poltava as part of the regional target program of environmental protection, rational use of natural resources and ensuring environmental safety considering the regional priorities of Poltava region for 2017 – 2021 years. The methodology of such an integrated approach to the assessment of atmospheric air pollution with the confirmatory results of its relevance and perspective can be used by other cities not only in Ukraine but also in the practice of foreign settlements. It is especially right choice for cities with a population of about 250 – 350 thousand people.

Therefore, the choice of the most effective comprehensive analysis for the research of the atmospheric air pollution state in Poltava is relevant and promising in view of improving the emissions control level from stationary and, in the long term, pollution mobile sources and, as a result, city air quality improving.

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THE EXPERIENCE OF CIVIL ENGINEERING SPECIALISTS WITH HIGH QUALIFICATION AT BIALYSTOK UNIVERSITY OF TECHNOLOGY

This article shows the learning process of students-engineers in Poland, disciplines which were learning and the projects which were submitted. It also describes what the elements university consists of – library, main departments: architectural, civil and environmental engineering, electrical engineering, computer science, mechanical engineering, management, forestry, amount of students and teachers. The article marks studying process features and courses and subjects content. The main principles of teaching were set out. They were provided with using of the shown information variety methods. Comparison of education processes in Ukraine and in Poland is highlighted in the article.

Keywords: *Bialystok University of Technology, education, learning agreement, composite structures, concrete for special application, concrete structures, demolition, steel structures, structural mechanics, timber and masonry structures, foundations, matura.*

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ДОСВІД ПІДГОТОВКИ ФАХІВЦІВ БУДІВНИЦТВА ВИЩОЇ КВАЛІФІКАЦІЇ В БІЛОСТОЦЬКІЙ ПОЛІТЕХНІЦІ

Викладено відомості про процес навчання студентів-будівельників у Польщі: які дисципліни вивчалися та які проекти були виконані. Описано структурні підрозділи університету – бібліотека та факультети: архітектурний, будівництва та охорони навколишнього середовища, електричний, інформатики, механіки (мехатроніки), менеджменту, лісничий, кількість студентів і викладачів. Відзначено особливості навчального процесу й склад навчальних дисциплін. Наведено основні принципи викладання, різноманітні методи відображення інформації. Зазначено результати порівняння освітніх процесів в України та Польщі.

Ключові слова: *Білостоцька політехніка, узгодження навчальних дисциплін, сталезалізобетонні конструкції, бетони спеціального призначення, залізобетонні конструкції, знесення будинку, конструкції зі сталі, будівельна механіка, дерев'яні та кам'яні конструкції, основи і фундаменти, вступний іспит.*

Introduction. Education methods studying in different parts of the world and bringing global experiences to train future engineers in Ukraine is important. Poland is one of the most developed European countries, with high education and science level. Education of civil engineers there has a leading role. Poland attracts modern technologies with active resources use, which are provided by the European Union.

The purpose of this article is to demonstrate the learning process of civil engineers in Poland, to show the specific examples based on the studied subjects, to learn teaching methods in Polish universities and the education system overall. Twelve students from the Building Department of Poltava National Technical Yuri Kondratyuk University had the opportunity to study for one term in Poland, in Bialystok University of Technology. They also explored the educational process.

Main content and results. Bialystok Polytechnic has 65-years history; it has about 12 thousand students and employees, and more than 600 teachers. University has 7 faculties: architectural, civil and environmental engineering, electrical engineering, computer science, mechanical engineering, management, forestry.

To combine the educational programs between the two universities «Learning Agreement» was signed where the objects studied in Poltava National Technical Yuri Kondratyuk University, were agreed with the following courses:

– Composite structures;
– Concrete for special applications;
– Concrete structures;
– Demolition;
– Steel structures;
– Structural mechanics;
– Timber and masonry structures;
– Foundations.

As it was shown by Poltava students, education in Poland is conducted at high level. Bialystok University of Technology has modern laboratories, a large number of qualified teachers, and easy access to academic resources.

During the time of the studies in Poland, Poltava students had the opportunity to see the University library (Fig. 1), built and funded by the European Union. It is one of the best libraries in Poland, allowing students full access to information from various sources. There, students are granted tranquil atmosphere to concentrate while doing essays and to relax in a special room.

Figure 1 – Modern library located on the area of University

The course «Concrete for special applications» consisted of two parts – theoretical and practical. The theoretical part was concluded by passing an exam. The practical part had a project called «Demolition of the house» which will be described later on.

In this course laboratory work was conducted, where were tested various types of admixtures and concretes (Fig. 2).

Figure 2 – Labarotory works from subject «Concrete for special application»

The next studied subject was concrete structures. It consisted of two parts – theory and practice. The theoretical part consisted of passing the exam at the end of the course. The practical part, on other hand, had a project, which included the calculation of reinforcements numbers for the slab and beam (Fig. 3). [1]

The lectures at the course were given by the member of the European Committee for standardization, Professor Victor Tur (Belarus, Brest).

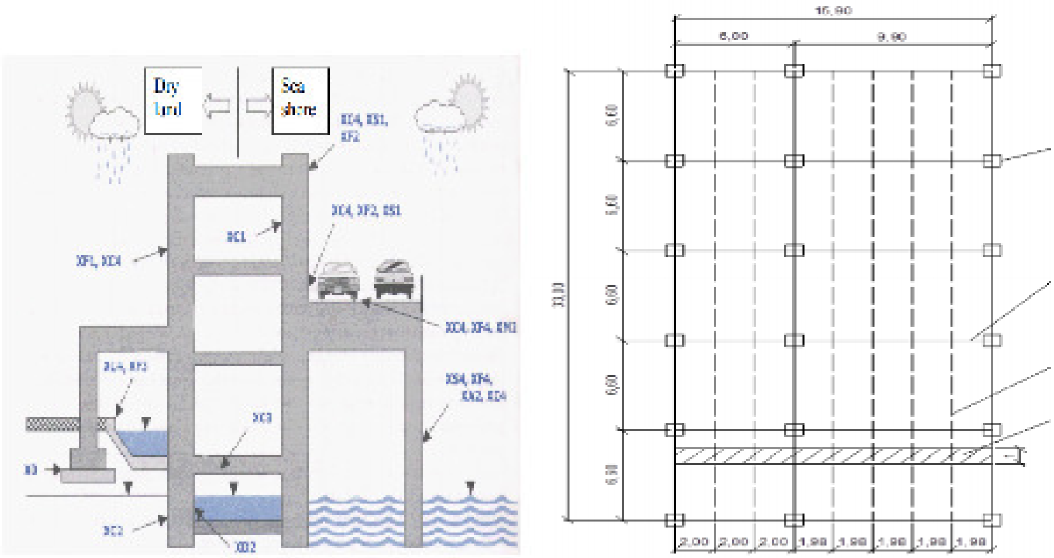


Figure 3 – Lectures and chemes for the project conducted in the course «Concrete structures»

At the course «Foundations» students designed the structures use of regulatory document - Eurocode 7. The course had only practical part which consisted of two projects – calculation of foundation based directly on the ground and calculation of the retaining wall (Fig. 4). [2]

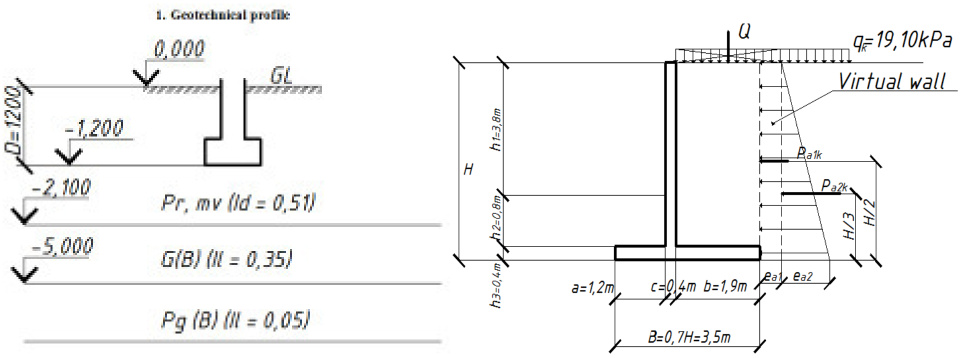


Figure 4 – Main schemes in the projects from discipline «Foundations»

During the Timber and Masonry course lectures were delivered. At the end of lectures students passed the exam, and project which consisted of two parts – the calculation of timber truss and brick walls, that perceived the load from the truss and roofing. The calculation required the use of the normative documents – Eurocode 5 and Eurocode 6 (Fig. 5). [3; 4; 5; 6]

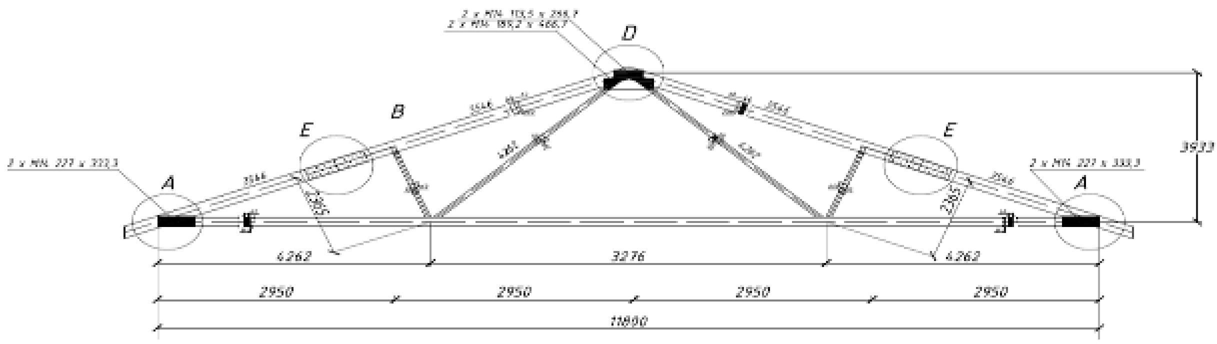


Figure 5 – Timber truss in the project «Timber and masonry structures»

At the Steel structures course students were asked to submit calculated project, which consisted of three parts – steel beam calculation, steel column calculation and calculation of connection between beam and column. The calculations were provided according to Eurocode 3 (Fig. 6,7) [7].

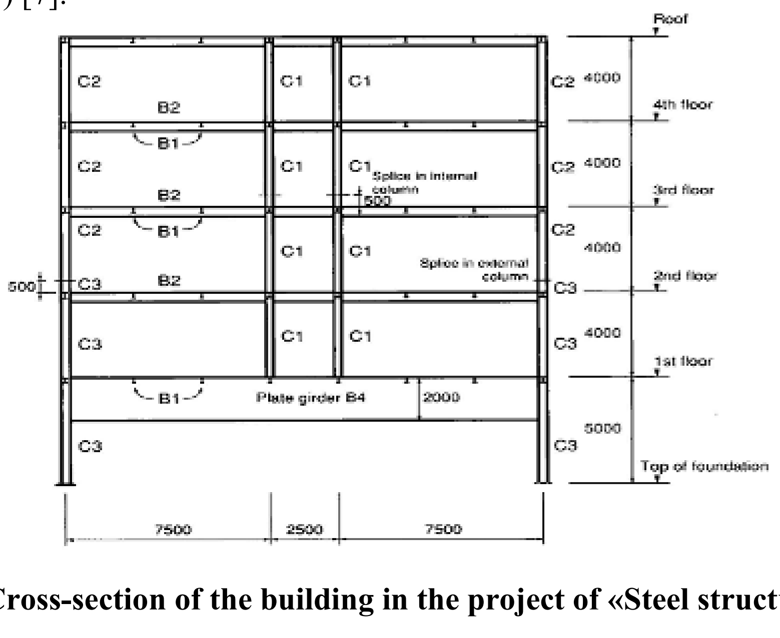


Figure 6 – Cross-section of the building in the project of «Steel structures»

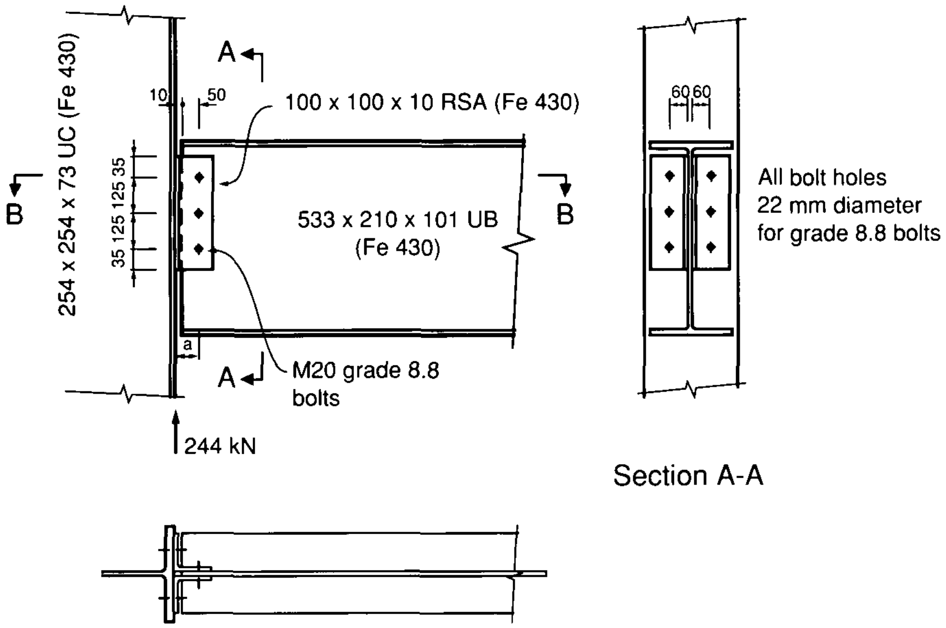


Figure7 – Typical joints in the Steel Structures project

The discipline «Structural mechanics» in Bialystok University of technology was aimed to familiarize students with calculation of systems by force method. This method was presented during the lectures. Lectures were conducted by two teachers using a variety of methods to display the information (Fig. 8).

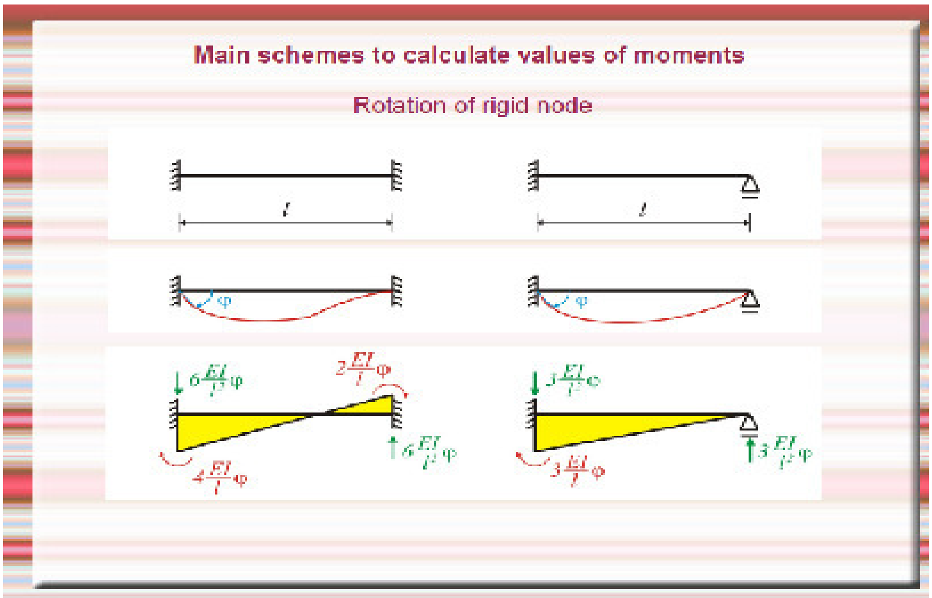
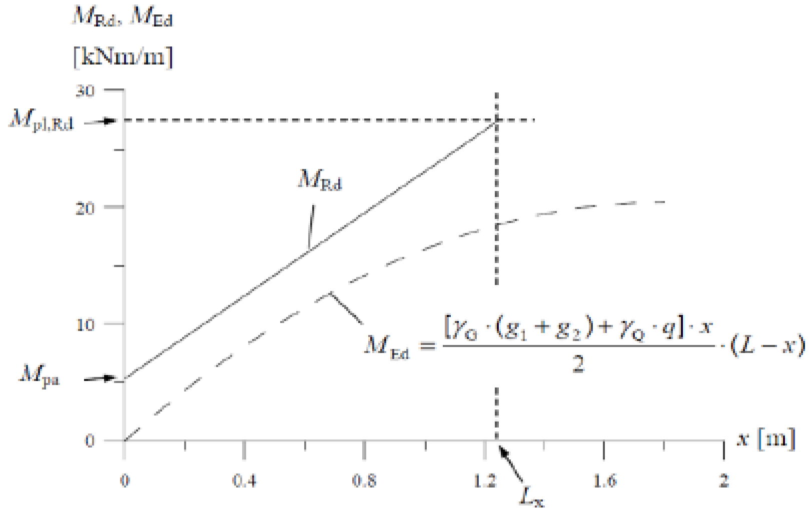


Figure 8 – Fragment of a lecture at the subject «Structural mechanics»

The subject of «Composite structures» consisted of theoretical and practical parts. For the practical part a project was submitted, which consisted of the composite slab calculation. Design consisted of slab loadings calculations and displacements. It was provided according to Eurocode 3 and Eurocode 4 (Fig. 9) [7; 8] .

a)



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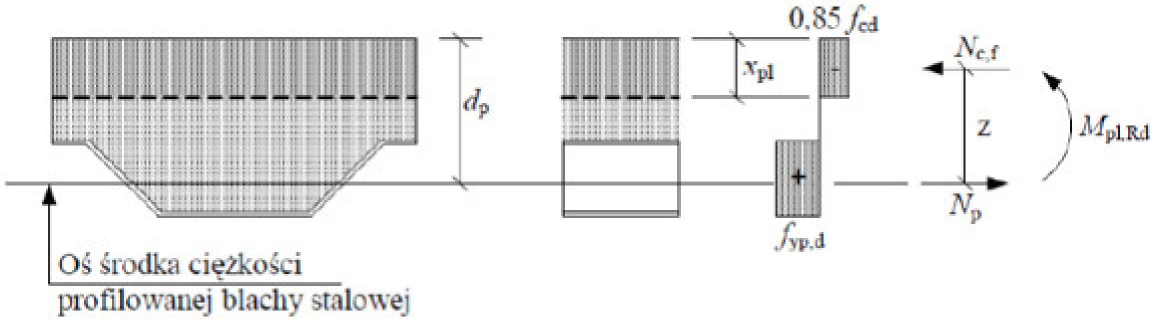

c)

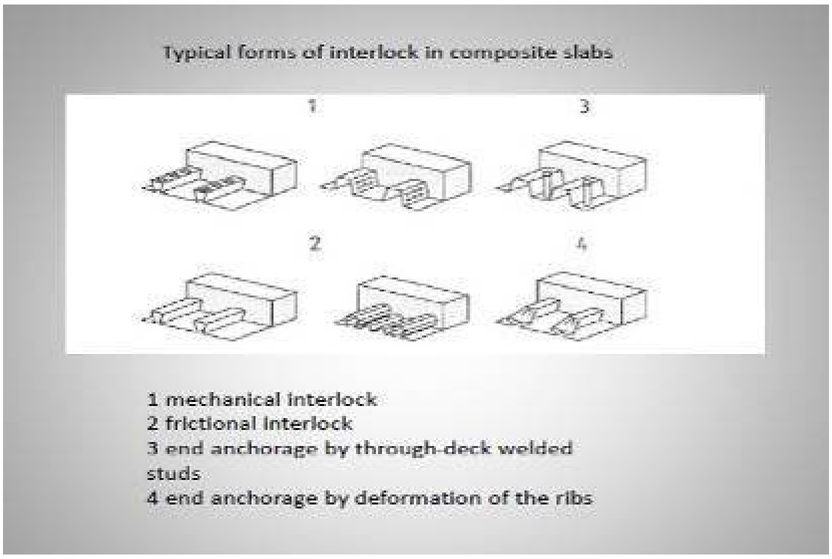

Figure 9 – Pictures in the discipline «Composite structures»:
A and B – calculation of composite slab; C – fragment of lecture.

As it was noted previously, the course of «Concrete for special applications» had a practical part, where the projects on the topic «Demolition of building»were issued. The main idea of it was to select an appropriate method of building demolition – mechanical or explosive. It was necessary to enforce safety precautions when performing the work. After that it was also necessary to make the classification and to calculate the amount of materials remaining after the demolition. In the case of the houses made of concrete it was neccesary to ensure the selection of appropriate technology for the separation of aggregate from the «cement paste». Then, after the separation, to enforce using of the wastes again in the manufacture of concrete. During the design it was appropriate to make use of Internet sources and Polish waste catalogue.

It is advisable to specify particular aspects of education in Poland and to compare them with Ukrainian ones. In reference to educational process in general, before studying at the University each student is required to pass Polish national entrance examination called «matura». This exam is held every year in May. This includes: a mandatory test of Polish language, mathematics and one foreign language and specialized subjects required for the admission (for example, in civil engineering – physics or chemistry). The second step is selection: each student who passed the «matura», needs to visit the website of University and sign up. Then the admission makes a ranking of students according to their scores. The third and the last step after the studentss list announcement is the filling of all documents. Student enrollment is conducted in three waves.

Analyzing the above, it can be concluded that the process of admission to Polish university is similar to the process in Ukraine. [9]

Throughout the term Polish full-time students must write two colloquims, or to carry out projects, and report in laboratory works. The projects of Polish students have more theoretical content, they, unlike the Ukrainian, contain a small amount of drawing material and designing directly in the classroom.

During the term each Polish student must score 30 points, this number is the same for all disciplines. To take the exam on a particular subject, a student must score at least 20 points. In the case, if the number of points is less, then, at least one subject must be recurred the entire academic year after paying to the University the appropriate amount of money.

In first year of studying Polish civil engineering students have similar to Ukrainian students disciplines such as higher mathematics, physics, chemistry, engineering, geodesy, informatics, and descriptive geometry.

Holidays in Polish Universities usually start after final exams in July, sometimes at the end of June (depends on schedule). The new academic year in Poland, unlike in Ukraine, starts in late September. If a Polish student has not passed the exams during the summer session, he/she has another chance during the first days of September. If the examination has been successfully passed, he/she keeps their scolarship if they had one. Also Polish students have a few days off during New year and Easter (Orthodox and Catholic holidays), on the eve of holidays classes are usually shortened do.

Part-time students in Polish University have almost the same education process as full-time students. There are only two differences. Students must pay for the education, the amount depends on the speciality (technical speciality usually cost more than humanitarian). Part-time students have classes only at the weekend.

In the end it is appropriate to say that learning processes in Poland and in Ukraine are similar, as they include 30 credits but differ in the way of course project implementation. In Ukraine there are 2-3 dimensional projects from the major disciplines and in Poland there are dimensional projects for each subject, which are carried out directly in the classroom.

Conclusions.

1. Studying in the Bialystok University of Technology showed what the European educational process, design standards, and implementation of projects and experiments are like.

2. Projects in Polish Universities have a significant amount of calculations and a smaller amount of drawing material.

3. Acquired knowledge in the insight and application of standards of the Eurocode in the future can be used for new projects designing and master`s degree diploma in Poltava National Technical Yuri Kondratyuk University.

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ANNIVERSARY CELEBRANT

OLEKSANDR SEMKO

60 years ago, on May 15, 1958, Olexandr Semko was born in Kharkov. Doctor of Technical Sciences (DSc) on specialty 05.23.01 «Building structures, buildings and constructions»; Professor of metal, wood and plastic structures department, senior researcher; the head of architecture and urban construction department in Poltava National Technical Yuri Kondratyuk University since 2007; member of Construction Academy of Ukraine. He was awarded the State Prize of Ukraine in Science and Technology. Hero of an anniversary participated in the development of curriculum for architects and builders, module creation in the fields of energy saving, thermo modernization, building physics, building structures, including the method of transition to calculation and design by Eurocodes. Over 30 years of production, scientific and educational activities, he has been involved in solving a number of scientific and practical problems of probabilistic design of existent and projectable building structures.

Oleksandr Semko is a graduate of the Poltava Engineering and Construction Institute Faculty of Industrial and Civil Construction (the former name of PoltNTU) in 1980. The head of the Oleksandr`s diploma work was Professor Sergii Pichugin (a young assistant professor in those years). Thanks to his training Oleksandr Semko received the necessary experience and knowledge in solving both scientific and practical problems. After completing his military service Oleksandr Semko returns to his native university where, under the supervision of Professor Andriy Pavlikov, successfully defends his PhD thesis and for three years he works in a laboratory of building constructions survey. Professor Leonid Storozhenko notices a proficient young scientist and invites Oleksandr to teach at the metal, wood and plastic structures department. Since this time O. Semko has begun to study steel-reinforced concrete structures under the supervision of professor Storozhenko. Many scientific and practical developments are the result of this cooperation; it allows obtaining a doctoral degree in technical sciences in 2006.

Professor Semko has significant experience in the field of construction and a wide range of interests. Under his leadership, 15 PhD and 1 doctoral degree thesis were prepared and successfully defended.

But it is not possible to work effectively without interesting and meaningful rest. Professor Semko can be called a bibliophile and it is hard to imagine him without books. As Oleksandr Semko grew up in Kremenchuk on the banks of the Dnipro River, another hobby is fishing. He catches fish whenever it is possible: during conferences – in the Polesye lakes or the Black Sea, during the holidays – in the waters of the Red, the Mediterranean Sea or the Adriatic Sea, during the weekend – in the reservoirs of his native Poltava region.

Oleksandr Semko is a successor of the designing, installation and inspecting buildings and structures dynasty for over 70 years. The Editorial Board of the Journal wishes anniversary celebrant good Cossack health, new scientific achievements and goodness and harmony in the family.

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