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SCIENTIFIC SCHOOL «RELIABILITY OF BUILDING STRUCTURES»: NEW RESULTS AND PRESPECTIVES

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The article presents the results obtained by scientific school «Reliability of building structures» for the spice of five years 2015 – 2019. Some aspects of the general approach to structural reliability assessment have been developed. Current normative calculations of structures were combined with the assessment of their reliability. Engineering methods were developed that allow to take into account an increase snow loading on coverage of building of variable height, to estimate the processes of snow thawing on roofs of the heated buildings and snow laying on cold roofs. It were investigated the specific wind load in the mountainous Carpathian region and loads of travelling cranes of different producers. Reliability of steel structures of trunk pipelines and reinforced concrete beams with carbon-fiber reinforcement has been evaluated. Work features of sheet steel structures were investigated. Construction accidents have been analyzed.

Keywords: construction, building, structure, reliability, probabilistic method, load.

НАУКОВА ШКОЛА «НАДІЙНІСТЬ БУДІВЕЛЬНИХ КОНСТРУКЦІЙ»: НОВІ РЕЗУЛЬТАТИ І ПЕРСПЕКТИВИ

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Висвітлено наукові результати, одержані науковою школою «Надійність будівельних конструкцій» за 2015 – 2019 рр. Розвинуто деякі аспекти загального підходу до оцінювання надійності конструкцій. Опубліковано монографію, орієнтовану на практичні розрахунки надійності конструкцій. Діючі нормативні розрахунки конструкцій поєднано з оцінюванням їхньої надійності. Розроблено метод визначення ймовірності теплової відмови огороджувальних конструкцій за трьома основними теплотехнічними показниками. Сформовано інженерні методи, які дозволяють ураховувати підвищення снігового навантаження на покриття будівель з перепадами висот, оцінювати процеси танення снігу на покрівлях опалюваних будівель та відкладення снігу на холодних покрівлях. Досліджено специфічне вітрове навантаження у гірському Карпатському регіоні та навантаження від мостових кранів різних виробників. Розгорнуто огляд, присвячений важливому питанню сполучення навантажень. Розроблено практичну методику ймовірнісного розрахунку надійності конструкції лінійної частини сталевого магістрального трубопроводу. Розв’язано задачу оцінювання надійності трубопроводу на етапах проектування, спорудження, експлуатації та капітального ремонту – враховувалися кільцеві і поздовжні напруження у трубопроводі, на які впливають деформації випадково-неоднорідної основи, випадкові внутрішній робочий тиск та температурні перепади. Визначено ймовірність відмови трубопроводу для різноманітних умов прокладання. Оцінено надійність залізобетонних балок з вуглепластиковим підсиленням. Проведено порівняльний аналіз конструктивних рішень та досліджено особливості роботи листових сталевих конструкцій. Проаналізовано аварії у будівництві. Показано можливі перспективи подальшого розвитку складових проблеми надійності будівельних конструкцій: імовірнісного опису випадкових навантажень, визначення надійності конструктивних елементів, оцінювання безвідмовності складних статично невизначених систем.

Ключові слова: будівництво, будівля, конструкція, надійність, імовірнісний метод, навантаження.

Introduction

Reliability is an important parameter of any technique. Its content is briefly formulated as «quality in time». Therefore, the problem of ensuring reliability in technology is urgent in today's environment. This problem has some specifics about building objects. Buildings and structures have a long service life. This makes it difficult to predict their reliable operation. Buildings and structures are not numerous or even unique objects. So, they are not suitable for conventional methods of mathematical statistics of mass phenomena. In addition, building loads have a complex random nature. Buildings themselves are complex systems of many elements. Therefore, some issues of building reliability assessment remain unresolved, and further research into the reliability of construction sites is a pressing issue.

Review of the research sources and publications

Since the 1970s, research into the reliability of buildings and structures has been actively conducted at the National University «Yuri Kondratyuk Poltava Polytechnic» (formerly PoltNTU). Subsequently, this scientific area was organized as one of the scientific schools of the University. In the years that have passed, this scientific school has trained six doctors of science and more than twenty PhDs. The school members wrote dozens of scientific monographs and hundreds of scientific articles, and received many patents for inventions. The results of the research of the scientific school have been reported at many national and international scientific conferences. The Scientific School «Reliability of Building Structures» has high authority in Ukraine and abroad.

The interim summaries of the scientific school's work were published in a collective monograph published in 2010 [1]. Further achievements of the university on the problem of reliability were covered in the thematic issue of the scientific collection of PoltNTU, which was published in 2015 [2]. A general method of the reliability calculating of building structures by the criterion of bearing capacity was developed. This method involves solving the following problems: probabilistic description of loads, mechanical characteristics of materials and joints and other random parameters; development of issues of casual load combinations; assessment of the reliability of structural elements, taking into account the models of their failure; reliability calculation of structural systems, in particular, complex units and redundant systems (RS), taking into account the possible nature of their destruction; quantitative assessment of the reliability of various buildings and structures.

Definition of unsolved aspects of the problem

The overall level of theoretical results of studies of the building structures reliability is quite high to date. However, the state of development of reliability issues among practitioners remains insufficient. Meanwhile, current codes require them to perform appropriate calculations and estimates. Therefore, we need books and guides with available examples of numerical reliability calculations for different designs. Another drawback is the gap between the reliability calculations and the current method of calculating structures by limit states. We need competent explanations regarding the compatibility of the basic principles of both methods and the probabilistic nature of the parameters of the limit-state technique. The presence and influence on the reliability of the designs of individual high loads with low probability of implementation remains beyond the scope of research. The probabilistic description of atmospheric loads lacks statistics in recent years and climatic characteristics of the mountainous regions of Ukraine. The new capabilities of modern computational complexes for statistical modeling of random loads, in particular, crane loads, are insufficiently exploited.

Problem statement is to highlight the scientific results obtained by the School of Structural Reliability for the five-year period 2015 – 2019 in the above mentioned directions, together with possible prospects for further development of research on the reliability of structural structures.

Basic material and results

General reliability approach. A voluminous article is published in the scientific journal, which is published in the NDB SKOPUS [3]. The article describes the probabilistic method of the structures reliability estimating. It takes into account the random nature of the loads and the strength of the steel, the compatible action of the loads, the specific nature of the operation and failure of the steel elements, units and steel structural systems as a whole. Based on the developed method, numerical reliability calculations were performed on a wide range of structures such as crane beams, trusses, truss beams, columns and frames. As a result, the coefficients of design codes and the achieved economic effect were substantiated.

The monograph [4] describes the obtained scientific results in more detail. The main difference between this monograph is the orientation to practical calculations of the structures reliability. The volume of such calculations is significantly increased. The text of the book provides more than 40 numerical examples. Theoretical calculations are subordinate in nature and are presented in a reduced volume. They provide the necessary justification and explanation for the calculation examples. In this monograph, the description of probabilistic models used in reliability calculations is substantially supplemented, the provisions of national and foreign normative documents on construction objects reliability are given, it is expanded the range of structural elements for which reliability estimates are obtained. Orientation to practical reliability calculations is associated with a thorough and objective assessment of reliability and the mandatory obtaining of numerical values of reliability indicators, such as failure and uptime probabilities, failure rates, and more. In addition, national codes on the construction sites reliability, introduced in recent years, regulate the quantitative standards of reliability (failure probability, safety characteristics) to be determined in the design. Meanwhile, there is a lack of practical guidance in the technical literature, and the book [4] aims to remedy this deficiency to some extent.

During the years mentioned in this article, cooperation with our long-time friend and colleague, DSc., prof. A.V. Perelmuter, a leading employee of SCAD Soft Ltd. (Kyiv), a recognized authority in the field of reliability and computer design calculations. One of the consistent results of our research was the suggestion of a new temporal feature called «mobilization». It shows how much the system is capable of responding to local time (impulse) unexpected perturbation. As a measure of mobilization, it is proposed to take the value obtained by multiplying the reliability index (the number of standards that separate the load mean value from the calculated value) the ratio of the time interval between the emissions of the loading process for the level of the calculated value and the estimated lifetime of the structure. The values of this characteristic are significantly different for the most common loads (snow, wind, crane and their combinations), which are classified by design standards to one class of temporary loads. The lack of mobilization of the structure should lead to increased attention and the use of protective measures, such as heightened control or design decisions. These results have been published in Ukraine [5] and abroad [6, 7].

Current normative calculations of structures were combined with the assessment of their reliability in the monograph [8]. It thoroughly substantiates the main provisions of the method of building structures calculation by limit states. The history, advantages, disadvantages and components of the method, criteria of limit states are covered. The loads acting on the structure and the physical and mechanical characteristics of the structural materials are considered in detail. The nature of work and possible destruction of structures under load are analyzed. Probabilistic analysis of the structures reliability is performed, the prospects for the development of the limit-state method are outlined. Data from domestic and foreign design documents are presented and compared. The monograph is a textbook for students of construction specialty and it is also recommended for graduate students, researchers and engineering staff.

Most reliability calculations use a failure model based on loss of bearing capacity or rigidity. In contrast, a method for determining the probability of thermal failure of envelopes by three main thermo-technical indicators was developed [9, 10]. Based on minimization of economic losses, the method of normalization of required failure probability level of envelopes by the reduced thermal resistance criterion was proposed.

Research at the scientific school under consideration has traditionally included the following components: development of probabilistic description of loads, evaluation of the reliability of structures and their elements, determination of the reliability of structural systems, implementation of the obtained results. In all the above areas, significant scientific results have been obtained in recent years.

Probabilistic description of loads. The study of loads has always been one of the priority areas of the scientific school «Reliability of building structures». The results of a multi-year study of snow load are outlined in a collective article published in Poland in 2017 [11]. The probabilistic model of the snow accumulation on the roofs with height discontinuity was worked out, the decreasing coefficient for the snow load weight was received and the ways of their application in the designing were elaborated. The proposed approach allows to differentiate the coefficient of roof exploitation conditions C_e depending on the roof thermal resistance, and in some cases – significantly to reduce the design snow load values and to give substantial saving of steel. The probabilistic model for impulse stochastic process of snowfall sequence has been developed. According to the data from 132 meteorological stations in Ukraine its statistical characteristics have been found: average annual snowfall amount and exponential distribution of values of one snowfall. The law of intensity distribution of snow melting has been determined experimentally. The territorial zoning map of Ukraine by characteristic values of the snow load on the roofs that emanate heat was developed.

Friendly relations bring together scientists of the scientific school with researchers of reliability and loads from Lviv and Uzhgorod. One of the joint work with them was the study of specific wind load in the mountainous Carpathian region [12]. It was found that wind parameters at the peaks of the Carpathians with absolute marks above 1330 m (m/s Play) have not been investigated. The analysis of observations on climatic parameters in 1955–2005 at 9 meteorological stations was carried out using 4 directions between the initial meteorological stations: Beregovo – 113 m, Uzhgorod – 114,6 m, V. Berezniy – 209 m and Hoverla Station – 2061 m. Formulas of altitude coefficients allowed to calculate the values of summer and winter characteristic wind pressures for the 9 peaks of the Carpathians. On this basis a detailed wind zoning of the mountain district of Transcarpathian region is proposed.

The study of crane loads is a «firm» direction of the Poltava school of reliability, which received important results included in the normative documents. Studies of this problem continued.The comparative analysis of travelling cranes of national and foreign producers was performed [13]. The travelling cranes of concern Demag were taken for consideration among the foreign cranes. The operation conditions of cranes were analyzed according to different codes of practice. The geometrical parameters, load and weight characteristics of overhead cranes were also compared. For the calculation the horizontal and vertical loads of cranes the most unfavorable schemes of location of bridge cranes on the structures of a production building were defined. According to these schemes, the maximum loads on the frame of the building were calculated and the maximum efforts in the crane beams were determined. Using obtained internal efforts the cross sections of crane girders with a span of 6 and 12 m were calculated. The results of the comparison showed the advantages of the modern cranes in materials saving of steel structures of industrial buildings.

Due to the great difficulty of experimental investigations of the crane loads, which have the complicated statistical nature, it is possible to consider analytically such special features of these loads as a trolley approximation, zero crane operation zones and crane impact zones [14]. Nowadays statistical computer modeling can take into account a greater number of variable parameters (position of the cranes in the span, position of the crab on the bridge crane, crane position relative to each other). Comparison of polygons of vertical load on the crane wheel with the analytical results showed that design value of load, calculated according codes DBN realizes very rarely. The further numerical modeling of loads arises an interest for obtaining a more precise definition of the load factors and efforts in the structures of industrial buildings.

An important issue of load combination is devoted to a detailed review (44 pages, 140 bibliographic sources) [15]. The history of formation and development of approaches to a choice of settlement combinations of loadings for check of reliability of building designs is considered. It is underlined parallelism of researches of domestic and foreign scientists. Data about rationing of this problem and about the works which formed the basis of design codes are cited. It is noted that the problem of load combination is not fully solved and requires further research, in particular, in the following areas: development of correlated coupling of random loads; taking into account combinations of actual random load distributions; a detailed description of the specific features of the loads, closely related to the technology of production and features of structures operation - crane, technological, useful, etc.; taking into account logical relationships between download options; solution of the problem of combination or the limit states of the second group taking into account the operational values of the loads.*Reliability assessment of trunk pipeline designs*. Particular attention was paid to these structures due to the fact that Ukraine is one of the main hydrocarbon transiters to the countries of Eastern and Western Europe. The significant volumes of transit of oil, gas and their derivatives through the territory of Ukraine determine its strategic position in the energy security of Europe. For this purpose a powerful system of gas and oil pipelines of Ukraine is used, which should have high reliability. The conducted researches were devoted to this problem, namely to development of probabilistic methods of reliability estimation of linear parts of main pipelines. The choice of linear parts as the object of study is due to their considerable length, significant effects of climatic and soil conditions and the high risk of accidents with severe environmental consequences. The scientific results obtained on this subject formed the basis of two theses [16, 17], a series of articles [18] and two monographs [19, 20].

The developed methods of probabilistic estimation of reliability indicators take into account: pipeline construction; diameter and thickness of pipe walls; statistical characteristics of steel strength; load from internal pressure; characteristics of soils in which the underground pipeline is laid; temperature effects; real corrosion damage to pipe walls. The listed impact factors were presented in the form of random variables and random processes. This opened up the possibility of developing methods for estimating the probability of failure of pipelines under different combinations of loads and impacts, in different soil conditions and design situations. The performed calculations of the real pipelines made it possible for the first time to estimate their level of reliability and to analyze the influence of the calculation factors on the failure probability of the linear parts. The effect of corrosion damage on the level of reliability of the linear parts of pipelines is also taken into account. The issues of planning of repair of pipelines with corrosion damages are developed. As a whole, the complex of problems related to the reliable operation of the linear parts of the pipelines in real climatic and geological conditions, taking into account the initial imperfections and corrosion damages, has been solved.

Reliability estimation of reinforced concrete beams with carbon-plastic external strengthening. Strengthening of building structures is an important scientific and technical problem, actuality of that grows presently. Feature of this building branch during the last years is appearance, research and active introduction in practice of structures strengthening of new modern materials being high strengthening and operating characteristics. Ones of such materials are composite materials on the basis of carbon fibre (FAP). Their application in the system of external reinforcement can exemplify by composite materials of row of trade marks, intended for the increase of durability and of longevity reinforced concrete, concrete, brick and stone structures. For the receipt of reliability estimations we used the worked out reception with the substitution of probabilistic parameters in the deterministic decisions of durability of reinforced concrete beams. Taken into account thus, that most random arguments of reserve of bearing strength of reinforced concrete beams can be reasonably described by a normal law (by distribution of Gausse), in particular, durability of concrete, armature, carbon-plastic, and also row of loadings (permanent, technological, crane etc.). The reinforced concrete beams of rectangular section were considered with the single stretched armature with strengthening of carbon-plastic. The reliability estimation of reinforced concrete beams with carbon-plastic external strengthening was worked out. It was shown, what strengthening promotes reliability of beams considerably [3].

Investigation of features of work of sheet steel structures. In collaboration with National Aviation University staff, design and construction organizations, research into the operation and reliability of these specific structures has been initiated. The paper [21] deals with summarizing research concerning analytical technique of calculation of vertical cylindrical capacities for grain storage under the influence of unsymmetrical wind influences. In particular, the analysis of vertical stiffeners' work was conducted and illustrated the body deformation for high and low capacities, depending on the number of rigid elements. Considering the character of the wind load's influence, there were also made propositions considering defining normal and tangential strains in cross-section, longitudinal directions and also radial, circular and longitudinal displacements, accordingly to the general theory, was made. In addition, capacities with different edges' fortification are considered, for example, strengthened by absolutely rigid circle, free or fixed by elastic circle. General assessment of the total deflected mode of the capacity's construction was made, depending on the number of decomposition coefficients, which were put in calculation. Conclusions are accompanied with graphic isometric understandings, which were obtained on the basis of practical calculation.

Next paper [22] deals with the using of high strength steel for constructions of vertical silo capacities. It was analyzed the most widespread trademarks of steel that were used for manufacturing of vertical cylindrical silos for grain storage. It were experimentally tested mechanical and chemical features of series of examples of high strength steel of European and American manufacturers, which were used for making of the body's sheet panels. The checking calculation of storage capacity with the diameter of 11 m from shaped corrugated sheet of different thickness was made. It was shown critical factors of the body sheets, bolted joints and vertical stiffeners of the silos. It was mentioned reasonable conclusions according to the using of the concerned material for getting economically rational project decisions.

It was analyzed the structural solutions of steel silos for bulk material [23]. The design elements of the silo and the silo with cone and flat bottom were described. The general characteristics of metal silos for bulk materials, their classification by the type of housing structure and the advantages and disadvantages were given. The possibility of storage of bulk materials in cylindrical shells was analyzed depending on the type of construction. The history of occurrence of structures of spiral-fold silos was considered. The set of equipment for the construction of the housing of the spiral-fold silos was given, the step-by-step process of formation of the folding lock and features of the installation process were presented. The analysis of the structure was made and the advantages and disadvantages of spiral-fold silos were determined.

Structures such as tanks, silos, bins are very responsible structures. Their failures lead to great economic and environmental damage. Therefore, it is important to analyze the accidents of such structures. The work [24] presents the statistics of accidents of steel vertical tanks with a detailed review of the established causes and consequences of its occurrence. Information on accidents was collected on the basis of Internet sources, literature on this issue, as well as scientific publications of previous years. It also were investigated the reasons for the emergence of the need to study this issue, developed statistics materials for past years. On the basis of the collected and studied material, a chart of the percentage of the accident type at the high-security facility from 2009 – 2019 has been created. As a result of the work done, the most common types of accidents for steel vertical tanks were identified and appropriate conclusions were made.

The reliability of redundant systems. One of the significant achievements of the Poltava School of Reliability is the solution of the problem of reliability of statically indeterminate (redundant) systems [1 – 4]. The redundant structure failure estimation is a very complicated problem as depends upon the system complexity. Redundant structure failures occur after some member failures in the form of transmission to different workable states. These states match different designing schemes with various probabilistic parameters. The important thing is that determination of the elements significance in the reliability of redundant frames [25]. For this the probabilistic method of limit equilibrium is applied. All possible mechanisms of structural failure are considered. The influence of each section on the work of the frame as a whole is taken into account. Stochastic strength and load characteristics are used in the calculations. The graphs of specific contributions of individual sections and the most probable mechanisms of destruction are presented. The task was to align the impact of the frame sections without reducing the specified reliability, but it is possible to obtain a design with the same specific contributions, which is most economically justified. Specific contributions are increased or decreased as necessary to obtain a design with equal probability of failure. The method provides an opportunity to obtain optimal designs and the use of modern software systems for static calculation. Recommendations for the design of these structures have been developed. It is proposed to use the reliability coefficient of redundant steel structures.

*Analyze of accidents in construction.*The problem of buildings and structures crashes remains relevant in the current environment. Cases of destruction of construction sites with significant economic losses and human casualties make the issue more detailed. That is why the School of Building Reliability has studied building accidents in recent years [26]. On the basis of collected and processed information the accidents classification in the building was presented, depending on the building erection stage. The article gives a detailed description and analysis of such cases in the construction. The collection of information on accidents was carried out using various information sources: Internet resources, literary sources, scientific works and information from world-wide journalistic services. The findings are presented over the last ten years and cover worldwide construction incidents. The material is systematized and presented in the table form. The study results are the created accidents classification with the probability of their occurrence, which can be used later in the design of buildings and structures, in order to predict the various types of accidents in the construction.

Next paper covers the consequences of accidents at buildings and structures during the construction and acceptance in operation [27]. Attention is focused on the frequency of repetition of this type of accidents, using the graph of the structures' operation stages. The work describes and analyzes such incidents related to construction. The corresponding diagrams are constructed. The result of the study is the relevant conclusions about the typification of accidents during construction and their regularities. The most common causes of accidents are identified, which allow to obtain more detailed study of the problem and further provide for cases of such accidents at the construction site. Attention is focused on the dependence of the construction's quality on the level of the country's welfare. In addition, the conclusions contain the main tasks in the solution of this problem and ways of their implementation.

Implementation in design standards. Research findings of our University in the field of reliability have been highly praised and implemented in a number of state regulations. This is the highest level of implementation, as these standards are used by all designers throughout Ukraine. One of the normative documents developed in recent years is DSTU B B.2.6–210:2016 «Assessment of the technical condition of used steel structures» [28]. Several of our proposals are included in this national standard, including recommendations for testing the load-bearing capacity of beams with damage in the form of horizontal distortion, classification of technical conditions of structures and editorial amendments. Over the last five years, standards have slowed down: a few years ago, we have been preparing with our participation new revisions to the standards for load and reliability. I, together with. DSc. A.V. Perelmuter even participated in a discussion with DSc., prof. A.I. Lantuh-Lyashchenko (National Transport University) [29]. He argued that the existing DBN «General Principles for the Reliability of Building Structures» had serious drawbacks and had to be radically corrected. We opposed this proposal, because these rules are the only ones in the territory of the former USSR, they have stood the test of time for ten years, regulate a complex system for ensuring the reliability and safety of construction objects and require only partial amendments and references to new regulations.

Application of scientific principles in the humanitarian sphere. The preparation of the publications, partially mentioned in the list of literature, allowed to gain some experience in writing scientific books. This experience was successfully applied when writing a monograph of another humanitarian profile [30]. This book is dedicated to the famous Poltava artist Viktor Baturin (1937 – 1993). A prominent exposition artist, Honored Artist of Ukraine, he was the head of unique exhibitions creating of the Poltava Museums Ivan Kotlyarevsky, Panas Mirny, Mykola Gogol, Anton Makarenko, the Kiev Theater Museum, the Ochakov Military History Museum. For some time he was also the chief artist of Poltava, he developed the coat of arms of the city, created a monument to the fallen Cossacks and designed the Ukraine Pavilion at the World's Fair in Stuttgart (Germany). The author of the article was the author of the book. Working on the book required the combined efforts of museum workers, artists, people who knew Victor Baturin. By his 80th birthday, the book had been published, presented in a museum I.P. Kotlyarevsky and free distributed. Subsequently, the author of the article made presentations of the book in Poltava museums and libraries, spoke at the scientific conferences PNPU, dedicated to the memory of the outstanding teacher A.S. Makarenko, published an article in the scientific professional collection of PNPU.

Conclusions

Summarizing the work of the School of Structural Reliability for the five years 2015 – 2019, it should be noted that significant scientific results were obtained in this short period of time. They contributed to the defense of doctoral and PhD theses, the publication of five scientific monographs and 56 scientific articles, the publication of six textbooks, the participation in scientific conferences in the United Kingdom (London), Finland (Cottbus), Poland (Krakow, Green Gora), Azerbaijan (Baku), Kazakhstan (Taraz), as well as in Ukraine (Kiev, Odessa, Kharkiv, Dnipro, Rivne, Kropyvnytskyi), have acted as an official opponent for two doctoral and two PhD theses. At the same time, the questions for further prospective research, mentioned above in the article, remain unresolved.

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VIBRATING TABLES WITH THE SPATIAL OSCILLATIONS OF THE MOVING FRAME TECHNOLOGICAL PROPERTIES FOR FORMING REINFORCED CONCRETE PRODUCTS

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A brief description of vibration tables with spatial oscillations of a moving frame and comparison of their technological possibilities depending on their constructive execution are given. The methodology for evaluating the technological and structural parameters of the vibration tables is also presented based on the use of a dimensionless complex parameter that enables to evaluate the consumer qualities of the vibrating tables, their technical level and competitiveness during the design and operation stage. The vibrating machines constructive designs schemes, their characteristics and the description of the scope of application are presented, which enables designers and manufacturers to orientate themselves in the variety of currently used vibration machines with spatial oscillations of the moving frame for the formation of the same type of reinforced concrete products.

Keywords: vibration table, vibration exciter, forming concrete reinforced products

ТЕХНОЛОГІЧНІСТЬ ВІБРАЦІЙНИХ ПЛОЩАДОК З ПРОСТОРОВИМ РУХОМ РУХОМОЇ РАМИ ДЛЯ ФОРМУВАННЯ ЗАЛІЗОБЕТОННИХ ВИРОБІВ

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Наведено короткий опис вібраційних площадок з просторовими коливаннями рухомої рами та порівняння їх технологічних можливостей залежно від їх конструктивного виконання. На виробництві збірного залізобетону для ущільнення бетонних сумішей використовують віброплощадки різноманітних конструкцій, які значною мірою визначають технологічну ефективність обладнання і продуктивність підприємства. Широкого розповсюджені набули розроблені у різні роки КБ «Вібротехніка» ПолтНТУ вібраційні площадки з просторовим рухом робочого органа, які мають можливість якісного та продуктивного формування залізобетонних виробів усієї номенклатури. Існуючий дефіцит вібраційних машин і невизначеність у виборі пріоритетних напрямів їх розвитку створюють на виробництві значні труднощі щодо проектування чи технічного переоснащення формувальних постів підприємств збірного залізобетону. При проектуванні формувальних постів та виборі технологічного обладнання для ущільнення бетонної суміші при виготовленні залізобетонних виробів необхідна оцінка технологічних можливостей усіх різновидів віброплощадок з просторовими коливаннями рухомої рами. Також наведено їх короткий опис та порівняння технологіічних можливостей залежно від конструктивного виконання. Наведено методику оцінки технологічних і конструктивних параметрів віброплощадок, що ґрунтується на використанні безрозмірного комплексного параметра. Вона дозволяє як на стадії проектування, так і в процесі експлуатації оцінити споживчі якості віброплощадок, їхній технічний рівень і конкурентоспроможність. Аналіз наведених схем конструктивних виконань вібраційних машин, їх характеристик та опис сфери застосування дозволять проектувальникам та виробничникам орієнтуватися в різноманітті використовуваних сьогодні вібраційних машин для формування однотипних залізобетонних виробів.

Ключові слова: вібраційна площадка, вібробуджувач, формування залізобетонних виробів.

Introduction

The development of rural, including farms, involves intensive construction of housing and farm buildings that are cheap in price and have high quality. In addition, natural disasters that have occurred in Ukraine in recent years. have identified the need for the development of technology of high quality and fast construction in difficult conditions. Emergency situations require prompt and immediate construction in unsuitable conditions for its conduction. Therefore, in modern construction, concrete products are still in demand. At the production of prefabricated reinforced concrete for consolidating concrete mixtures, vibration tables of various structures are used. The researches described in [1 – 7] the equipment technological efficiency of equipment and enterprise productivity has been determibed. The industry of Ukraine and CIS countries vibroforming equipment is not serially produced, and enterprises are forced to replenish it independently in conditions of metal and component parts scarcity. Existing deficit of vibration machines and uncertainty in the choice of priority directions of their development create significant difficulties in the production of designing or technical re-equipment of forming posts of prefabricated reinforced concrete enterprises.

Review of research sources and publications

The variety of such equipment is used in the production [1 – 7], which is explained by various scientific concepts of vibration technology and concrete mixtures consolidation, which have repeatedly changed, although the requirements for the quality of prefabricated reinforced concrete products practically remained unchanged. Existing deficit of vibration machines and uncertainty in their development priority directions choice create considerable difficulties in the production of technical re-equipment of prefabricated reinforced concrete enterprises forming posts technical re-equipment production.

The design bureau «Vibrotechnics» of the Poltava National Technical Yuri Kondratyuk University has developed a uniform series of low-frequency (24 Hz) vibrating tables with spatial oscillations of a moving frame (Figure 1) [7] which contains nine sizes of load-carrying capacity from 10 to 30 t for the production of products in the plan of 1.5x6 to 3×12 m. Vibration tables are extremely simple construction with a minimum of components, they are economical for power consumption, reliable in operation and are used both in closed production facilities and as in open landfills of reinforced concrete. By joining on short edges of two identical vibration tables, their maximum load-carrying capacity can be increased to 60 t, and the dimensions of the product - up to 3x24 m. Such vibration tables were produced in small quantities for construction companies.

Definition of unsolved aspects of the problem

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

Problem statement

The purpose of this work is to evaluate the technological capabilities of all varieties of vibration tables with moving frame spatial oscillations for consolidating the concrete mixture.

Basic material and results

Vibration tables with spatial oscillations of a moving frame are vibratory machines where the unbalanced vibration exciter of circular oscillations are driven by electric motors installed on the basement, with the help of a belt transmission (Fig. 1 and 2) [7]. In such structures of vibration tables there are no gears.

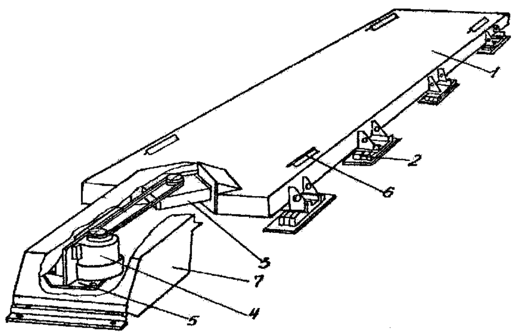

Figure 1 – Scheme vibration table with the end position of the vibration exciter:
1 – moving frame; 2 – elastic rubber-metal support; 3 – vertical shaft vibration exciter; 4 – electric motor; 5 – frame electric motor; 6 – stop wedge transverse; 7 – cover

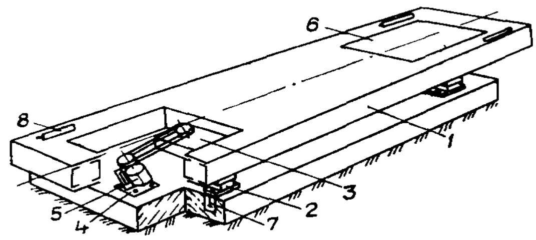

Figure 2 – Scheme vibration table with two inclined vibration exciters:
1 – moving frame; 2 – elastic rubber-metal support; 3 – vibration exciter; 4 – frame electric motor 5 – electric motor; 6 – cover; 7 – bolt basement; 8 – stop wedge transverse

Initial data when designing vibration tables for the formation of reinforced concrete products is the type of product, its mass and overall dimensions. The load-carrying capacity of the vibration table Q is determined, which is defined as the total mass of the form, the vibration equipment and technological equipment. The frame overall dimensions of the vibration table L_X and L_Y are calculated considering the overall dimensions in the form of the plan, with the need to further consideration the possibility of mounting the stop wedge transverses.

Spatial oscillations of moving frames provide sufficient technological efficiency at accelerations in the horizontal plane a range as a_h = (1,5...2,5)g, and at the level of sound pressure do not exceed the sanitary-hygienic norms and do not require additional measures.

When forming products of various sizes from mixtures with slump a range as 1 ... 4 cm, a form freely set on vibration moving frame and restrict from slipping through stop wedge transverses. When using concrete mixtures with the stiffness a range as 11 ... 20 s requires the jamming of forms on a moving frame, vibration tables two or three pairs of hard wedge stops. In addition to the carrying capacity, the size of the moving frame and the intensity of vibration, the technological capabilities of these vibration tables largely depend on the variants of the vibration exciters location on the moving frame vibration table with vertical or inclined unbalanced shafts relative to the vertical axis passing through the center of the oscillatory system masses. The advantage of using inclined vibration exciters is to increase the technological efficiency of low-frequency vibration tables by increasing the vertical component of the amplitude vibrations of the moving frame, that is important for the qualitative consolidating of stiff concrete mixtures.

Considered technological capabilities of vibration tables with spatial oscillations of the moving frame for the formation of the same type of reinforced concrete carried out according to the design schemes presented in Figure 3.

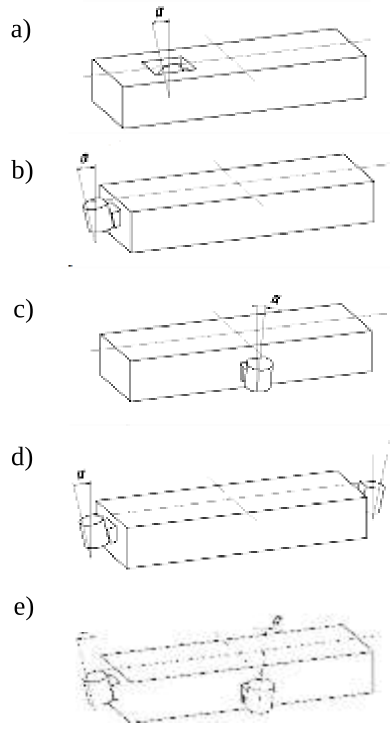

Figure 3 – Generalized technical scheme vibration tables with variants of the location of vibration exciters on the moving frame

When choosing vibration tables for the formation of reinforced concrete products it is convenient to use generalized technical characteristics of vibration tables with spatial oscillations of the working organ, given in Table 1.

Table 1 – Generalized technical characteristics of vibration tables

Parameter		Constructive execution (Figure 2)				
		a	b	c	d	e
Load-carrying capacity, t		0,2…25	10…25	0,4…20	12…100	2…..80
Dimensions in plan of products to be formed, m: – width – length		 0,8…3 1,5…12	 1…3 3…14	 1…2 2…12	 1,5…3- 3…15	 2…12 2…12
Power driven electric motors, kW		0,55-22	11-22	0,75-22	(11…30)x2	(1,1…30)x2
Weight of vibration table, t		0,3-8	5-11	0,4-10	5,6-16	0,6-14
Concrete mixture	slump, cm	5…10	1…4	1…4	–	1…4
	stiffnes, s	–	–	–	11…20	–
Application for the manufacture of reinforced concrete products		On aggregate-current and conveyor lines	On aggregate-current lines and at outdoor landfills		On aggregate-current and conveyor lines	At outdoor landfills
Types of formed products		Ribbed plates, wall linear and selective elements in cassette forms	Ribbed plates, pails	Plates of overlappings are linear and spatial elements	Many hollow Plates of overlappings, electric supports	Plates of overlappings, spatial elements, hydro-technical blocks

Analysis of constructive performances of vibration tables with spatial oscillations of a working frame:

a) with an in-depth location of the vibration exciter in the window of the moving frame, in other equal conditions, has the simplest construction and minimal dimensions in plans of the form relative size. The components of the vibrational displacements amplitudes on three coordinate axes are evenly distributed along the perimeter of the moving frame. The reduced values of the vertical components of the vibro displacement in the central part of the moving frame in the case of the installation of a vertical shaft vibration exciter may be aligned due to the tilting of the rotation axis of the unbalanced shaft a range as 5...15˚ from the vertical.

b) with the end face vibration exciter, like all the following, increases the dimensions of the vibration table relative to the size of the forms and requires the installation of protective covers over the vibration drive. This location of the vibration exciter is most convenient in terms of maintenance and repair. In this constructive version, along the length of the moving frame, two zones of reduced intensity of vibration appear: in the central part, due to the small vertical components of the amplitudes of the vibrational displacements, and in the zone distant from the vibrational platform at a distance equal to 1/4 ... 1/3 of the moving frame length, where the small transverse components of these amplitudes exist. Inevitable flexible oscillations of the form pallet elements and its sides, smoothes the display of the amplitudes calculated lower values of vibro displacement in these zones, and their presence has little effect on the quality of concrete mixtures compaction with slump a range as 3 ... 5 cm and more, however, this technological feature should be foreseen in quality control and commodity type of long-range products in the specified «risk areas». The application of the first portions of the concrete mixture to these places of the form for obtaining a long product promotes the uniformity of sealing the mixture throughout its length. In general, the good technological feasibility of this constructive option was determined by its recognition by manufacturers and relatively widespread at outdoor landfills of the year-round operation;

c) with a vibration exciter, mounted on the long side of the moving frame of the vibration table, provides more even distribution of the transverse amplitudes of vibrational displacements. Their calculated «lowered zone» extends beyond the boundary of the moving frame, but the vertical components in its central part retain reduced values, which is a definite condition for its application for the formation of spatial elements of different height with a central cavity (pressure elements of pipes of any diameter, rings for wells, elements of elevators, block rooms, etc.). The unbalanced shaft rotation slope enables to eliminate the «lowered zone» of the amplitudes vertical components of the vibrational displacements complicating the arrangement of the vibration drive;

d) with two independent vibration exciters installed on the ends or in the windows of the moving frame. When working with vibration exciters, there are forces of circular action that cause spatial oscillations of the moving frame and are transmitted through the form to the concrete mixture. The oscillating system is designed so that the vibrational displacement of the moving frame points has the character of the bits, which period varies from 1 to 8 s for different types of sizes. When intervals corresponding to the antiphase rotation of unbalances, when vibration exciters for a short time enter the self-synchronization mode, the moving frame makes oscillations predominantly in transverse and vertical planes. This mode of vibration enables to form concrete mixtures efficiently that increases the technological efficiency of the vibration table. The disadvantage of this design option is the complexity of the design and the increase in the number of component parts by doubled;

e) with two independent vibration exciters installed on the adjacent sides of the movable frame, considering the self-synchronization mode. Such a constructive implementation provides for an increase in mainly horizontal oscillations along its diagonal and to a lesser extent causes other components of the amplitudes of vibrational displacements. Such a technical solution is effective for the formation of spatial elements of mass and height with a square or rectangular cross-section. Vertical formwork forms perceive simultaneously the same dynamic pulses along the direction, and, accordingly, all four vertical walls of a spatial product receive the same degree of consolidation. Formation of reinforced concrete products with a mass of less than half of the calculated load-carrying capacity can be carried out by switching on one vibration exciter. The inclination of the vibration exciters increases the vertical component of the vibrations in the formation of large-sized products, with dimensions in the plan of 6×6, 9×9, or 12×12 m. On such a vibration tables installed on an open landfill near the construction site, in the area of the crane, the large-sized fragments of the assembly monolithic constructions. It enables to implement progressive volumetric planning decisions and to shorten the construction period.

It is convenient to create stationary vibration forms for the production of large-sized and spatial reinforced concrete products, to carry out modernization of cassette and other installations on the basis of unified nodes of vibration tables in accordance with the above considered variants of constructive execution of vibration tables.

The unified vibration kits VK-1, VK-2, VK-3 (Table 2) enable to organize the production of large-size reinforced concrete structures in improved stationary vibration forms of different configuration and mass, both in closed production facilities and in oupan landfills in the short term and with minimal cost,. The fastening nodes of vibration exciter and elastic supports are modernized to a concrete form individually.

It is also easy to create small vibration tables with a load-carrying capacity from 0,2 to 2,5 t with the use of one or two suspended universal vibration exciters for the formation of common small-sized reinforced concrete products in single or cassette forms.

For example, the vibration table PL-2,5 with two hinged vibration exciters IV-105 with a frequency of 24 Hz and a total power of 2.2 kW has a carrying capacity of 2.5 t and enables to form products with dimensions in the plan of 1.6x7.2 m. The feasibility of manufacturing small vibration tables is confirmed by many foreign firms in Italy, France, Japan. In the CIS countries, such vibration tables are not manufactured.

Table 2 – Generalized technical characteristics of unified vibration kits

Parameter	Vibration kit		
	VK-1	VK-2	VK-3
Load-carrying capacity (mass of form with a concrete mixture), t	5-12	15-25	30-55
Frequency of oscillations of a moving frame, Hz	24–30		
Vibration drive power, kW	7	11	22
Weight of units of vibration kit (depending on the number of supports), kg	700-840	870-1190	1590-2060

Technological and operational parameters of vibration tables [8 – 10] are also easy to evaluate according to the calculation method proposed by us [10]. The method of calculation, based on the use of a dimensionless complex parameter, enables both at the design stage and during the process of exploitation to evaluate the consumer quality of vibration tables, their technical level and competitiveness by the following formula

$$I = \frac{\sqrt{(ka_h fg\omega)^2 + a_v^2\omega^3}}{N/Q} \cdot \frac{Q^2}{M_v M_b} \cdot \frac{115 - L}{L} \cdot \frac{180}{t}, \quad (1)$$

where k – coefficient considering the influence of the tangent component of the amplitude of the oscillations of the concrete mixture, is taken within the range as 0.2 ... 0.4 depending on the stiffness of the concrete mixture;

f – coefficient considering the friction of the concrete mixture on the pallet of the form (f = 0.10 ... 0.12);

g – acceleration of free fall;

a_h and a_v – amplitude of the horizontal and vertical vibrations of the moving frame respectively;

ω – angular frequency of oscillations, s^{-1};

N – installed power of the vibration drive, kW;

Q – maximum load-carrying capacity of the vibration table, t;

M_v – weight of the vibration table, t;

M_b – weight of basement vibration table, t;

L – actual equivalent noise level at the workplace, dBA;

t – the total duration of the vibration table inclusions during the cycle of products formation of the same type, s.

The complex parameter includes four factors, which respectively reflect: the efficiency of energy consumption, the material content of the vibration table, considering the weight of the basement, relative sanitary and hygienic conditions of work on the noise level at the workplace, technological efficiency of the vibrating machine. The higher the dimensionless complex parameter I, the more perfect the vibration table. In tab. 3 there are shown the results of the comparison with the proposed dimensionless complex parameter of the operational qualities of the most common in the production of four vibration tables for the formation of the same type of products with a size of 3.0×6.0 m. In this case, the amplitude of the vibro displacements indicated in Table 3 corresponds to the regimes adopted on practice that ensures the performance of these vibrating machines and the proper quality of consolidating the concrete mixture. The dimensionless complexity of the vibrating table VPG-2M-07 is higher in comparison with vibrating table areas: SMG-538A in 1,66 times; SMG-200G is 7.16 times, SMG-773 is 1.33 times.

Table 3 – Comparison of technological and operational qualities the most common vibration tables with the help of dimensionless complex parameter

Parameter	Values of parameters for vibration tables			
	SMG -538A	SMG -200G	SMG -773	VPG-2M-07
Load-carrying capacity, t	18	15	20	16
Angular frequency of oscillations, s^{-1}	152	300	152	188
Actual equivalent noise level, dBa	80	100	85	80
Amplitude of vibrations of the working body, m: – horizontally – vertically	 – $0.75 \cdot 10^{-3}$	 – $0.35 \cdot 10^{-3}$	 – $1.0 \cdot 10^{-3}$	 $0.60 \cdot 10^{-3}$ $0.28 \cdot 10^{-3}$
Installed power of the vibration drive, kW	44	88	50	37
Basement mass, t	75	120	90	48
Total vibration drive activation time, s	180	90	180	180
Dimensionless complex parameter	0.206	0.048	0.259	0.34

Conclusions

1. The main property of vibration tables with spatial oscilations of a moving frame is the possibility of the entire nomenclature products qualitative and productive formation according to the planned volume at a separate factory of reinforced concrete products, which facilitates their manufacturing, maintenance and ongoing repair of forming equipment.

2. The given schemes of constructive variants of execution of vibration tables with spatial oscilations of a moving frame enable designers and manufacturers to be more easily guided in the variety of currently used vibration machines for the formation of the same type of reinforced concrete products.

3. The method for evaluating the performance of vibration tables with the help of a dimensionless complex parameter enabes both at the design stage and during the process of exploitation to evaluate the consumer qualities of vibration machines of different design, to determine their technical level and competitiveness on the set of basic technical parameters.

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DIFFERENT CHARACTERISTICS BRAKE AUTOMOTIVE SYSTEM SALES RESEARCH MANUFACTURED BY ADDITIVE TECHNOLOGIES

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Problems in designing hydraulic brake systems of cars have been analyzed. The study of the braking process has conducted considering the distribution of forces on the axles during braking. The general support model research and support geometric characteristics are developed. The calculation of the car brake system has been conducted that enables to determine the load on the caliper during the braking process and using graphic dependencies, and to determine the concentration of maximum stresses in the mounting areas of the support. Based on analytical studies in the Autodesk Fusion 360 3D Modeling program, a design has been implemented that features a smaller metal capacity at the expense of rational design and providing uniform strength across the entire surface. Adaptive technologies also significantly accelerate the production of new parts due to the lack of need for the production of auxiliary equipment, such as molds.

Keywords: brake system of the car, additive technologies, support.

ДОСЛІДЖЕННЯ МІЦНІСНИХ ХАРАКТЕРИСТИК СУПОРТА ГАЛЬМІВНОЇ СИСТЕМИ АВТОМОБІЛЯ, ЩО ВИГОТОВЛЕНИЙ ШЛЯХОМ АДИТИВНИХ ТЕХНОЛОГІЙ

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Проаналізовано проблеми, що виникають при проектуванні гідравлічних гальмівних систем автомобілів. Запропоновано за допомогою адитивних технологій виготовити деталі гальмівної системи складної конфігурації, чого неможливо досягти іншими способами, наприклад виготовляти деталі, що мають внутрішні канали, порожнини, отвори складної форми і т. п. У деяких випадках завдяки цьому можна досягти зменшення кількості складальних вузлів, кріплень та відповідно зменшити розміри і масу вузлів. Виконано аналітичні дослідження процесу гальмування з урахуванням розподілу сил по осях автомобіля під час гальмування. Оскільки шина автомобіля є єдиним сполучним елементом між автомобілем та дорожньою поверхнею, у розрахунковій схемі розподілу сил, крім неї, представлено супорт і гальмівний диск. На прикладі супорта гальмівної системи автомобіля розроблено загальну модель дослідження міцнісних та геометричних характеристик деталі. Наведено розрахунок гальмівної системи автомобіля, який дозволяє визначити навантаження на супорт у процесі гальмування й за допомогою графічних залежностей визначити концентрацію максимальних напружень у місцях кріплення супорта. Базуючись на аналітичних дослідженнях, у програмі для 3D-моделювання Autodesk Fusion 360 реалізовано проект конструкції супорта, який відрізняється тим, що має меншу металоємність за рахунок раціонального дизайну та забезпечення однакового коефіцієнта міцності по всій поверхні. Запропоновано виготовлення такої деталі за допомогою адитивних технологій, що дозволяє досягти значної економії матеріалу завдяки багатошаровому нанесенню матеріалу й виключенню ряду технологічних операцій з обробітку заготовки. Адитивні технології також забезпечують високу технологічність виробництва та значно прискорюють виготовлення нових деталей через відсутність потреби у виробництві допоміжного обладнання, наприклад прес-форм.

Ключові слова: гальмівна система автомобіля, адитивні технології, супорт.

Introduction

This publication topicality is based on the fact that it addresses the additive technologies use issue for the car parts manufacture. These technologies make it possible to produce parts with material rational distribution in the product volume. It reduces the mass of individual parts, and as a consequence, the mass of nodes and the whole vehicle.

Adaptive technologies enable to fabricate details of a complex configuration that cannot be achieved in other ways. For example, it becomes real to produce parts that have internal channels, cavities, holes of a complex shape, etc. In some cases, it is possible to achieve reduction in the number of assembly units, fasteners, and, accordingly, to reduce its dimensions and mass.

The research is aimed to improve the efficiency of the car by reducing its mass and to ensure its management safety. For detailed processing, the brake system of the car is chosen as such one that during the operation takes considerable load, and its serviceability and reliability influence the driving safety.

Review of research sources and publications

In the modern world, the complexity of hydraulic systems is quite high and constantly increasing (Fig. 1) [7 – 8]. The problem is that the usual methods of manufacturing parts cannot provide the manufacturing of those technical solutions that bring the given system to a new level. Due to manufacturing problems, engineers cannot implement innovative solutions for individual systems and components.

Nowadays, the parts used in hydraulic systems have a lot of weight. And there are concepts with innovative solutions that enable to make a breakthrough in the industry. The problem with designing the braking systems is that the braking system details have a complex structure of the internal channels where hydraulic losses occur. Due to existing manufacturing methods, the channels have a curvilinear shape, increasing the fluid resistance.

Figure 1 – BMW mechatronics

The additive manufacturing technologies development current level gives engineers freedom in designing parts and systems. With the help of additive technologies, any form of part and its internal structure can be made with high accuracy virtually (Fig. 2).

Figure 2 – An example of a component made by additive technology

Thus, by applying additive technology, it can be significantly reduced the weight of the brake system component and hydraulic losses throughout the system.

Definition of unsolved aspects of the problem

In modern braking systems of cars parts are used that have a significant metal content and an inflated strength across the entire surface. It is due to the lack of precision in the calculation of loads on the part and the organization of the technological process of their production, based on traditional foundry processes and metal cutting operations. In order to reduce the metal content of the car brake systems, today more precise calculations of loads on the part should be carried out and when designing modern graphic 3D simulation programs should be applied. It is expedient to produce details of the braking system of a complex configuration with the help of additive technologies that are difficult to obtain in other ways, for example, to produce parts having internal channels, cavities, holes of complex shape and so on.

Problem statement

The purpose of this article is to analyze the problems that arise during the design of cars hydraulic brake systems, the development of proposals for the calculation, 3D modeling and the application of modern additive technology in the system parts manufacture.

Basic material and results

Investigation of the braking the car process in most cases is an investigation of emergency braking and is reduced to determining the car speed and the brake path [1, 2, 3]. The tire is the only connecting element between the car and the road surface. Tire engagement is a decisive factor in road safety. So, in the distribution scheme, the support, the brake disk and the tire are depicted. In the process of braking the car instead of the torque, the braking torque is applied to the vehicle driving wheels, and instead of the traction effort, the braking force directed towards the reverse movement is substituted (Fig. 3).

The equation of motion in braking has the form

$$-P_г = P_f + P_w \pm P_i - P_j \,. \qquad (1)$$

The amount of braking force is determined from expression

$$P_г = \gamma_г G_k g \;, \tag{2}$$

where $\gamma_г$ – coefficient of specific braking force equal to the ratio of the braking forces sum occurring on all brake wheels to the weight of the vehicle ($\gamma_г$ = 0.9) [1]; g – free fall acceleration.

Equation of Negative Acceleration Motion j

$$j = \frac{1}{\delta}\left(\frac{P \cdot w}{G} + \gamma_г \pm i + f\right). \tag{3}$$

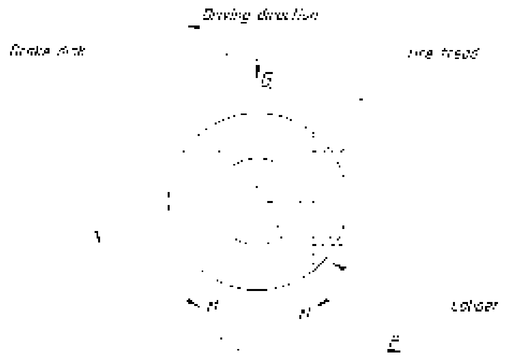

Figure 3 – Diagram of the distribution of forces during braking:
M_{kr} – torque; P_q – braking force; M_q – braking torque; G_k – the mass of the car falling on the wheel.

In modern cars with brakes on all wheels during emergency braking, the limiting value γ_t equals the coefficient of tire adhesion with the cover φ when driving on a road straight line and φ_1 – when moving along the curve.

Since during braking the car speed is sharply reduced, the air resistance can be neglected, then at δ = 1 (direct transmission)

$$j = (\varphi \pm i + f). \tag{4}$$

Also, the value of the braking distance on which the driver can stop a car that moves at the rated speed is normalized. The path of complete braking can be found in the formula of the equilibrium motion

$$v = \sqrt{2aS_m}\;, \tag{5}$$

where a – absolute negative acceleration, m/s^2

$$a = g \cdot j = g(\varphi \pm i + f). \tag{6}$$

Thus, the braking distance at v in m / s is according to the formula

$$S_m = \frac{v^2 K_e}{2g(\varphi \pm i + f)}\;, \tag{7}$$

where K_e – the braking efficiency coefficient, considering that brakes state operation is equal to 1.2 for cars and 1.3 to 1.4 for trucks [1].

When braking a car on a horizontal road, an inertia force is formed P_i, applied to the center of gravity and equal to the sum of braking forces [1]. In this case, there is a redistribution of normal loads on the axes: the front loaded, and the rear axle is unloaded. In the static state of the vehicle, the load on the axle is determined by the distances a and b of the center of mass O from the front and rear axles.

$$G_1 = G\frac{b}{L}\;,\quad G_2 = G\frac{a}{L}\;. \tag{8}$$

The normal reaction of the road acting on the front and rear wheels during braking is defined as

$$R_{z1} = G\frac{b + \varphi_x \cdot h}{L}\;,\quad R_{z2} = 1 - \frac{\varphi_x \cdot h}{L}\;. \tag{9}$$

Normal reactions R_{z1} and R_{z2} are perceived by the wheels during braking, differ from the loads that fall on the wheels in the static state. Redistribution of axial weights is estimated by the coefficients of variation of the responses m_{p1}, m_{p2} that are equal to the horizontal road

$$m_{p1} = 1 + \frac{\varphi_x \cdot h}{b}\;,\quad m_{p2} = 1 - \frac{\varphi_x \cdot h}{a}\;. \tag{10}$$

Then the normal road responses are of the form

$$R_{z1} = m_{p1} \cdot G_1\,,\quad R_{z2} = m_{p2} \cdot G_2\;.. \tag{11}$$

When braking, the limit values of the reaction change coefficients are 1,5÷2,0 front wheels and 0,5÷0,7 – for the rear. The highest inhibition of the vehicle is achieved with the full use of clutch on all wheels, which is possible only on the road with optimum clutch ratio φ_{opt} = 0.4…0.5 [5].

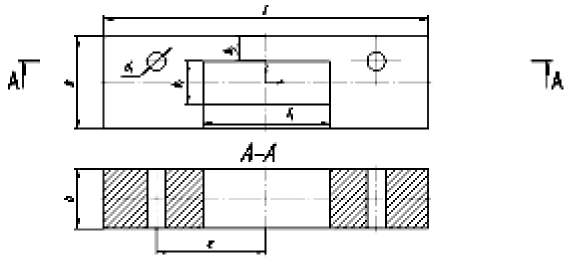

Figure 4 – Simplified support scheme

The carrier car accepts different types of loads, so it is necessary to examine it tension and compression. During the car parking on a sloping road, the caliper perceives static loads, and in the case of emergency braking – dynamic. The static load on the support is considered. For calculations, a simplified model of the support is suggested (Fig. 4).

When braking, the pads transmit the braking force F from the brake disc to the servomotor at the point of contact. An equilibrium braking force and the reaction of the support in the force diagram are depicted (Fig. 5).

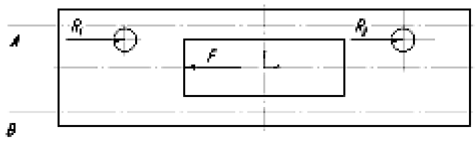

Figure 5 – Scheme of forces acting on the caliper and the axes A and B

During normal stretching and compression in the cross sections of the support, normal stresses are distributed uniformly in the section [4]. They are calculated by the formula

$$\sigma = \frac{N}{A}\ , \tag{12}$$

where N – the longitudinal force;
A – cross-sectional area.

Obviously, for tension and compression, the shape of the cross section does not affect the voltage value.

In the sections close to the places of forces application, the law of stresses distribution is more complicated, but using the boundary conditions mitigation principle, these deviations are neglected and it is considered that all cross sections of the cable voltage are evenly distributed and that in the section where along the axis concentrated force is applied to the beam, the values of the longitudinal force and stresses vary jump-like.

Since, the concentrated force and forces of the support reaction do not act on one axis, it is necessary to solve the equation of moments. Due to the fact that our model has a window in the middle, the equation relative to the two axes A and B is solved (Fig. 5).

Consequently, the equations of moments is:

$$\sum M_a = 0,$$

$$\frac{-F\left(\frac{b_1}{2}+\frac{b_2}{2}\right)+R_1\left(b_1+\frac{b_2}{2}\right)+R_2\left(b_1+\frac{b_2}{2}\right)}{b_1+b_2} = N_1; \tag{13}$$

$$\sum M_a = 0,$$

$$\frac{-F(\frac{b_1}{2}+\frac{b_2}{2})+R_1\frac{b_2}{2}+R_2\frac{b_2}{2}}{b_1+b_2} = N_2. \tag{14}$$

After calculations, we substitute the greatest moment obtained into (12) and find the stress.

This figure should not exceed the permissible stresses for the particular material from which the caliper is made

$$\sigma \leq [\sigma]\ . \tag{15}$$

The cutoff forces also appear in the caliper, because the offsetting force transmitted from the brake disk is applied at a distance from the mounting points of the support.

It is believed that the tension stresses are distributed over the support section evenly. Then their value is equal, Pa,

$$\tau_{зр} = \frac{N}{A_{зр}}\ , \tag{16}$$

where $A_{зр}$ – the cross-sectional area of the cut;
N – longitudinal force.

The resulting stresses should not exceed the permissible stresses on the cut for the selected material

$$\tau_{зр} \leq [\tau]\ , \tag{17}$$

For static loads, the permissible stresses on the cut are assumed to be approximately 70% of the permissible stresses on tension or compression for the same material

$$[\tau] = 0{,}7\ [\sigma]\ . \tag{18}$$

For further calculations the prototype of the vehicle with the average Indicators – Daewoo Lanos 1.6 16V 2000 year of production is accepted.

Required technical specifications for calculations:
– allowable load on the front axle – 890 kg (445 kg per wheel);
– the size of tires – 175/50R13;
– the size of the brake disk – 236 mm.

It is suggested the following dimensions of the simplified model of the support for checking the strength characteristics (Figure 3-4).

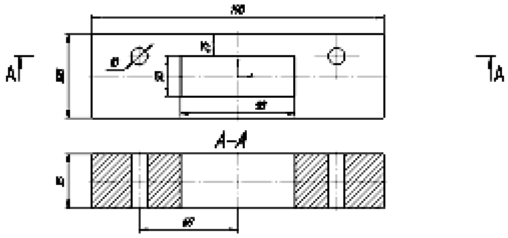

Figure 6 – Dimensions simplified model for calculating

Above-mentioned permissible load on the front wheel and the value of the coefficient of grip, are substituted in the formula (2) and the braking force is found that occurs on the front wheel $P_г$ = 0.9 · 445 · 9.81 = 3929 H

Braking force acting on the $P_{гс}$ caliper, by the ratio shoulders (Fig. 7) is found. The size 175/50R13 has an external diameter of 500 mm, so the braking force to the tire is 250 mm. And the force acting on the support is applied in the center of the pads and has a shoulder to the center of the wheel 218 mm.

Thus, the effort on the support $P_{гс}$ is equal to

$$P_{гс} = 3929 \cdot 250 / 218 = 4505\ \text{H}.$$

Depending on the dependencies (1), (2), (3), (8), (9), (10), (11), longitudinal forces are found, and from the equations of the moments (13) and (14) relative to the two axes A and B, respectively, N_1 = 901.13 H, N_2 = –901.13 H.

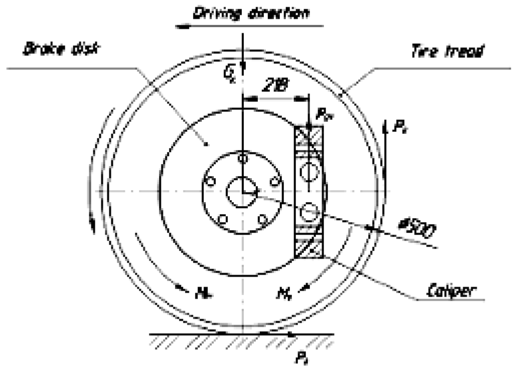

Figure 7 – Scheme of the ratio of the forces of the application of the arms

It is found the maximum stresses in the section by substituting the maximum value of the force in the dependence (12)

$$\sigma = \frac{901.13}{2 \cdot 0.025 \cdot 0.035} = 514931 \text{ Pa.}$$

The stresses found are 0.51 MPs, but do not exceed the permissible stresses, which for aluminum are 30-80 MPa.

The distribution of the stresses along the *X*, *Y*, *Z* axes is reduced to build a graph (Fig. 8).

According graphical dependencies stress distribution in cross section, their concentration in the area of mount is shown

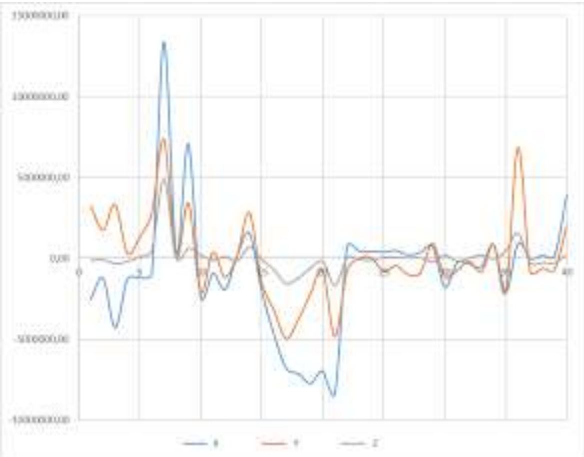

Figure 8 – Distribution of stresses along the axes X, Y, Z

To ensure regular geometric characteristics, it is necessary to study the scheme displacement stress concentrators in their respective sections. The given data are summarized reducing to build a graph (Figure 9).

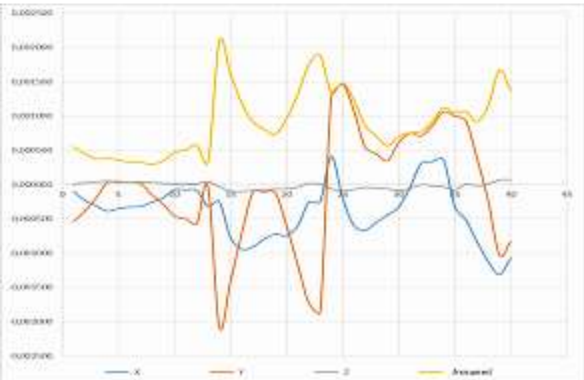

Figure 9 – Moving relative to the axes

Next, the tensile stresses in the section Pa are found and the cross section for the cut is calculated. The maximum calculated longitudinal force into formula IS substituted (16)

$$\tau = \frac{901.13}{0.04 \cdot 0.035} = 180200 \text{ .}$$

The result is converted into MPa, 0.18 MPa is got.

By formula (18) the permissible voltages for aluminum on the cut, MPa is found

$$[\tau] = 0{,}7 \cdot 30 = 21.$$

Based on the results of analytical studies, the support design is suggested, which is based on the above-calculated overall dimensions, but with a significantly reduced metal capacity, which reduces its mass. This model of support is implemented in the program for 3D modeling Autodesk Fusion 360 (Figure 10).

Figure 10 – The proposed design of the support

Conclusions

Execution of support proposed design strength research, which was made by means of additive technologies, enables to achieve the following results:

– the calculated section has an excess factor of strength, which adversely affects its mass; therefore, to reduce metal strength and weight, a rational design of the support is proposed;

– due to layered construction, considerable material savings are achieved in the manufacture and further processing of parts;

– adaptive technologies can significantly accelerate the production of new parts due to the fact that there is no need for the production of auxiliary equipment (e.g., molds etc.). Due to this, the speed of the entire production cycle increases significantly;

– it is possible to make details of complex configurations that cannot be made by other known methods. Due to this, parts with less weight without losing their durability can be made.

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MATHEMATICAL MODEL OF PRESSURE CHANGE IN AUTOMOBILE PNEUMATICAL TIRE DEPENDING ON OPERATING TEMPERATURE

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It has been established that in the process of operation pressure ratings in the tires of many cars differs from those recommended by the production plant. Is leads to performance degradation of tires traveling properties and their loss of life. The pressure excursion from the normative value may be caused either by an error during tire inflation, or by the fact that the difference between the operating temperature and the temperature of the inflating air has not considered. Using athematical-statistical methods of data processing, there has been deduced the mathematical relationship between pressure in the pneumatical tire at the operating temperature and the required pressure of inflating air into the tire, if the temperatures of inflation and operation differ.

Keywords: pneumatical tire, inflation pressure, three-level plan, planning matrix, mathematical model.

МАТЕМАТИЧНА МОДЕЛЬ ЗМІНИ ТИСКУ В АВТОМОБІЛЬНІЙ ПНЕВМАТИЧНІЙ ШИНІ ЗАЛЕЖНО ВІД ТЕМПЕРАТУРИ ЕКСПЛУАТАЦІЇ

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Установлено, що в процесі експлуатації значення тиску в шинах багатьох автомобілів відрізняється від рекомендованого заводом-виробником, що призводить до погіршення експлуатаційних характеристик шин та скорочення їхнього ресурсу. Відомо, що відхилення тиску від нормативного значення може бути спричиненим як похибкою при накачуванні шини, так і неврахуванням різниці між температурою експлуатації та температурою повітря, що закачується. Для проведення дослідження виділено найбільш значущі фактори, які впливають на значення тиску в пневматичній шині, що дозволило скласти рівняння регресії, котре описує явище. Доведено адекватність моделі через визначення похибки та перевірку критеріїв. За допомогою математико-статистичних методів оброки даних виведено математичну залежність між тиском у пневматичній шині при температурі експлуатації та необхідним тиском закачування повітря в шину, якщо температури накачування та експлуатації відрізняються. Отже, отримано математичну залежність, яка дозволяє при відомих температурі в приміщенні, де здійснюється накачування, температурі навколишнього середовища, де буде експлуатуватися шина, та рекомендованому заводом-виробником тиску в шині автомобіля визначити необхідний тиск закачування повітря. Практичним використанням результатів дослідження є значення необхідного тиску накачування залежно від температур.

Ключові слова: пневматична шина, тиск накачування, трирівневий план, матриця планування, математична модель.

Introduction

The durability of pneumatic tires depends on many factors. Gas pressure in the tire is one of the most important ones. Tires of the same model are installed on the cars of different vehicle brands. The car production plant carries out research and establishes the optimum tire pressure when it has the vehicle maximum loss of life. In the case of pressure excursion from the normative value, the service life of the tire is significantly reduced [1, 2]. Also, the safety and comfort of the vehicle movement, fuel consumption, durability of a car suspender depend on the tire pressure [3-6].

However, it is known that the pressure does not meet the norm in 60 – 90% of tires in operation. Due to the fact, that the pressure does not conform to this norm, there are lost from 6 – 15% of the tire life and from 1.5 to 3.0% of fuel [7, 8].

Thus, there is a problem of securing operation of tires with standard pressure. The solution to this problem is complicated by the fact that the pressure change is determined by a large number of different factors. Some factors influence has not been studied sufficiently. Consequently, the crucial task is the study changing gas pressure process in pneumatical tires and the development of measures aimed at the reduction of its deviations from the norm.

Review of research sources and publications

It the paper it has been established that environmental temperature shift affects the number of technical and operational parameters of a pneumatical tire [9, 10]. Pressure change is particularly notable.

In the car operation process, the temperature fall can reach several dozens of degrees [7]. For example, during car tire fitting at the premises of a service station or a garage, the air, which temperature is equal to the air temperature in the room, is inflated into the tire. However, outdoors, especially when it is cold, the atmospheric temperature can vary significantly. At the same time, the pressure change takes place in the tire.

Definition of unsolved aspects of the problem

As it has been noted above, the air temperature inside and outside the tire can vary significantly in different periods of time. It affects the pressure amount, when the air temperature inside the tire is equal with ambient temperature. But the tire pressure is regulated by the manufacturer [4, 11]. So, there is a need for further study of this effect for the necessary adjustments and description in the mathematical dependencies field [12 – 14].

Therefore, the issue of forecasting the pressure change in the tire during its operation at different temperatures has not been solved fully.

Problem statement

The goal of the research is determination of mathematical relationship between pressure in the pneumatic tire at the operating temperature and the required pressure of inflating air into the tire if the inflating and operation temperatures differ.

Dasic material and results

To derive a mathematical expression that establishes the functional relationship between the values of standard pressure in the tire, the tire air pressure after finished inflation, the inflating air temperature and the operating temperature for the tire, mathematical-statistical methods have been used [15].

In the investigation process the following assumptions have been introcced:

– the tire volume does not change with the tire pressure change;

– the composition of the inflating air corresponds to the composition of an air at the sea level;

– the inflation is carried out at an atmospheric pressure of 101.3 kPa.

– the main factors of the influence on the standard tire pressure value during operation are: pressure and temperature of the tire inflating air, ambient temperature. Other factors are neglected.

When computations are conducting, the selected factors vary on three levels – middle (main), lower and upper, which are remote from the main level to the same value. This value is called the variation interval (Table 1)

It simplifies the records and subsequent calculations, when the upper level of factors is denoted by the symbol «+», the middle one is «0», and the lower one is «-», which is equivalent to the transfer of factors to the new code scale.

$$x_i = \frac{X_i - X_{i0}}{\Delta X_i}, \tag{1}$$

where x_i – value of *i*th factor on the new code scale;

X_i – value of *i*th factor on a natural scale;

X_{i0} – main *i*th factor level;

ΔX_i – *i*th factor variation interval.

In our case, it assigns the following notations to the factors that consider this calculation:

– the tire air pressure after finished inflation is x_1

$$x_1 = \frac{X_1 - 202.6}{50.65}; \tag{2}$$

– the inflating air temperature is x_2

$$x_2 = \frac{X_2 - 10}{10}; \tag{3}$$

– the operating temperature for the tire is x_3

$$x_3 = \frac{X_3 - 0}{30}. \tag{4}$$

The matrix of the second order for processing of the computational results has been used, since the studied dependence is unknown [15]. Calculations are made according to the matrix plan, and they are reduced to table 2.

Table 1 – The limits of change and factors variation intervals

Factors	The limits of factors change	Variation intervals
Tire air pressure after finished inflation P_i, kPa (atm)	151.95 – 253.25 (1.5 – 2.5)	50.65
Inflating air temperature t_i, °C (K)	0 – 20 (273 – 293)	10
Ambient temperature when the tire is operated t_o, °C (K)	–30 – +30 (243 – 303)	30

Table 2 – Three-level plan of the second order with number of factors $k=3$ $\left(N=N_1+N_\alpha+n_0\right)$

Calculation number		Planning matrix (x_i)			Squared variables (x_i^2)			Factor interaction $(x_i x_j)$			Tire pressure, kPa
		x_1	x_2	x_3	x_1^2	x_2^2	x_3^2	x_1x_2	x_1x_3	x_2x_3	y_i
1		2	3	4	5	6	7	8	9	10	11
N_1	1	+	+	+	+	+	+	+	+	+	266
	2	–	+	+	+	+	+	–	–	+	160
	3	+	–	+	+	+	+	–	+	–	281
	4	–	–	+	+	+	+	+	–	–	169
	5	+	+	–	+	+	+	+	–	–	214
	6	–	+	–	+	+	+	–	+	–	128
	7	+	–	–	+	+	+	–	–	+	225
	8	–	–	–	+	+	+	+	+	+	135
N_α	9	+	0	0	+	0	0	0	0	0	246
	10	–	0	0	+	0	0	0	0	0	148
	11	0	+	0	0	+	0	0	0	0	192
	12	0	–	0	0	+	0	0	0	0	203
	13	0	0	+	0	0	+	0	0	0	219
	14	0	0	–	0	0	+	0	0	0	175
n_0	15	0	0	0	0	0	0	0	0	0	196
	16	0	0	0	0	0	0	0	0	0	197
	17	0	0	0	0	0	0	0	0	0	198

The calculated values of the pressure in the tire (columns 11 in Table 2) are obtained on the basis of the air composition data at the sea level [16], and under the condition that the test tire has a volume of 25 liters and the gas mass (inside the test tire) corresponds to the air mass at a pressure of 202.6 kPa and a temperature of 20°C. The calculation is carried out according to the formula

$$P_o=\frac{M_a\cdot R_{da}\cdot T_a}{V_a}, \tag{5}$$

where M_a – air mass in the tire, kg;
R_{da} – specific gas constant for dry air [16], J/(kg·K);
T_a – air temperature in the tire, K;
V_a – air volume in the tire, m^3.

The table columns 2 - 4 represent a matrix that defines the initial conditions for conducting calculations. The table columns 5, 6 and 7 show the squared variables obtained as a result of the data columns 2 - 4 squaring. They acquire the values +1 or 0 and are marked as x_i^2. Column 8 is obtained by sequential multiplication of factors (variables interaction). For example, for the 8th calculation according to the plan factors $x_1 = -1$ and $x_2 = -1$ should be set at the lower level, and the estimated value of the interaction is

$$x_1\cdot x_2=(-1)\cdot(-1)=+1. \tag{6}$$

The last column 11 shows the calculation results of the tire pressure at various combinations of factors («outputs»).

The calculation results are processed according to the method [15]. In this case, an algebraic equation is obtained. It reflects the relationship between the properties under investigation and the initial factors.

The algebraic equation in general terms is

$$\hat{y}_i=b_0+\sum_1^k b_i x_i+\sum_1^k b_{ii}x_i^2+\sum_1^k b_{ij}x_i x_j, \tag{7}$$

or

$$\begin{aligned}\hat{y}_i=&\,b_0+b_1x_1+b_2x_2+b_3x_3+b_{11}x_1^2+b_{22}x_2^2+b_{33}x_3^2+\\&+b_{12}x_1x_2+b_{13}x_1x_3+b_{23}x_2x_3,\end{aligned} \tag{8}$$

where $i,j=1,2,\dots,k$ – factors order numbers, $i\neq j$;
$\hat{y}_i$ – property under investigation – tire pressure;
x_1, x_2, x_3, x_4, … x_k – initial factors;
b_0, b_1, b_2, … b_{12}, b_{13}, … b_{ij}, b_{ii} – equation coefficients.

The coefficients for the plan of the second order with the number of factors k = 3 are calculated according to the formulas:

$$b_0=0.1831[0y]-0{,}0704\sum_1^k[iiy]; \tag{9}$$

$$b_i=0.1[iy]; \tag{10}$$

$$b_{ii} = -0.0704[0y] + 0.5[iiy] - 0.1268\sum_1^k [iiy]; \quad (11)$$

$$b_{ij} = 0.125[ijy], \quad (12)$$

where

$$[0y] = \sum_1^N y_u; \quad (13)$$

$$[iiy] = \sum_1^N x_{iu}^2 y_u; \quad (14)$$

$$[iy] = \sum_1^N x_{iu} y_u; \quad (15)$$

$$[ijy] = \sum_1^N x_{iu} x_{ju} y_u, \quad (16)$$

according as $i \neq j$;

y_u – value of property under investigation in *u*th calculation;

x_{iu} – value of *i*th factor in *u*th calculation;

x_{ju} – value of *j*th factor in *u*th calculation $(i \neq j)$;

N – total quantity of calculations in plan (null point included).

After processing the calculations for the adopted plan, the following equation is obtained

$$\begin{aligned} y &= 197.19 + 49.2 \cdot x_1 - 5.3 \cdot x_2 + 21.8 \cdot x_3 - 0.26 \cdot x_1^2 + \\ &+ 0.24 \cdot x_2^2 - 0.26 \cdot x_3^2 - 1.25 \cdot x_1 \cdot x_2 + 5.25 \cdot x_1 \cdot x_3 - \\ &- 0.75 \cdot x_2 \cdot x_3. \end{aligned} \quad (17)$$

Statistical analysis of the quadratic dependence is carried out according to the method [15]. In order to check the significance of the coefficients in the factors, the error is artificially set in the calculations of pressure at the null points within ±5%.

According to the results of calculations at the null points, it is determined:

a) arithmetical average

$$\bar{y}_0 = \frac{\sum_1^{n_0} y_{0u}}{n_0}, \quad (18)$$

where y_{0u} – value of property under investigation at the null point in *u*th calculation;

n_0 – quantity of calculations at the null point;

b) dispersion at the null point

$$S_{\bar{y}}^2 - S_0^2 = \frac{\sum_1^{n_0}\left(\bar{y}_0 - y_{0u}\right)^2}{n_0 - 1}; \quad (19)$$

c) mean-square deviation

$$S_{\bar{y}} = S_0 = \sqrt{S_0^2} = \sqrt{\frac{\sum_1^{n_0}\left(\bar{y}_0 - y_{0u}\right)^2}{n_0 - 1}}; \quad (20)$$

d) root-mean-square error in determining coefficients

$$S\{b_i\} = \frac{S_{\bar{y}}}{\sqrt{N_1}}. \quad (21)$$

The estimated value of Student-test is determined in the following form:

$$t_c = \frac{|b_i|}{S\{b_i\}}, \quad (22)$$

and compare the resulting value t_c with the tabular one t_t with number of freedom degrees $f_{\bar{y}}$, by which it was determined $S_{\bar{y}}$ [15];

$$f_{\bar{y}} = n_0 - 1. \quad (23)$$

In the case $t_c < t_t$ with a level of significance $\alpha = 0.05$, the coefficient is taken to be equal to zero, and the corresponding equation member is rejected.

After mathematical processing of data, the refined equation is obtained in the form:

$$y = 197.19 + 49.2 \cdot x_1 - 5.3 \cdot x_2 + 21.8 \cdot x_3 + 5.25 \cdot x_1 \cdot x_3. \quad (24)$$

To verify the suitability of the obtained refined equation, the adequacy dispersion is calculated

$$S_{ad}^2 = \frac{\sum_1^{N_1}\left(y_u - \hat{y}_u\right)^2}{N_1 - m}, \quad (25)$$

where y_u – value of property under investigation in *u*th calculation;

$\hat{y}_u$ – value of property under investigation in *u*th calculation, which is calculated according to the refined equation;

m – quantity of significant coefficients together with b_0.

The estimated value of F-test F_p is obtained

$$F_p = \frac{S_{ad}^2}{S_{\bar{y}}^2} \quad (26)$$

and compare it with the table value *F* [15] for degrees of freedom with which were defined S_{ad}^2 and $S_{\bar{y}}^2$, that is

$$f_{ad} = N_1 - m, \quad (27)$$

$$f_{\bar{y}} = n_0 - 1. \quad (28)$$

In our case $F_p < F$, therefore, the equation is considered suitable for calculations.

The resulting equation (17) connects tire pressures at different ambient temperatures and the required inflating pressure at a given temperature of the inflating air.

The designations $P_o = y$, $P_i = x_1$, $t_i = x_2$, $t_o = x_3$, are introduced, and the values of x_1, x_2, x_3 from the formulas (2 – 4) are substituted into (17). Then the equation of tire pressure dependence P_o from the named factors in the natural form is obtained after simple mathematical transformations:

$$\begin{aligned} P_o &= 5.69 + 0.971 \cdot P_i - 0.53 \cdot t_i + 0{,}027 \cdot t_o + \\ &+ 0.0035 \cdot P_i \cdot t_o \end{aligned}. \quad (29)$$

Since, in the conditions of service stations, the practical interest is not so much the pressure in the tire depending on the ambient temperature, but the required pumping pressure depending on the temperature when the pump is carried out. So the value of the pump pressure P_i, kPa, is set from the equation (29):

$$P_i = \frac{P_o - 5.69 + 0.53 \cdot t_i - 0.027 \cdot t_o}{0.971 + 0.0035 \cdot t_o}. \quad (30)$$

Conclusion

As a result of the undertaken research, mathematical relationship between pressure in the pneumatic tire at the operating temperature and the required pressure of inflating air into the tire, if the inflating and operation temperatures differ, has been obtained.

Using the given equation at the known temperature in the room, where the inflation is made, and at the known temperature of the environment, where the tire is operated and at the pressure in the car tire, recommended by the production plant, it is possible to determine the required pressure of air inflation.

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COMPRESSED ELEMENTS WITH A VARIABLE IN LENGTH STIFFNESS EQUILIBRIUM FORM STABILITY DETERMINATION

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One of the most powerful modern methods of calculating complex building structures is the finite element method in the form of a displacement method for discrete systems, which involves the creation of a finite element model, that is, splitting the structure into separate elements within each of which the functions of displacements and stresses are known. On the basis of the displacement method and the methods of iterations and half-division, an algorithm for stability calculation of the first kind equilibrium form of compressed reinforced concrete columns with hinged fixing at the ends, considering the stiffness changing has been developed. The use of the above methods enables to determine the minimum critical load or stress at the first bifurcation and their stability loss corresponding form. The use of matrix forms contributes to simplification of high order stability loss equation. This approach enables to obtain the form of stability loss that corresponds to the critical load.

Keywords: equilibrium form, compressed reinforced concrete columns, software complex, stability loss equation

ВИЗНАЧЕННЯ ФОРМИ РІВНОВАГИ СТИСНУТИХ ЗАЛІЗОБЕТОННИХ ЕЛЕМЕНТІВ ЗІ ЗМІННОЮ ПО ДОВЖИНІ ЖОРСТКІСТЮ

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Одним з найбільш потужних сучасних методів розрахунку складних будівельних конструкцій є метод скінченних елементів в формі методу переміщень для дискретних систем, який передбачає створення скінченно-елементної моделі, тобто розбиття конструкції на окремі елементи у межах кожного з яких відомі функції переміщень та напружень. Метод скінченних елементів також передбачає формування та розв’язування системи лінійних алгебраїчних рівнянь порядок якої обумовлений кількістю невідомих вузлових переміщень. Розроблено алгоритм розрахунку стійкості першого роду форми рівноваги стиснутих залізобетонних колон з шарнірним закріпленням на кінцях, з урахуванням зміни жорсткості, на основі методу переміщень разом з методами ітерацій і повинного ділення. Використання вищеописаних методів дозволяє визначити мінімальне критичне навантаження або напруження при першій біфуркації та їх втрати стійкості відповідної форми. Використання матричних форм сприяє спрощенню рівняння втрат стійкості високого порядку. Такий підхід дозволяє отримати форму втрат стійкості, що відповідає критичному навантаженню. Описані вище особливості розрахунку таких конструкцій обумовлюють необхідність у використанні потужностей сучасних ПК, що у свою чергу потребує автоматизації розрахункових процесів у вигляді закінчених програмних комплексів та систем. На базі кафедри будівельної та теоретичної механіки Полтавського національного технічного університету імені Юрія Кондратюка були розроблені програмні утиліти, які реалізують взаємопов’язані задачі оцінювання міцності та надійності будівельних конструкцій та їх елементів.

Ключові слова: форма рівноваги, стиснуті залізобетонні колони, програмний комплекс, рівняння втрати стійкості.

Introduction

There are various methods for calculating the stability of the equilibrium form of discrete systems, due to the large volume of computations associated with the solution of the analytical condition for the equilibrium stability loss. The solution of the analytical condition for the compressed discrete systems equilibrium stability loss, which has high orders, and the stability loss form critical load definition, are very important problems to be solved.

The algorithm for calculation of equilibrium stability loss form of compressed discrete systems of the first kind by the displacement method in combination with the methods of iterations and bisection with examples is given in [6]. It is implemented in the software package "Persist" for a PC in Windows OS.

To solve engineering tasks this computer program is implemented in a modern compiler, contains several subroutines that are combined and presented in the form of the same software complex.

Review of research sources and publications

The solution of the problem of calculating the analytical condition for the compressed discrete systems equilibrium stability loss, which has high orders, and the determination of the corresponding stability loss form critical load, generated a large number of methods by many mathematicians (Krylov, Laverier, Danilevsky, Jacobi (iterations) etc.).

Definition of unsolved aspects of the problem

When calculating the first kind equilibrium stability form of compressed reinforced concrete columns with hinged fixing at the ends, considering the stiffness changing (damage to the column sections) provided that the initial modulus of elasticity is constant; it is assumed the necessity of solving the stability loss equation which is non-linear transcendental and, as it is known, does not have an analytical solution [1 – 4, 7 – 12]. In addition, these equation elements are complex mathematical dependencies, which contain Zhukovsky functions in their composition, which also greatly complicates the equation solution.

The purpose of the work is to develop an algorithm and software for the PC in Windows OS, which enables students and engineers to automate calculations of stability of equilibrium forms of compressed discrete systems.

In recent years, the software complex "Persist" has been tested and successfully implemented in the training of specialists for the building industry [5, 6].

Problem statement

The calculation of the compressed discrete system on the stability of the equilibrium form actually reduces to the solution of the difficultly described nonlinear transcendental equation, which is the equation of stability loss. The difficulty lies in the absence of analytical solution of such equation due to the presence of complex functions of Zhukovsky, which have transcendental functions in their structure. Such solution can be performed only with the use of numerical methods. This problem of calculating the analytical condition for compressed discrete systems equilibrium stability loss, as well as the determination of the critical load of the stability form, is proposed to be solved by displacement, iteration and bisection methods, which enables to significantly simplify calculations.

Basic material and results

It has been obtained stability loss equation of the equilibrium form of reinforced concrete columns with hinged fixing at the ends considering stiffness changing due to the displacement's method in expanded form provided that compression and stretch deformations are ignored [5, 6, 13].

Output data:

$$h_1 = m_1\, l\,;\ h_2 = m_2\, l\,;\ h_3 = m_3\, l\,;\ h_4 = m_4\, l\,;$$

$$E = \text{const} \neq \infty\,;$$

$$i_j = \left(\frac{EJ}{l}\right)_j \left(j = \overline{1.4}\right).$$

It has been calculated: the value of the minimum critical longitudinal force $N_{cr}^{\min}$ at stability loss of the first kind equilibrium form considering damages on any column sections (Figure 1).

It has been accepted, $i = i_{01}$, $J = J_{01}$ and it has been expressed all rigid stiffness on the bend of the column sections due to these parameters and the first section length $h_1 = m_1\, l$.

$$i_{01} = \frac{EJ_{01}}{h_1};\quad i_{12} = \frac{EJ_{12}}{h_2}; \tag{1}$$

$$i_{23} = \frac{EJ_{23}}{h_3};\quad i_{34} = \frac{EJ_{34}}{h_4}\ ,$$

where

$$J_{01} = J;\quad J_{12} = C_2\, J;\quad J_{23} = C_3\, J;\quad J_{34} = C_4\, J. \tag{2}$$

The values of the coefficients of the rigid stiffness ratio on bend, expressed through the first section rigid stiffness, are equal to:

$$K_{01} = \frac{i_{01}}{i} = 1\,;\qquad K_{12} = \frac{i_{12}}{i} = \frac{C_2 m_1}{m_2}\,; \tag{3}$$

$$K_{23} = \frac{i_{23}}{i} = \frac{C_3 m_1}{m_3}\,;\quad K_{34} = \frac{i_{34}}{i} = \frac{C_4 m_1}{m_4}\ .$$

The rigid stiffness on each of the four sections bend is expressed as:

$$J_{01} = J;\quad i_{01} = K_{01}\, i;\quad i_{12} = K_{12}\, i\,; \tag{4}$$

$$i_{23} = K_{23}\, i;\quad i_{34} = K_{34}\, i\,.$$

Considering the above expressions, the rigid stiffness final value on the bend of each of the four sections is:

$$i_{01} = \frac{EJ_{01}}{m_1 l};\quad i_{12} = \frac{EJ_{12}}{m_2 l}; \tag{5}$$

$$i_{23} = \frac{EJ_{23}}{m_3 l};\quad i_{34} = \frac{EJ_{34}}{m_4 l}\ .$$

Thus, this approach enables to change the flexural stiffness of each section of the column (see calculation scheme, Figure 1) by varying the moments of inertia at the corresponding axes, provided that the initial elastic modulus remains constant.

Determination of the minimum critical force ($N_{cr}^{\min}$) is obtained using the displacement method expanded form [5, 6, 9].

It is recorded the displacement method equilibrium equation in accordance with the column calculation scheme (Figure 1).

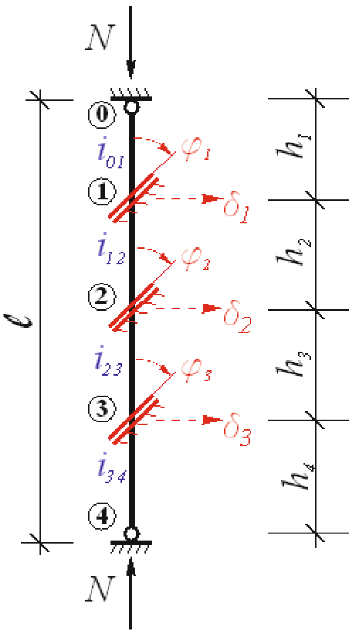

Figure 1 – The column calculation scheme

Displacement method equilibrium equations takes the form (Figure 2).

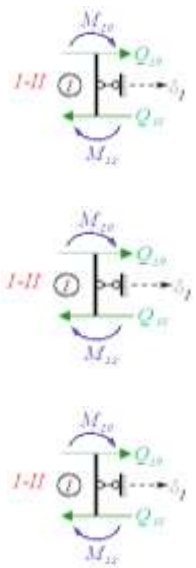

Figure 2 – Calculation schemes of the column nodes

$$\begin{cases} M_{10} + M_{12} = 0; \\ Q_{10} - Q_{12} = 0; \\ M_{21} + M_{23} = 0; \\ Q_{21} - Q_{23} = 0; \\ M_{32} + M_{34} = 0; \\ Q_{32} - Q_{34} = 0; \end{cases} \quad (6)$$

The same equations are recorded in a matrix form:

$$r \cdot \vec{Z} = 0\,, \quad (7)$$

where r – stiffness matrix of column sections elements (reactive forces in fictitious joints from unknown vector $\vec{Z}$ unit values)

$$r = \begin{pmatrix} r_{11} & r_{12} & r_{13} & r_{14} & r_{15} & r_{16} \\ r_{21} & r_{22} & r_{23} & r_{24} & r_{25} & r_{26} \\ r_{31} & r_{32} & r_{33} & r_{34} & r_{35} & r_{36} \\ r_{41} & r_{42} & r_{43} & r_{44} & r_{45} & r_{46} \\ r_{51} & r_{52} & r_{53} & r_{54} & r_{55} & r_{56} \\ r_{61} & r_{62} & r_{63} & r_{64} & r_{65} & r_{66} \end{pmatrix}. \quad (8)$$

According to the reciprocity of reactions theorem $r_{ik} = r_{ki}$ $(i,k = \overline{1,6})$. Therefore, the stiffness matrix is represented only by the values of its upper triangle parameters.

$$r = \begin{pmatrix} r_{11} & r_{12} & r_{13} & r_{14} & r_{15} & r_{16} \\ & r_{22} & r_{23} & r_{24} & r_{25} & r_{26} \\ & & r_{33} & r_{34} & r_{35} & r_{36} \\ & & & r_{44} & r_{45} & r_{46} \\ & & & & r_{55} & r_{56} \\ & & & & & r_{66} \end{pmatrix}. \quad (9)$$

$\vec{Z}$ – vector of the displacement method unknown in accordance with the column calculation scheme (Figure 1)

$$\vec{Z} = \begin{Bmatrix} Z_1 \\ Z_2 \\ Z_3 \\ Z_4 \\ Z_5 \\ Z_6 \end{Bmatrix} = \begin{Bmatrix} \varphi_1 \\ \varphi_2 \\ \varphi_3 \\ \varphi_4 \\ \varphi_5 \\ \varphi_6 \end{Bmatrix}. \quad (10)$$

The system of equilibrium equations (6) is a system of linear algebraic homogeneous equations that has the following properties:

a) the system has a zero (trivial) solution (the physical content of the vector $\vec{Z}$ =0, that is, the column maintains a straightforward form – there is no equilibrium form stability loss);

b) the system has no zero solution if det(r) = D = 0. In this case, there is a problem of eigenvalues, that is, system (6) has many solution roots. Physical content: vector $\vec{Z} \neq 0$ (there are angular and linear displacements of the system nodes, indicating the appearance of a new deformation form – the longitudinal bending, i.e., the system loses the equilibrium form stability (the first kind stability loss). Thus, det(r) there is an analytical condition for the equilibrium stability loss (stability loss equation), the solution of which is able to determine $N_{cr}^{\min}$ with the methods of iterations and half-division.

Compressed column equilibrium form stability loss equation has the following general form

$$\det(r) = D = \begin{vmatrix} r_{11} & r_{12} & r_{13} & r_{14} & r_{15} & r_{16} \\ & r_{22} & r_{23} & r_{24} & r_{25} & r_{26} \\ & & r_{33} & r_{34} & r_{35} & r_{36} \\ & & & r_{44} & r_{45} & r_{46} \\ & & & & r_{55} & r_{56} \\ & & & & & r_{66} \end{vmatrix} = 0 .$$

(11)

Equilibrium form stability loss equation (11) is a transcendental equation, that has many solution roots. To determine the smallest value of the critical load (critical longitudinal force $N_{cr}^{\min}$), numerical methods of iterations and half-division are used.

In the equation (11), the compressive force N is an unknown value and is expressed by Zhukovsky functions ($\alpha, \beta, \gamma, \bar{\alpha}, \bar{\gamma}, ...$), which are expressed through the parameter t, which in turn is expressed in longitudinal force N terms.

By preliminary calculating the relationship between the parameters t in the compressed column through the base parameter t_0, it is determined the critical parameters t in each compressed element, and then their critical longitudinal forces and critical load.

$$t_j^{\,2} = \left(\frac{Nl^2}{EI}\right)_j ; \quad t_j^{\,2} = \left(\frac{Nl}{i}\right)_j ; \tag{12}$$

$$N_{j,cr} = \left(\frac{t^2_{cr} EI}{l^2}\right)_j = \left(\frac{t^2_{cr} \cdot i}{l}\right)_j . \tag{13}$$

In this case, the critical forces in each section of the column should be the same. It is $N_{cr}^{\min}$ for the column.

The equation (14) is calculated if the stiffness of the system elements changes. To do this, it is needed to set the appropriate correlation of the relationship parameters: m_1, m_2, m_3, m_4 and C_1, C_2, C_3, C_4.

So, for the given calculation scheme of the column (Figure 1), the relationship between the parameters t of each section is:

As the base parameter it is taken $t = t_{01}$, then:

$$D = \begin{vmatrix} \bar{\alpha}_{01} + \frac{2C_2m_1}{m_2}\alpha_{12} & \left(\frac{1}{m_1}\bar{\alpha}_{01} - \frac{2C_2m_1}{m_2^2}(\alpha+\beta)_{12}\right) & \frac{2C_2m_1}{m_2}\beta_{12} & \frac{2C_2m_1}{m_2^2}(\alpha+\beta)_{12} & 0 & 0 \\ & \left(\frac{1}{m_1^2}\bar{\gamma}_{01} + \frac{2C_2m_1}{m_2^3}\gamma_{12}\right) & -\left(\frac{2C_2m_1}{m_2^2}(\alpha+\beta)_{12}\right) & \frac{2C_2m_1}{m_2^3}\gamma_{12} & 0 & 0 \\ & & \frac{2C_2m_1}{m_2}\alpha_{12} + \frac{2C_3m_1}{m_3}\alpha_{23} & \frac{2C_2m_1}{m_2^3}\gamma_{12} + \frac{2C_3m_1}{m_3^3}\gamma_{23} & \frac{2C_3m_1}{m_3}\beta_{23} & \frac{2C_3m_1}{m_3^2}(\alpha+\beta)_{23} \\ & & & \frac{2C_2m_1}{m_2^3}\gamma_{12} + \frac{2C_3m_1}{m_3^3}\gamma_{23} & \frac{2C_3m_1}{m_3^2}(\alpha+\beta)_{23} & \frac{2C_3m_1}{m_3^3}\gamma_{23} \\ & & & & \frac{C_4m_1}{m_4}\bar{\alpha}_{34} + \frac{2C_3m_1}{m_3}\alpha_{23} & \frac{C_4m_1}{m_4^2}\bar{\alpha}_{34} - \frac{2C_3m_1}{m_3^2}(\alpha+\beta)_{23} \\ & & & & & \frac{C_4m_1}{m_4^3}\bar{\gamma}_{34} + \frac{2C_3m_1}{m_3^3}\gamma_{23} \end{vmatrix} = 0 \tag{14}$$

$$t = t_{01} = t_1 = \sqrt{\frac{N \cdot h_1}{i_{01}}} = \sqrt{\frac{N \cdot m_1 \cdot l}{i}} \quad ; \tag{15}$$

$$t_{12} = t_2 = \sqrt{\frac{N \cdot h_2}{K_{12} \cdot i}} = \sqrt{\frac{N \cdot m_2^2 \cdot l}{C_2 \cdot m_1 \cdot i}} \quad ; \tag{16}$$

$$t_{23} = t_3 = \sqrt{\frac{N \cdot h_3}{K_{23} \cdot i}} = \sqrt{\frac{N \cdot m_3^2 \cdot l}{C_3 \cdot m_1 \cdot i}} \quad ; \tag{17}$$

$$t_{34} = t_4 = \sqrt{\frac{N \cdot h_4}{K_{34} \cdot i}} = \sqrt{\frac{N \cdot m_4^2 \cdot l}{C_4 \cdot m_1 \cdot i}} \quad ; \tag{18}$$

$$\zeta_1 = \frac{t_{01}}{t} = 1. \quad ; \tag{19}$$

$$\zeta_2 = \frac{t_{12}}{t} = \frac{t_2}{t} = \frac{m_2}{m_1}\sqrt{\frac{1}{C_2}} \quad ; \tag{20}$$

$$\zeta_3 = \frac{t_{23}}{t} = \frac{t_3}{t} = \frac{m_3}{m_1}\sqrt{\frac{1}{C_3}} \quad ; \tag{21}$$

$$\zeta_4 = \frac{t_{34}}{t} = \frac{t_4}{t} = \frac{m_4}{m_1}\sqrt{\frac{1}{C_4}} \quad . \tag{22}$$

Thus,

$$t_{01} = t_1;\ t_{12} = \zeta_2 \cdot t;\ t_{23} = \zeta_3 \cdot t;\ t_{34} = \zeta_4 \cdot t \quad . \tag{23}$$

To determine $N_{cr}^{\min}$ (calculation of the equation of stability loss of equilibrium form (4)) it is used the software complex "Persist" specially developed by the authors for the corresponding output data (m_j, C_j, ζ_j), which algorithm is based on numerical methods of iterations and half-division [5, 6]. The iteration method enables to determine the subinterval with the minimum value of the base critical parameter t_{cr}, and the half-division method to specify its value to the predefined accuracy.

At the same time, since everything was expressed due to the basic rigid stiffness *i* and the length of the rod l, the final value $N_{cr}^{\min}$ should be multiplied by the rigid stiffness *i* and divided by the length of the rod l.

It is considered a calculation example, when the stiffness of all column elements is constant (Figure 1). In this case, the source data are:

$$\begin{gathered} h_1 = h_2 = h_3 = h_4 = \frac{l}{4};\ E = const \neq \infty; \\ J = J_{01} = J_{12} = J_{23} = J_{34};\ i = i_{01} = i_{12} = i_{23} = i_{34} \end{gathered} \tag{24}$$

$$C_1 = C_2 = C_3 = C_4 = 1;\ m_1 = m_2 = m_3 = m_4 = \frac{1}{4} \quad . \tag{25}$$

The relation between the parameters t are written as:

$$t_0 = t_{01} = t_{12} = t_{23} = t_{34}, \tag{26}$$

$$\zeta_1 = \zeta_2 = \zeta_3 = \zeta_4 = 1. \tag{27}$$

The stability loss equation for such output data is:

$$D = \begin{vmatrix} \bar{\alpha}_{01} + 2\alpha_{12} & 4\bar{\alpha}_{01} - 8(\alpha+\beta)_{12} & 2\beta_{12} & 8(\alpha+\beta)_{12} & 0 & 0 \\ & 16\bar{\gamma}_{01} + 32\gamma_{12} & -8(\alpha+\beta)_{12} & -32\gamma_{12} & 0 & 0 \\ & & 2\alpha_{12} + 2\alpha_{23} & 8(\alpha+\beta)_{12} - 8(\alpha+\beta)_{23} & 2\beta_{23} & 8(\alpha+\beta)_{12} \\ & & & 32\gamma_{12} + 32\gamma_{23} & -8(\alpha+\beta)_{23} & -32\gamma_{23} \\ & & & & \bar{\alpha}_{34} + 2\alpha_{23} & -4\bar{\alpha}_{34} + 8(\alpha+\beta)_{23} \\ & & & & & 16\bar{\gamma}_{34} + 32\gamma_{23} \end{vmatrix} = 0 \tag{28}$$

To further calculating the column for the equilibrium form stability we use the software complex "Persist" (Figure 3–10).

Here is a step-by-step algorithm for entering input data and necessary parameters for the software complex "Persist":

1. It is entered the relation for the parameter t :

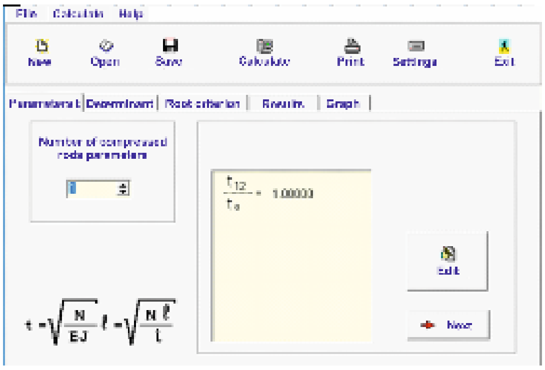

Figure 3 – Recording correlations for parameters *t*

2. It is entered the elements of the upper triangle of equilibrium stability loss equation (28).

For example, it is entered an element r_{11}. To do this, it is necessary to select element 1.1, and press the "Build" button:

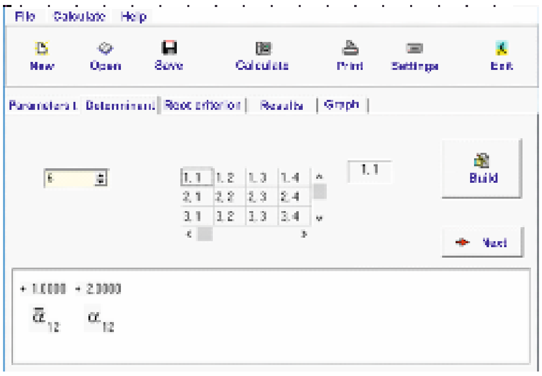

Figure 4 – Entering values for an element r_{11}

At the next step, it is entered the expression value for the element in the menu item "Determinant", the button "Build". Similarly, it is introduced all other elements of the upper triangle (28).

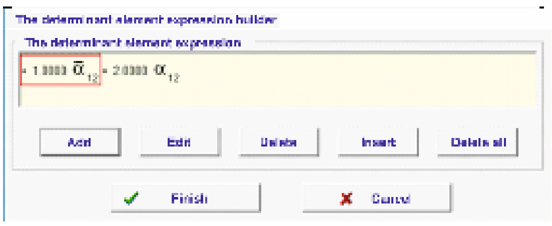

Figure 5 – Writing the expression for the element r_{11}

Expression for an element r_{66}

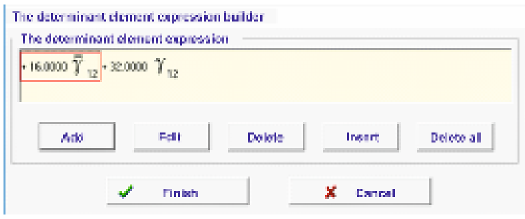

Figure 6 – Writing the expression for the element r_{66}

3. It is chosen the desired accuracy for calculating the root of the stability loss equation t_0 (the determinant minimum value, 10^{-4}). Also, it is set the base parameter initial value (0), and also basic parameter to changing step t_0 (0,1). If necessary, this step can be reduced.

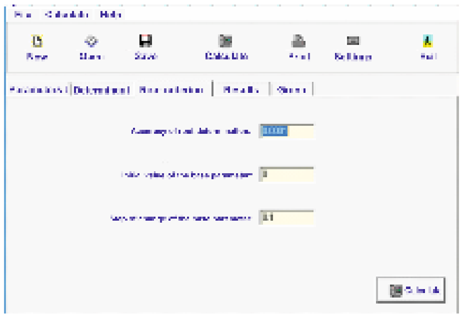

Figure 7 – Configuring accuracy for calculate the stability loss equation root

4. In the menu bar, press the "Calculate" button and go to the "Results" tab:

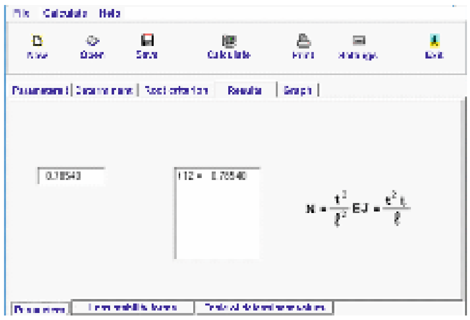

Figure 8 – View of the calculation results

If necessary, activate the tab "Table of determinant values", and it can be viewed step-by-step calculation results by iteration and half-division methods:

Figure 9 – Table of determinant values

If necessary, by activating the "Graph" tab, it can be viewed a step-by-step graphic representation of the stability loss equation solution (28):

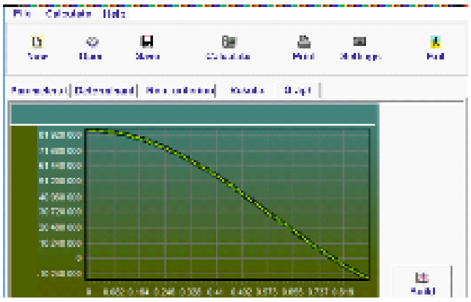

Figure 10 – Graphical representation of the stability loss equation solution

This $N_{cr}^{\min}$ is equal to:

$$N_{cr,j}^{\min} = \left(\frac{(t_{cr}\cdot n)^2\cdot i}{\ell}\right) = \frac{(0,7854\cdot 4)^2\cdot i}{\ell} = \frac{3{,}1416^2\cdot i}{\ell}.$$

It should be noted that the dimension of the critical force depends on the given dimensions of the stiffening on bend *i* and the rod length *l*.

Conclusions

The method of calculation and algorithm is developed, which are implemented in the software complex "Persist". The program com-plex has been successfully implemented in the educational process in the study of the discipline "Structural Mechanics" at the educational-scientific Architecture and Civil Engineering Institute of Poltava National Technical Yuri Kondratyuk University. This software can be used by students and engineer-designers for engineering calculations, including calculating the first kind equilibrium form stability of compressed reinforced concrete columns with hinged fixing at the ends, considering the stiffness changing.

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DESIGN OF COMPOSITE SKIN PANEL FOR ROOF IN ACCORDANCE WITH THE REQUIREMENTS OF EN 1995-1-1 AND GSN B.1.2-2:2006

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In the article design of composite (timber and plywood) skin panel has been considered according European and national standards. The panel consists of timber webs and plywood skin. The design parameters are depth of webs and distance between webs within the defined skin parameters. In design the method of fictitious cross-section is applied. Check of the ultimate limit states is conducted in accordance with EN 1995-1-1 (ДБН В.2.6-161:2017). Loads to the panel are determined according to ДБН В.1.2.-2:2006. The usage of European and national standards complies with the actual requirements and widen methodological base.

Keywords: roof members; two-sided composite skin panel, design of building constructions, loads on building constructions

ПРОЕКТУВАННЯ КЛЕЄФАНЕРНОЇ ПАНЕЛІ ПОКРИТТЯ ВІДПОВІДНО ДО ВИМОГ EN 1995-1-1 і ДБН В.1.2-2:2006

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Розглянуто проектування композитної панелі покриття згідно з європейськими і національними будівельними нормами. Панелі композитного типу (дерев’яні ребра й обшивка з фанери або OSB) часто застосовуються для покриття легкого типу. Для конструкцій, які мають у своєму складі матеріали з різними фізико-механічними властивостями, при проектуванні застосовується метод зведеного перерізу, що значно спрощує розрахунки. Оскільки обшивки панелі знаходяться в більш напруженому стані, ніж ребра, матеріал ребра (деревина) приводиться до матеріалу обшивок за допомогою коефіцієнта приведення. Він знаходиться як відношення модулів пружності матеріалів. Як приклад у статті наведено конструктивний розрахунок за першим граничним станом. Остання редакція національних норм щодо розрахунку дерев’яних конструкцій повністю базується на європейських будівельних стандартах. Із цих міркувань у статті розглянуто проектування конструкції відповідно до оригінального джерела. Натомість визначення характеристик міцності рекомендовано виконувати згідно з національними нормами, які одночасно виконують функції європейського стандарту EN 338. Навантаження на панель визначаються відповідно до норм, що є не тільки традиційним в національній методології проектування будівельних конструкцій, але й, на думку авторів, більше відповідає критеріям надійності. Застосування в розрахунках одночасно національних і європейських проектних норм відповідає сучасним вимогам та розширює методологічну базу.

Ключові слова: клеєфанерна панель покриття, проектування будівельних конструкцій, навантаження на будівельні конструкції.

Introduction

Nowadays new building codes adapted to European ones are used in Ukraine. There is a need to adapt the design methodology for the elements of timber structures.

Two-sided composite (timber and plywood or OSB) skin panel are most common. This type of panels is designed as a typical glued thin flanged internal I-beam with flanges on the top and bottom faces and subjected to a moment.

Review of research sources and publications

Design methods of composite skin panels for products in USA, Canada, and Europe are considered in publications [1 – 4]. As the flanges are thin, the stress in each flange due to bending is effectively an axial stress and the design value is taken to be the average value across the flange thickness. In calculating section properties for stressed-skin panels, the designer must consider the composite nature of the unit. Unless all materials in the panel have similar moduli of elasticity, some method must be employed to make allowance for the differences. Different moduli of elasticity may be reconciled by the «transformed section» use. The transformed-section approach is common to structural design of composite sections. It consists of «transforming» the actual section into one of equivalent strength and stiffness, but composed of a single material. Sections are generally transformed to the material of the panel most highly stressed portion.

The design requirements for the web are that it must be able to support the flexural stresses that arise, that the shear stress in the web must be acceptable, and that the glued joints between the web and the flanges must be able to transfer the horizontal shear stresses at the interface.

Definition of unsolved aspects of the problem

European design recommendations are based on the Eurocode load standards [5, 6]. Loads for building Ukrainian constructions can be used according rules of Ukrainian standard [7]. Dead loads determined rules of *chapter 5* of [7]. Snow (variable) loads determined rules of *chapter 8* of [7].

Problem statement

In the article the possibility of using the Ukrainian standards for loads on building constructions and European standards for design timber structure elements have been discussed.

Basic material and results

It is considered the composite skin panel design in accordance with the requirements [8]. Typical two-sided composite (timber and plywood) skin panel is used as a roof member for different types of roofing materials. Plywood is used for panel top and bottom skins and timber is used for panel webs.

For composite skin panel design it can be used rules of Ukrainian standard [9] which is based on rules [8]. In Ukraine characteristic values of strength classes according [9] should be used.

The effective flange width concept applies to flanges in compression and in tension, and unless a more detailed calculation is carried out, in accordance with the requirements of 9.1.2(3) [8]. The effective flange width, b_{ef}, as shown in Figure 1, for internal I-shaped sections, is as follows ([8] *eq. (9.12)*):

$b_{ef.c} = b_{c.ef} + b_w$ and $b_{ef.t} = b_{t.ef} + b_w$.

Effective flange width of an I-beam section of the panel ([8], *9.1.2*):

– in compression ([8], *Table 9.1*),

$b_{c.ef} \le min\{ 0{,}1 \cdot L \ ; \ 20 \cdot \delta_{tf} \};$

– in tension ([8], *Table 9.1*),

$b_{t.ef} \le 0{,}1 \cdot L$ (only shear lag).

Designations accepted in the formulas:

$b_{c.ef}$ – design width of the flange in compression;
$b_{t.ef}$ – design width of the flange in tension;
b_w – design width of the web;
L – span of panel;
δ_{tf} – top flange thickness;
δ_{bf} – bottom flange thickness.

The values $b_{c.ef}$ and $b_{t.ef}$ should not be greater than clear distance between webs b_f .

Panel has a cross-section of two different materials. It is convenient to go transformed section or fictitious section.

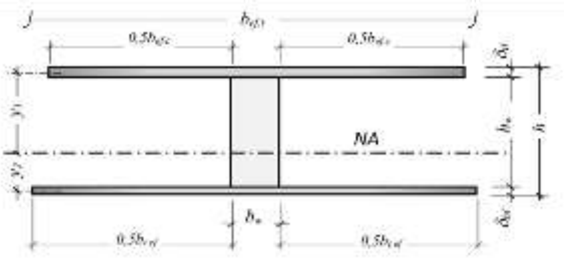

Figure 1 – Transform cross-section as I-beam

Coefficient for transformed section, as ratio of modulus

$$n_E = E_{0.mean} / E_{plw.0.mean} \ . \qquad (1)$$

Transformed web thickness (into plywood)

$$b_{w.tfd} = b_w \cdot n_E \ .$$

Areas of cross-sections flanges:

– flange in compression

$$A_{ef.f.c} = b_{ef.c} \cdot \delta_{tf} \ ; \qquad (2)$$

– flange in tension

$$A_{ef.f.t} = b_{ef.t} \cdot \delta_{bf} \ . \qquad (3)$$

Area of the webs

$$A_{ef.w} = b_{w.tfd} \cdot h_w \ . \qquad (4)$$

Transformed area

$$A_{ef} = A_{ef.f.c} + A_{ef.f.t} + A_w \ . \qquad (5)$$

First moment of area of the section about the top face:

$$A_{1st} = A_{ef.f.t} \, (h - \delta_{bt} /2) + A_{ef.w} \, (h_w/2 + \delta_{tf}) + A_{ef.f.c} \, \delta_{tf} /2 \ . \qquad (6)$$

Neutral axis (*NA*) depth from the top face

$$y_t = A_{1st} / A_{ef} \ . \qquad (7)$$

Second moment of area of the web about the *NA*

$$I_{ef.w}=\frac{b_{w.tfd}\cdot h_w^3}{12}+A_{ef.w}\cdot(y_t-(\delta_{tf}+\frac{h_w}{2}))^2\,, \qquad (8)$$

Second moment of area of the top flange about the *NA*

$$I_{ef.tf}=\frac{b_{ef.c}\cdot\delta_{tf}^3}{12}+A_{ef.f.c}\cdot(y_t-\frac{\delta_{tf}}{2})^2\,. \qquad (9)$$

Second moment of area of the bottom flange about the *NA*

$$I_{ef.bf}=\frac{b_{ef.t}\cdot\delta_{bf}^3}{12}+A_{ef.f.t}\cdot(h-y_t-\frac{\delta_{bf}}{2})^2 \qquad (10)$$

Instantaneous second moment of the transformed section area

$$I_{ef}=I_{ef.w}+I_{ef.tf}+I_{ef.bf}\,. \qquad (11)$$

Stress in the flanges due to bending:

– bending stress (compression) in the top flange

$$\sigma_{f.c.max.d}=\frac{M_d}{I_{ef}}\cdot(y_t-\frac{\delta_{tf}}{2})\,; \qquad (12)$$

– bending stress (tension) in the bottom flange

$$\sigma_{f.t.max.d}=\frac{M_d}{I_{ef}}\cdot(h-y_t-\frac{\delta_{bf}}{2})\,; \qquad (13)$$

– bending stress check in the web

$$\sigma_{w.c.d}=\frac{M_d}{I_{ef}}\cdot y_1\cdot n_E\,, \qquad (14)$$

where $y_1=max\{(y_t-\delta_{tf});\ (h-\delta_{bf}-y_t)\}$ – maximum distance from the *NA* to the extreme fibre.

Shear stress at the NA position:

$$\tau_{v.d}=\frac{V_d\cdot S_{tf.NA}}{I_{ef}\cdot b_{w.tfd}}\cdot n_E\,, \qquad (15)$$

where $S_{tf.NA}$ – first moment of area of the section above the *NA*

$$S_{tf.NA}=b_{ef.c}\cdot\delta_{tf}\cdot(y_t-\frac{\delta_{tf}}{2})+b_{w.tfd}\cdot\frac{(y_t-\delta_{tf})^2}{2}\,. \qquad (16)$$

Shear stress of the glued joint between the web and the flanges:

– first moment of top flange area above the *NA*

$$S_{tf}=b_{ef.c}\cdot\delta_{tf}\cdot(y_t-\frac{\delta_{tf}}{2})\ ; \qquad (17)$$

– first moment of the bottom flange area about *NA*

$$S_{bf}=b_{ef.t}\cdot\delta_{bf}\cdot(h-y_t-\frac{\delta_{bf}}{2})\,; \qquad (18)$$

Maximum value of first moment of area about *NA*

$S_f=max\{S_{tf};\ S_{bf}\};$

Mean shear stress in the flange across the glue line

$$\tau_{mean.d}=\frac{V_d\cdot S_f}{I_{ef}\cdot b_{w.tfd}}\cdot n_E\,. \qquad (19)$$

Design example of composite skin panel. Top skin of panel is plywood with thickness δ_{tf} = 9 mm, bottom skin of panel is plywood with thickness δ_{bf} = 6 m. Strength class of plywood *F20/10 E40/20* with the faces aligned parallel to the direction of span [9]. The timber used for the web is class *C22* [9]. Panel is glued between the flanges and the web. Spanning between two supports *L*=4,5 m apart. Nominal wide of panel – 1,5 m. Construction sizes for panel are 448×149 cm. The structure functions in service class 2 conditions.

The design of panel complies with the rules in [8] at the ULS and [7] (loads).

Dead loads to the panel - weight of materials.

Table 1 – Dead load to the 1 m² panel

Materials	Characteristic load, q^0, кN/m²	Exploitation load, q^e, кN/m²	Safety factor, γ_M	Design load, *q*, кN/m²
1. Profile Steel Roofing Sheets	0,15	0,15	1,3	0,195
2. Plywood skin	0,105	0,105	1,1	0,115
3. Timber web	0,078	0,078	1,1	0,086
4. Mineral wool in panel	0,060	0,060	1,2	0,072
5. PE steam insulation	0,005	0,005	1,1	0,006
Total dead load for panel – q_p	0,398	0,398		0,474

Variable loads to the panel – snow load may be set by National Norm [7]

Design snow load and exploitation snow load:

$S_m=\gamma_{fn}S_0C$= 1,04·1450·1 = 1508 Pa = 1,508 kN/m²;

$S_e=\gamma_{fe}S_0C$= 0,49·1450·1 = 710,5 Pa = 0,71 kN/m²,

where S_0 =1450 Pa – characteristic snow load for Poltava ([7], *Annex A*).

Other values in these formulas are calculated according ([7].

Total loads to the panel

Loads for 1 meter of panel span

Total design loads for 1 meter of panel span (used table 2):

$q_l=q_{sum}\cdot B,$

where *B=1,5 m* – wide of panel.

Table 2 – Total loads to the 1 m² panel

Type of load	Characteristic load, q^0+S_0,	Exploitation load, q^e+S_e, kN /m²	Design load, q_p+S_m, kN /m²
1. Dead load to the panel	0,398	0,398	0,474
2. Variable loads to the panel (snow)	1,450	0,710	1,508
Total loads for panel – q_{sum}	1,848	1,108	1,982

Exploitation load

q^e_l= 1,108·1,5 = 1,662 kN/m.

Design load

q_l= 1,982·1,5 = 2,973 kN/m.

Actions. Design bending moment and design shear force

$M_d = q_l\, l^2 / 8$ = 2,973·4,42²/8 =
= 7,26 kN·m = 726 kN·cm;

$V_d = 0{,}5\ q_l\ l = 0{,}5 \cdot 2{,}973 \cdot 4{,}42 = 6{,}57$ kN,

where l = 4,48–0,06 = 4,42 m – design span calculated with support effect.

Modification factor for permanent duration action ([8], *eq .2.6*):

$k_{\text{mod.perm}} = (k_{\text{mod.perm1}} \cdot k_{\text{mod.perm2}})^{0,5}$ =

= $(0{,}6 \cdot 0{,}6)^{0,5} = 0{,}6$,

where $k_{\text{mod.perm1}}$ = 0,6 for solid timber (webs) and *service class 2* ([8], *Table 3.1*);

$k_{\text{mod.perm2}}$ = 0,6 for plywood (flanges) and *service class 2* ([8], *Table 3.1*).

Factor for medium-duration action ([8], *eq. 2.6*):

$k_{\text{mod.medium}} = (k_{\text{mod.medium1}}\ k_{\text{mod.medium2}})^{0,5}$ =

= $(0{,}8 \cdot 0{,}8)^{0,5} = 0{,}8$,

where $k_{\text{mod.medium1}}$ = 0,8 for solid timber (webs) and *service class 2* ([8], *Table 3.1*);

$k_{\text{mod.medium2}}$ = 0,8 for plywood (flanges) and *service class 2* ([8], *Table 3.1*).

Load sharing factor, k_{sys} = 1,0 ([8], 6.6) (k_{sys} = 1,1 can be used if it is required).

Depth factor for solid timber (webs) – is taken as $k_h = 1$, as the depth is greater than 150 mm ([8], *eq. (3.1)*).

Geometric properties.

Structural dimensions of the panel are shown in the Fig. 2.

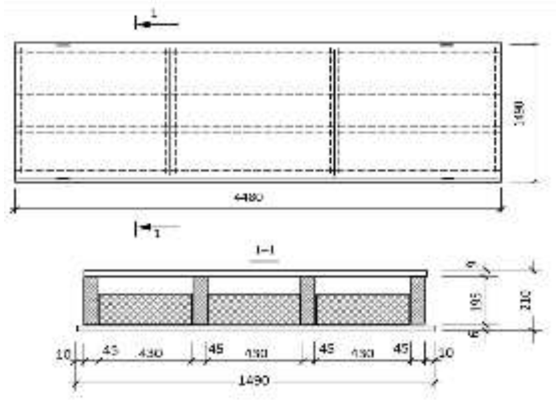

Figure 2 – Dimensions of panel

Clear distance between webs is b_f = 430 mm. Top flange thickness is δ_{tf} = 9 mm. Bottom flange thickness is δ_{bf} = 6 mm. Dimensions of web are $b_w \times h_w$ = 45×195 mm (50×200 mm dimensions of sawn timber before planning according).

Depth parameters of cross-section

$h = h_w + \delta_{bf} + \delta_{tf}$ = 19,5 + 0,6 + 0,9 = 21 cm.

The effective flange width concept applies to flanges in compression and in tension, and unless a more detailed calculation is carried out, in accordance with the requirements of [8], 9.1.2(3). The effective flange width, b_{ef}, as it is shown in Figure 2, for internal I-shaped sections, is as follows:

$b_{ef.c} = b_{c.ef} + b_w$ and $b_{ef.t} = b_{t.ef} + b_w$ ([8], *eq (9.12)*).

Effective flange width of an I-beam section of the panel ([8], *9.1.2*):

– in compression ([8], *Table 9.1*),

$b1_{c.ef} \le min\{ 0{,}1 \cdot L\ ;\ 20 \cdot \delta_{tf} \} \le 180$ mm=18 cm;

– in tension ([8], *Table 9.1*),

$b1_{t.ef} \le 0{,}1 \cdot L \le 450$ mm=45 cm (only shear lag).

The values $b1_{c.ef}$ and $b1_{t.ef}$ should not be greater than value b_f = 430 mm.

So, design sizes of the flanges are:

– in compression, $b_{c.ef} = b1_{c.ef}$ = 18 cm;

– in tension, $b_{t.ef} = b_f$ = 43 cm.

Effective flange width:

– in compression, $b_{ef.c} = b_{c.ef} + b_w$= 18+4,5 = 22,5 cm;

– in tension, $b_{ef.t} = b_{t.ef} + b_w$ = 43+4,5 = 47,5 cm.

Instantaneous – transformed section properties. As the panel has a cross-section of two different materials, it is convenient to go transformed section or fictitious section.

Coefficient for transformed section, as ratio of modulus according (1):

$n_E = E_{0.mean} / E_{plw.0.mean}$ = 10 / 4=2,5 ,

where $E_{0.mean}$ = 10 kN/mm² – mean modulus of elasticity parallel for solid timber, strength classes *C22* ([9], *Table Б.1*);

$E_{plw.0.mean}$ = 4 kN/mm² – modulus of elasticity parallel for plywood, strength classes *F20/10 A40/20* (([9], *Table Б.5*).

Transformed web thickness (into plywood):

$b_{w.tfd} = b_w \cdot n_E$ = 4,5·2,5 = 11,25 cm.

Areas of cross-sections flanges (2,3)

$A_{ef.f.c}$ = 20,25 cm², $A_{ef.f.t}$ = 28,5 cm².

Area of the webs (4)

$A_{ef.w}$= 219,38 cm².

Transformed area (5)

A_{ef} = 268,13 cm².

First moment of section area about the top face (6)

A_{1st} = 2935,41 cm³.

Neutral axis (*NA*) depth from the top face (7)

y_t = 10,9 cm.

Second moment of web area about the *NA* (8)

$I_{ef.w}$= 6971 cm⁴.

Second moment of the top flange area about the *NA* (9)

$I_{ef.tf}$= 2333cm⁴.

Second moment of the bottom flange area about the *NA* (10)

$I_{ef.bf}$= 2758 cm⁴.

Instantaneous second moment of the transformed section area (11)

I_{ef} = 11962 cm⁴.

Bending stress check in the flanges and web (ULS).

Stress in the flanges due to bending:

– bending stress (compression) in the top flange (12):

$$\sigma_{f.c.max.d} = 0{,}637\ \text{kN/cm}^2 = 6{,}37\ \text{N/mm}^2.$$

– bending stress (tension) in the bottom flange (13):

$$\sigma_{f.t.max.d} = 0{,}592\ \text{kN/cm}^2 = 5{,}92\ \text{N/mm}^2$$

Strength of the top flange for plywood *F20/10 E40/20* and $f_{plw.c.k}$ = 15 N/mm^2 ([9], *Table Б.5*):

$$f_{plw.c.d} = f_{plw.c.k} \cdot k_{\text{mod.medium}} \cdot k_{sys} =$$
$$= 15 \cdot 0{,}8 \cdot 1 = 12\ \text{N/mm}^2.$$

Strength of the bottom flange for plywood *F20/10 E40/20* and $f_{plw.t.k}$ =9 N/mm^2 (([9], *Table Б.5*):

$$f_{plw.t.d} = f_{plw.t.k} \cdot k_{\text{mod.medium}} \cdot k_{sys} =$$
$$= 9 \cdot 0{,}8 \cdot 1 = 7{,}2\ \text{N/mm}^2.$$

Strength is satisfactory:

$$\sigma_{f.c.max.d} < f_{plw.c.d}\ ,\ \sigma_{f.t.max.d} < f_{plw.t.d}.$$

Bending stress check in the web:

– maximum distance from the *NA* to the extreme fibre,

$$y_1 = max\{(y_t - \delta_{tf});\ (h - \delta_{bf} - y_t)\};$$

$$y_1 = max\{(10{,}9 - 0{,}9);(21 - 0{,}6 - 10{,}9\} = 10\ \text{cm};$$

– bending stress in the web (14),

$$\sigma_{w.c.d} = 1{,}525\ \text{kN/cm}^2 = 15{,}25\ \text{N/mm}^2.$$

Bending strength of the web for solid timber *C22* and $f_{m.0.k} = 22\ N/mm^2$ (([9], *Table Б.1*):

$$f_{m.0.d} = f_{c.0.} \cdot k_{\text{mod.medium}} \cdot k_{sys} \cdot k_h =$$
$$= 22 \cdot 0{,}8 \cdot 1 \cdot 1 = 17{,}6\ \text{N/mm}^2.$$

Strength is satisfactory:

$$\sigma_{w.c.d} < f_{m.0.d}\ .$$

Shear stress of the web (ULS). First moment of the section area above the NA (16), $S_{tf.NA}$=780,48 cm^4.

Shear stress at the NA position (15):

$$\tau_{v.d} = 0{,}095\ \text{kN/cm}^2 = 0{,}95\ \text{N/mm}^2$$

Shear strength of the web material (solid timber *C22)* with $f_{V.k}$ = 2 N/mm^2 ([9], *Table Б.1*):

$$f_{V.d} = f_{V.k} \cdot k_{\text{mod.medium}} \cdot k_{sys} = 2 \cdot 0{,}8 \cdot 1 = 1{,}6\ \text{N/mm}^2.$$

Design shear strength is greater than the shear stress:

$$\tau_{v.d} < f_{V.d}.$$

Shear stress of the glued joint between the web and the flanges (ULS).

First moment of top flange area above the NA (17):

$$S_{tf} = 212{,}58\ \text{cm}^4.$$

First moment of the bottom flange area about NA (18):

$$S_{bf} = 277{,}94\ \text{cm}^4.$$

Maximum value of first moment of area about NA:

$$S_f = max\ \{S_{tf};\ S_{bf}\} = max\{212{,}58;\ 277{,}94\} =$$
$$= 277{,}94\ \text{cm}.$$

Mean shear stress in the flange across the glue line (19):

$$\tau_{v.d} = 0{,}034\ \text{kN/cm}^2 = 0{,}34\ \text{N/mm}^2$$

Rolling shear strength of the flange material (plywood *F20/10 E40/20*) with $f_{plw.V.k}$ = 3,5 N/mm^2 (([9], *Table Б.5*):

$$f_{plw.V.d} = f_{plw.V.k} \cdot k_{\text{mod.medium}} \cdot k_{sys} =$$
$$= 3{,}5 \cdot 0{,}8 \cdot 1 = 2{,}8\ \text{N/mm}^2.$$

Rolling shear strength is determined according to ([8], *9.1.2(6)*:

– if $b_w \le 8 \cdot \delta_{bf}$, then $f_{plw.V.d}$;

– if $b_w > 8 \cdot \delta_{bf}$, then $f_{plw.V.d} \cdot (8 \cdot \delta_{bf} / b_w)$.

Design shear strength is greater than the shear stress:

$$\tau_{v.d} < f_{plw.V.d}$$

Conclusion

For design cross-section of composite panel, it can be applies a simple method of fictitious cross-section. The presented design methodology can be used to design composite (timber and plywood) skin panel in accordance with the requirements [8] and [9].

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STRENGTH ANALYSIS OF GLUED-IN STEEL RODS WITH DIFFERENT LOCATIONS IN CLT PANELS CROSS SECTION

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New types of wood based building materials to which the CLT refers require an accurate evaluation of the strength of various types of connections. CLT panels connections with glued-in steel rods are of interest due to the possibility of creating quick-mounted and rigid joints in the factory. Since the CLT have the structure of the perpendicular orientated boards in adjacent layers, the strength and behaviour of the pasted rods is difficult to predict. The purpose of this study was to establish the strength of the glued-in rods by pull-pull tests with different locations relative to the boards layers in the cross-section of the CLT panel. Diameter of all considered steel rods was smaller than thick of timber planks 30 mm in 5-lyers CLT specimens without gaps and stress relieves. Anchored length of rods in all specimens was 100 mm by using two component epoxy adhesive system.

Keywords: cross laminated timber, CLT, connection, glued-in steel rods, joint, prefabricated panels, strength by pull-pull tests

АНАЛІЗ МІЦНОСТІ ВКЛЕЄНИХ СТЕРЖНІВ З РІЗНИМИ ТОЧКАМИ РОЗТАШУВАННЯ У ПОПЕРЕЧНОМУ ПЕРЕРІЗІ ПКД ПАНЕЛЕЙ

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Нові типи дерев’яних будівельних матеріалів, до яких відносяться CLT або ПКД панелі, вимагають точної оцінки міцності різних типів з'єднань. З'єднання ПКД панелей на вклеєних стержнях представляють інтерес через можливість створення на виробництві швидко монтуючихся і жорстких з'єднань. Оскільки ПКД має структуру перпендикулярно орієнтованих дощок у сусідніх шарах, міцність та поведінку наклеєних стрижнів важко передбачити. Метою цього дослідження було встановити міцність вклеєних стрижнів за допомогою випробувань на висмикування стрижнів розташованих у різних точках відносно шарів дощок у поперечному перерізі ПКД панелі. Діаметр усіх розглянутих сталевих стержнів був меншим за товщину дощок з деревини яка складала 30 мм у 5-шарових зразках CLT без щілин між дошками і компенсаційних пропилів у дошках для зняття напруги. Глибина вклеювання стрижнів у всіх зразках становила 100 мм та використовувалась двокомпонентна епоксидна система. Для стрижнів використовували сталеві шпильки з метричною різьбою М10, М12 та М14 класу міцності 5.8. Метод випробування проводився відповідно до вимог стандарту EN 1382:1999. Навантаження прикладалось з постійною швидкістю від 0,5 до 1,5 мм/хв до відмови або руйнування з’єднання. При випробуваннях вклеєних стержнів велику увагу слід приділяти не тільки несучій здатності, але й поведінці при руйнуванні. Для визначення міцності на висмикування вклеєних стержнів, встановлених у боковій поверхні плит CLT, було випробувано 75 зразків. Всього було розглянуто 5 можливих точок встановлення вклеєних стержнів у поперечному перерізі ПКД або CLT панелі, які можуть мати різну міцність. Це перш за все стрижні встановлені у повздовжні і поперечні дошки поперечного перерізу ПКД панелі, стрижні встановлені на межі повздовжньої та поперечної дошки і на межі двох повздовжніх дошок, а також розглядався варіант розташування стержня на межі двох повздовжніх і однієї поперечної дошки.

Ключові слова: вклеєні стержні, збірні панелі, з’єднання, поперечна клеєна деревина, ПКД, міцність при висмикуванні, стик

Introduction

Glued-in steel rods are very effective type of rigid and semi-rigid joints in GLT elements of different constructions and allow produce structural elements with high level of prefabrication for fast and reliable installation of timber buildings. Positive experience of using glued-in steel rods during large period in timber constructions in Eastern Europe praxis seems effective in joints of cross laminated timber (CLT) panels. In this paper showed the results of pull-pull tests of glued-in steel rods installed in edge face of CLT panels in different possible variations. Also was proposed calculation model and type of joint were it can be used in CLT buildings (Fig.1) for connecting of wall and floor panels or two wall panels.

Review of the research sources and publications

Analysis of the glued-in rods strength by pulling, depending position in the cross-section, is the same as described in paper [1] of prof. H. J. Blass (2007) where the positions of axially loaded self-tapping screws were varied.

Definition of unsolved aspects of the problem

The strength of glued-in rods as a rule depends not only on the length of the gluing of the rod and its diameter, but also on the orientation to the grain in the specimens. In CLT panels an interesting case of the arrangement of the rods is the variant on the boundary of two adjacent parallel and perpendicular boards because of the different shear strength of timber along and across the grain. For example, in the tests of glued-in rods in the LVL, the destruction of the cores was observed with the ovalization of the wood near the rod. This is logical and can be explained by the different values of the shear strength by acting force parallel to the layers of veneer and perpendicular to the layers of veneer.

Proposed joint of CLT panels with GiR as well as «X-RAD system» produced by Rothoblaas allow increase the level of prefabrication, greater efficiency and reduced times by mounting. In the installation process of this joint can be used for lifting CLT panels and ensure comfortable connecting of panels from the inside of the building. For connecting two wall panels and one floor panel envisaged gluing-in of rods in floor panel. One-sided semicircular apertures in the places of joining the panels allow to slightly reduce the thermal conductivity of the panel. Optimization of considered joint is obviously necessary, especially taking into account the group effect of glued-in rods.

Problem statement

The aim of the research was to develop proposals for calculating of load carrying capacity of joints with glued-in steel rods in CLT panels by taking into account the position or rod in cross section of CLT panel.. Only axial loaded glued-in steel rods installed in different parts of panels cross section was tested by pull-pull tests and described in this paper. Also withdrawal capacity of glued-in steel single rods in CLT are need for calculation for reinforcement different parts of panels and connections.

Interest to this type of connection in CLT panels occurred in a few papers. Some test results of withdrawal capacity of GIR in CLT elements are published in the STSM Report of B. Azinovic [2], G. Traetta (2007) [4], and in Master thesis of M. Andersen and M. Høier [3].

Figure 1 – Possible case prefabricated joint of CLT panels with glued-in steel rods

Basic material and results

Materials and methods

The present research work is considered the influence of glued-in rod (GiR) localization in edge of CLT panels on the value of maximal strength of withdrawal capacity. Diameter of all considered steel rods was smaller than thick of timber planks 30 mm in 5-lyers CLT specimens without gaps and stress relieves. Anchored length of rods in all specimens was 100 mm by using two component epoxy adhesive system. The moisture content of the timber was about 12%. The test set-up and the location of GiR are shown on figure 2, where (1) – parallel to the grain in one board; (2) – on the boundary of two parallel and one perpendicular boards; (3) – on the boundary of two parallel boards; (4) – on the boundary of one parallel and one perpendicular boards; (5) – perpendicular to the grain in one board. Differences between specimens were in positions of glued-in rods and diameters of rods, see table 1.

a)

b)

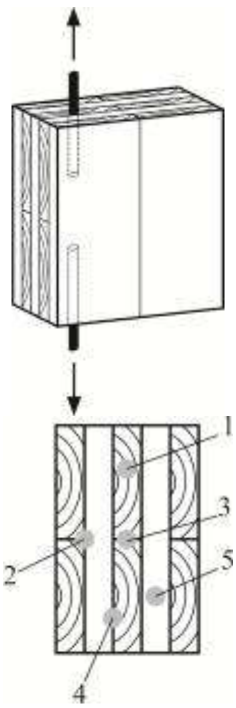


Figure 2 – Schemas:
a – test set-up; b – locations of glued-in steel rods in the tested specimens.

Table 1 – Test program

Test series	Glued-in length lad [mm]	Rods diameter d [mm]	Glue line thickness [ ]	Number of specimens	Number of locations
GiR-10	100	10	2	25	1-5
GiR-12	100	12	2	25	1-5
GiR-14	100	14	2	25	1-5

The purpose of the tests was to estimate the load carrying capacity of single GiR. The rods use were threaded steel bars with metric threads M10, M12 and M14 in strength grade 5.8. Test method was according to standard EN 1382:1999 [5]. The load was applied at a constant rate between 0.5 and 1.5 mm/min until failure. By tests great attention should be given not only for load capacity, but also to the behaviour of joints.

To determine the withdrawal strength of GiR installed in edge face of CLT plates 75 specimens were tested. In Table 1 show testing program and details of test series. In each location was tested 5 specimens. As shown on Fig.2-b) was considered all possible cases of rods installing which could influence on obtained results. Obviously very interesting seems results of rods located on the bonding lines of two adjacent cross orientated boards.

During the tests was controlled displacement by measuring devices and symmetry of glued-in rods in two planes as shown on figure 3.

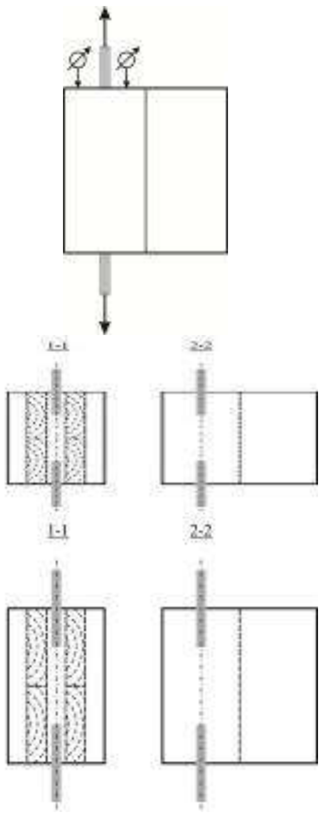


Figure 3 – Location of measuring devices and symmetry control of glued-in rods

Test results and discussion. The experimental data obtained for CLT have been compared with predictive equations from three references. First is simplified calculation model [6-9] for axial loading which could be summarized as:

$$R_{ax,k} = \pi \cdot d \cdot l \cdot f_{v,k} \ , \tag{1}$$

where $R_{ax,k}$ – characteristic pull-out capacity;
l – anchorage length;
d – diameter;
$f_{v,k}$ – shear strength parameter.

The correlation between test results and predicted values which based on models proposed in DIN 1052 and Russian standard (СП 64.13330.2017) [10] for GiR was analysed as second and third references according to equation (2) and (3)

$$R_{ax,d} = \pi \cdot d \cdot l_{ad} \cdot f_{k1,d}\,, \qquad (2)$$

where $R_{ax,d}$ – design axial resistance;
l_{ad} – effective anchorage length;
d – nominal diameter of rod;
$f_{k1,d}$ – characteristic value of bond line strength

$$T = R_{ск} \cdot \pi \cdot d_1 \cdot l_p \cdot k_c\,, \qquad (3)$$

where T – design axial pull-out or pierce capacity;
$R_{ск}$ – shear design resistance;
R – 4 MPa timber design pull-out resistance or pierce of rod;
l_p – design length, m;
k_c = 1,2-0,02·lp/d coefficient which takes into account uneven shear stress distribution in dependency of the anchorage length of rod.

Since in some cases of tested glued-in rods in the CLT panels, the rods were located in the boards perpendicular to the grain. In European practice designers are using the same equations for rods set perpendicular to the grain, or are referring to Widmann et al [12] where the pull-out strength is estimated as follows

$$F_{ax,mean} = 0.045 \cdot (\pi \cdot d_h \cdot l_{ad})^{0.8}, \qquad (4)$$

where d_h – diameter of drill hole in mm.

In accordance to Russian standard [10] the design equation for rods glued-in perpendicular to the grain considered as case of rods glued-in at an angle to the grain as follows

$$T = R \cdot \pi \cdot d_1 \cdot l_p \cdot k_c \cdot k_\sigma \cdot m_d \le F_a \cdot R_a\,, \qquad (5)$$

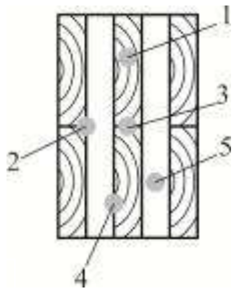
where k_σ = 1-0,001·σ coefficient which depends on the sign of normal stresses along the grain in the place of rods installation;
σ – maximal tension stresses, MPa;
m_d = 1,12-0,1d coefficient which takes into account dependency of design resistance from rod diameter;
F_a – cross section of rod, m^2;
R_a – design resistance of rod material, MPa.

Typical failure modes that occurred were shear failure along the rod in the adhesive layer, shear failure along the rod in the interface between the adhesive and the surrounding timber, shear failure along the rod in the surrounding timber. Figure 4 shows failure mode of tested specimens. Failure was reaching the shear strength.

Figure 5 show the test results for the GiR diameters 10, 12 and 14 mm depending from installation place of in CLT edge face. It is obvious that the load-carrying capacity increases with increasing of rods diameter, as mentioned above in equation (1).

The increase in deformations when pulling out rods installed across the grain are more intense than for rods installed along the grain, as was noted earlier and described in paper prof E. Serrano [11]. Load-deformation curves for rods glued on the border of the longitudinal and transverse boards had intermediate shapes of the curves. Results of the test to study the influence of the rod position in edge surface of CLT panel by pull-pull tests for three different rods diameters are shown in table 2.

Figure 4 – Failure mode of some tested specimens



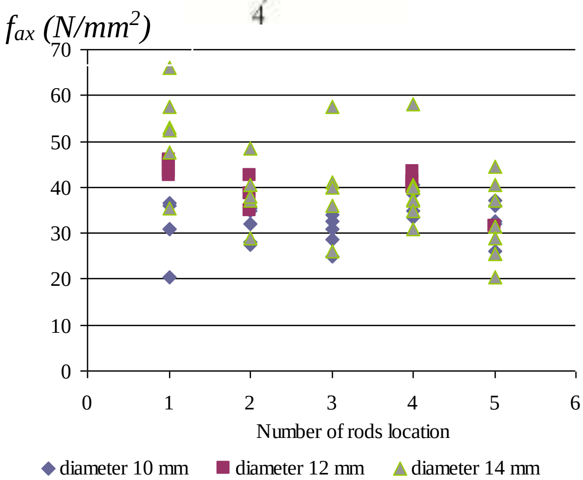


Figure 5 – Test results for different rods diameters by different places of installation

Table 2 – Test results

Test series	Mean shear strength τ, [kN/mm^2]	Coefficient of variation, [%]	Glued-in length lad [mm]	Rod diameter d, [mm]	Rods positions
GiR-10-1	8,62	13	100	10	1
GiR-12-1	7,17	11,5	100	12	
GiR-14-1	6,32	26	100	14	
GiR-10-2	8,38	11,9	100	10	2
GiR-12-2	8,8	9,7	100	12	
GiR-14-2	7,68	18,2	100	14	
GiR-10-3	7,85	11,9	100	10	3
GiR-12-3	7,51	13,7	100	12	
GiR-14-3	7,98	25,4	100	14	
GiR-10-4	9,69	8,7	100	10	4
GiR-12-4	9,42	3,1	100	12	
GiR-14-4	7,93	21,6	100	14	
GiR-10-5	8,23	20,8	100	10	5
GiR-12-5	10,07	3,5	100	12	
GiR-14-5	10,35	19,6	100	14	

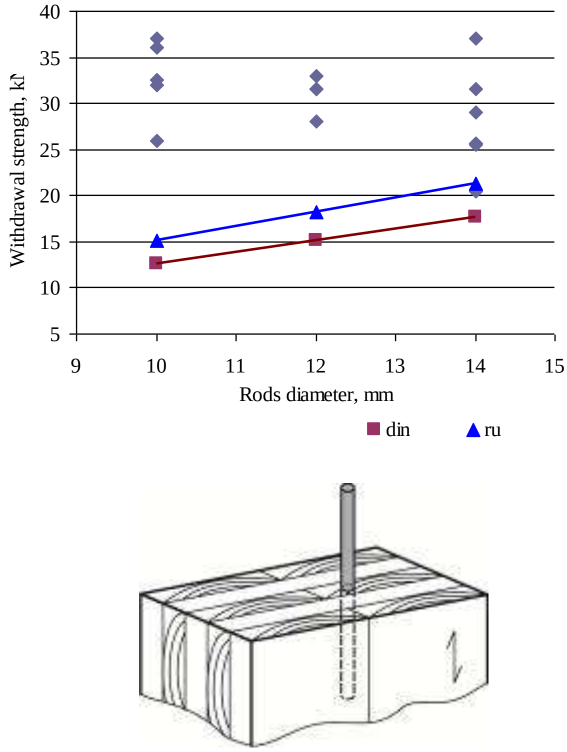


Figure 6 – Test results versus predicted strength for rods glued-in along the grain

Figure 6 shows the influence of rods diameter on withdrawal strength, where lines «din» and «ru» are predicted values according to German standard DIN 1052 and Russian standard СП 64.13330:2017 for roads glued-in along the grain. Test results for rods glued-in perpendicular to the grain versus predicted values obtained according Russian standard and equation (4) proposed by R. Widmann shown on figure 7. As a rule, the test results are slightly higher than the expected strength values of GiR, because in the calculation formulas (1) and (3) the characteristic value of the strength by shear is used.

Also by shear testing of specimens, the resulting strength value corresponds to a flat stress state. When pulling out the GiR installed along the grain is observed volume destruction and it is necessary to consider the volumetric stress state. For example, by tests of GiR in LVL was found ovalization of the fracture area, which is explained by different shear strength along the grain due to the layered structure of the material. This aspect is taken into account and used in the calculations according to our proposed method. For GiR located on the border of the longitudinal and transverse boards in CLT, a non-uniform stress state also arises depending on the part of the contact area of the GiR with one of the board surfaces.

For CLT panels with gaps and stress relieves, the installation of GIR is also possible. In the drilled hole, which coincides with the gap, in the lower part of the hole need to install a plug to avoid glue leakage. The designed connection type can be used when mounting the panels.

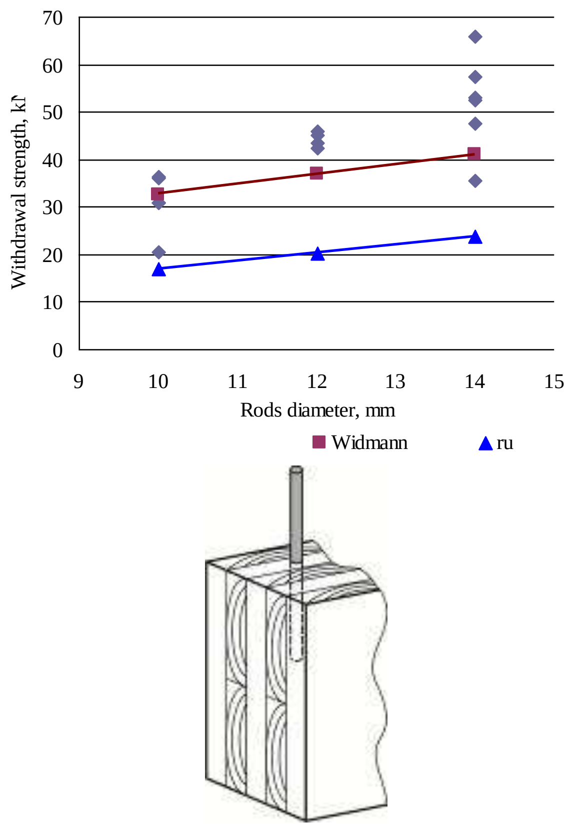


Figure 7 – Test results versus predicted strength for rods glued-in perpendicular to the grain

Conclusions

Results of pull-pull tests glued-in steel rods positioned in the edge of CLT panels shown new and interesting results, especially for rods installed on the boundary of parallel and cross bonded boards. In described above tests series diameters of rods was smaller than thickness of boards in CLT panel. Performance of analogical tests in CLT with thin boards seems reasonable and interesting.

For future research and implementation of glued-in steel rods in CLT are need investigation of multiple rods are required for joining of thick CLT plates or highly stressed connections. In this connection, it will be necessary to determine the distances between the rods and from the rods to the edge faces, taking into account the scheme of the boards in CLT panel. Future investigations of laterally loaded glued-in steel rods in CLT are required.

Developed model of GiR strength in CLT based on the linear regression analysis of obtained test results and now it is checked by additional tests that are necessary to confirm the formulated calculation method.

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STOCHASTIC CALCULATION OF A QUASI-HOMOGENEOUS BOLTED JOINTS OF THE BODY SHEETS OF THE STEEL SILOS

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This paper deals with the study of reliability of quasi-homogeneous bolted joints of the body of thin-walled constructions of the steel silos. Heterogeneity of this unit can be caused by number of reasons, in particular by presence of bolts of different strength in joint. In such case the reliability of the system is determined trough the probability of trouble-free work of the coefficient of the critical factor. The general conception of stochastic calculation consists in using the Monte-Karlo simulation procedure for the samples of random values of the large volume. It was formed the system of conditions, under which the reliability of joints is provided and analytical expressions for the value of coefficient of the critical factor is got, and also made practical calculation example with the following graphic presentation on a special coordinate plane – a critical stochastic scale. It was illustrated that random presence of bolts of less strength in field joint rapidly increases the risk of refusal.

Keywords: cylindrical silos, reliability, stochastic calculation, probability of the trouble-free work, quasi-homogeneous bolted joint, coefficient of the critical factor.

ІМОВІРНІСНИЙ РОЗРАХУНОК КВАЗІОДНОРІДНИХ БОЛТОВИХ З’ЄДНАНЬ ЛИСТІВ КОРПУСУ СТАЛЕВИХ СИЛОСІВ

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Стаття присвячена актуальній проблематиці дослідження надійності квазіоднорідних болтових з’єднань корпусу циліндричних сталевих силосів. У багатьох випадках проектна надійність таких стиків відрізняється від фактичних показників їх безвідмовної роботи. Джерелом цього є ряд об’єктивних причин, до числа яких можна віднести недоліки монтажних операцій, зокрема присутність болтів різного класу міцності, відсутність декількох болтів з’єднання, розкручування кріплення болта в процесі експлуатації споруди чи недокрут на монтажі й інші чинники такого типу. У цьому випадку надійність системи визначалася через імовірність безвідмовної роботи з використанням коефіцієнта критичного фактора. Відмова болтового з’єднання однозначно визначалася як перевищення випадковим напруженням випадкового рівня границі текучості сталі. Загальна концепція ймовірнісного розрахунку полягала у використанні імітаційної процедури Монте-Карло для вибірок випадкових величин великого об’єму. Увага концентрувалася на розрахунковій ситуації, коли болти з’єднання складаються з трьох незалежних вибірок. Була сформульована система умов, при якій забезпечується надійність з’єднання й отримані аналітичні вирази для величини коефіцієнта критичного фактора. Наведено практичний приклад розрахунку з наступним графічним представленням на спеціальній координатній площині – критичній імовірнісній шкалі. Проілюстровано, що випадкова наявність болтів менших класів міцності в монтажному стику стрімко збільшує ризик відмови. Установлено, що показники ймовірності безвідмовної роботи однорідних та квазіоднорідних болтових з’єднань значно різняться лише при великих значеннях коефіцієнтів варіації значень зусилля. Показано, що навіть за відсутності мінливості значень міцності болтів випадкова величина критичного фактора буде зростати при постійному значенні показника надійності.

Ключові слова: циліндричний силос, надійність, імовірнісний розрахунок, імовірність безвідмовної роботи, квазіоднорідне болтове з’єднання, коефіцієнт критичного фактора.

Introduction

The problem of reliability assessment is one f the most important scientific problems that arise in the area of the building construction's calculation. It is connected not only with obtaining on its results more justified results from the economic point of view, but also with general entity of the term reliability. Depending from the chosen index, the reliability serves as a qualitative and quantitative indicator of the whole complex of constuction's properties. Considering the problem of stochastic calculation of bolted joint of the elements of steel silos, we need to pay attention to the problem connected first of all with features of this class of construction – usage of thin-walled core and sheet corrugated elements, high level of load, etc.

Also, due to the large number of accidents caused by numerous mounting errors, the second most important consideration when determining the level of reliability connection is taking into account the most common construction defaults that arise in the construction process.

Review of the research sources and publications

The assessment of the stress-strain state and the problem of determining the strength of bolted joints of metal structures, including thin-walled elements, remain urgent, which often can be found in the scientific publications [1 – 4]. We will note that among them there are important applied problems of analytical and numerical calculation of the bolted joints of the composite thin-walled constructions with considering features of friction, corrugation, presence of sealing gaskets, etc. [5, 6], and also numerical and experimental studies [7, 8]. Traditionally, the area of stochastic calculation remains less studied, which is certainly caused by the complexity of mathematical transformations and the large amount of theoretical knowledge in the field of probability theory and mathematical statistics [9 – 10]. The modern studies of the outlined issues have repeatedly been raised in the works of Pichugin S.F. and students of his Reliability Science School [11 – 14].

Definition of unsolved aspects of the problem

In the practice of designing units of thin-walled constructions of steel silos, the bolted joints of elements became widespread. In many cases, the design reliability of such joints differs from the actual indicators of their trouble-free work. The source of this is a number of objective reasons, which include the disadvantages of installation operations – wrong tightening of bolts or the use of fastening elements of the non-design class of strength. In this case, the classic strength analysis of the bolted joint, as in the analytical form, so in the application of finite element apparatus, requires not only the change of the initial data of the solved problem, but also the correction of a number of influential factors. These features are beyond the guidance of regulatory documents, and, accordingly, are overlooked in the process of engineering calculations. As well as the stock values, the reliability levels of bolted connections made with mounting defects remain unknown. Their assessment is an important task that requires thorough consideration and detailed scientific analysis.

Problem statement

The main purpose of this study is the stochastic analysis of the heterogeneous bolted joint of thin-walled elements of the steel silo body. The heterogeneity of such a unit can be caused by a number of reasons, including the presence of bolts of different strength class, the absence of several connection bolts, untwisting of the bolt attachment during the exploitation period of the construction or short of rotation during mounting and other factors of a similar nature.

Basic material and results

The main structural elements of the body of the steel silos are the thin-walled elements of the corrugated sheets and the vertical stiffeners, which are interconnected by bolts made with gap and pre-tightening. The fasteners, which are used in the connection (fig. 1), consist of bolts and nuts, which are zinc-coated, and of sealing washers. Mostly, the structural units are designed for bolts of strength of classes 5.8 for the roof and 8.8 and 10.9 for the body of the capacity.

a)

b)

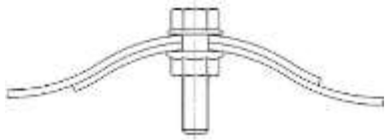

Figure 1 − Fasteners (a) and arrangement of bolted joint of the corrugated sheets of the silo body (b)

For the basic indicator of the quantitative measure of reliability when calculating the bolted connections, we take the coefficient of the critical factor, which is equal to the ratio of the random effort $\tilde{S}$ and the generalized strength $\tilde{R}$

$$\tilde{K}_R = \frac{\tilde{S}}{\tilde{R}} \le 1{,}0\ . \qquad (1)$$

The main models of the failure are the cut of the bolt and the bearing of the metal of the connected elements in the hole. In general, the bolted connection of the body sheet can be represented as a multi-element system, the failure of each element of which leads to the formation of a new design scheme of the joint and thus provokes a change in the reliability of the whole connection.

The general concept of a stochastic calculation will be based on the use of the Monte-Carlo simulation procedure for samples of the random variables $\gamma_{R,i}$ and $\gamma_{S,i}$ of the large volume [15]. For the critical factor of the basic metal of the sheets of the silo body, the following ratio will be valid

$$K_{R,i} = \frac{m_S}{m_R} \cdot \frac{1+\gamma_{S,i} V_S}{1+\gamma_{R,i} V_R} = m_K \cdot \gamma_{K,i} \,, \qquad (2)$$

where m_S and m_R – mathematical expectations of random variables $\tilde{S}$ and $\tilde{R}$;
$\tilde{\gamma}_R$ and $\tilde{\gamma}_S$ – the normalized random variables with a given distribution law;
V_R and V_S – the coefficients of variation of strength and load;
m_K – the expected value of a critical factor.

In the design case of a homogeneous bolted connection, when all bolts of the system are elements of one sample, the critical factor of the metal of bolts $\tilde{K}_{b,i}$ is expressed by the following expression [14]:

$$\tilde{K}_{b,i} = \frac{\tilde{S}_i}{\tilde{R}_{bs,i} \cdot A_b \cdot n} = m_b \cdot \frac{1+V_S \tilde{\gamma}_{S,i}}{1+V_{bs} \tilde{\gamma}_{bs,i}} \,; \qquad (3)$$

$$m_b = \frac{1,273}{d_b^2 \cdot n} \cdot \frac{m_S}{m_{bs}} \,, \qquad (4)$$

where $\tilde{R}_{bs,i}$ – the random value of the strength of one bolt per cut;
A_b – the value of the area of the bolts;
d_b – the diameter of the bolt;
n – the number of elements (bolts) of the sample;
m_S , m_{bs} and V_S , V_{bs} – respectively the values of the expected value and coefficients of variation of the random variables $\tilde{S}_i$ and $\tilde{R}_{bs,i}$;
$\tilde{\gamma}_{bs,i}$ – the normalized random value of the strength of the bolt;
$\tilde{\gamma}_{S,i}$ – the value of the normalized random value of maximums of the external force, which influence the connection.

The problem of stochastic calculation of a quasi-homogeneous bolted joint, when the bolts of connection consist of two independent samples $n = n_1 + n_2$, has been considered in studies of the group of authors [14]. We also should consider the design situation where the bolts of connection consist of three independent samples $n = n_1 + n_2 + n_3$.

This case will illustrate the change in system reliability when there are coupled three types of fasteners in a unit connection, such as bolts of different strength classes or different manufacturing plants.

The use of expression (3), taking into account the corresponding values of the expected value m_b and the coefficient of variation V_{bs} , will be valid only in cases $\tilde{K}_{b,i} < 1{,}0$, ie., before the failure

$$\tilde{K}_{b,i} = \frac{\tilde{S}_i}{A_b \cdot (\tilde{R}_{bs1,i} \cdot n_1 + \tilde{R}_{bs2,i} \cdot n_2 + \tilde{R}_{bs3,i} \cdot n_3)} . \qquad (5)$$

If there is a connection failure, a critical factor exceeded the single-level. This process can be implemented according to different schemes, each of them should be considered when calculating a random value of the critical factor. For the simplest connection of the three bolts, the random strength of which in the space of effort is corresponding to $\tilde{S}_{01}$, $\tilde{S}_{02}$ and $\tilde{S}_{03}$ the reliability of the connection will be ensured if the following demands are made for the *i*-th sample element:

1) the value of the random strength of each of the three bolts $\tilde{S}_{01,i} \vee \tilde{S}_{02,i} \vee \tilde{S}_{03,i}$ will be bigger than the force $\tilde{S}_i / 3$;

2) the strength of one of the bolts is less than $\tilde{S}_i / 3$ and it is ruined, but the strength of the remaining two ones is bigger than $\tilde{S}_i / 2$;

3) the strength of two bolts is less than $\tilde{S}_i / 3$, and of the third one is bigger than $\tilde{S}_i$;

4) the strength of one bolt is bigger than $\tilde{S}_i / 3$ but less $\tilde{S}_i / 2$; the strength of the second bolt is bigger than $\tilde{S}_i / 2$ and of the third one is bigger than $\tilde{S}_i$.

In this case, we will get a system of conditions for a critical factor:

I − the strength of all bolts is less than $\tilde{S}_i / 3$;

II… IV − strength of one of the bolts is less than $\tilde{S}_i / 3$;

V… VII − the effort $\tilde{S}_i / 3$ in both bolts simultaneously exaggerates their bearing capacity;

VIII − the bearing capacity of all bolts is less than $\tilde{S}_i / 3$.

This system can generally be schematized by the expression (6), where for values $\tilde{K}_{b12,i}$, $\tilde{K}_{b13,i}$ and $\tilde{K}_{b23,i}$ we will apply the term «conditional» critical factors. These random variables will be calculated just as for the case of connecting elements from two independent samples, which were described in the paper

$$\tilde{K}_{b,i}=\begin{vmatrix}\tilde{S}_i/\left(\tilde{S}_{01,i}+\tilde{S}_{02,i}+\tilde{S}_{03,i}\right), & if & \tilde{S}_i/\tilde{S}_{01,i}<3\wedge\tilde{S}_i/\tilde{S}_{02,i}<3\wedge\tilde{S}_i/\tilde{S}_{03,i}<3;\\ \tilde{K}_{b12,i}, & if & \tilde{S}_i/\tilde{S}_{01,i}<3\wedge\tilde{S}_i/\tilde{S}_{02,i}<3\wedge\tilde{S}_i/\tilde{S}_{03,i}\geq 3;\\ \tilde{K}_{b13,i}, & if & \tilde{S}_i/\tilde{S}_{01,i}<3\wedge\tilde{S}_i/\tilde{S}_{02,i}\geq 3\wedge\tilde{S}_i/\tilde{S}_{03,i}<3;\\ \tilde{K}_{b23,i}, & if & \tilde{S}_i/\tilde{S}_{01,i}\geq 3\wedge\tilde{S}_i/\tilde{S}_{02,i}<3\wedge\tilde{S}_i/\tilde{S}_{03,i}<3;\\ \tilde{S}_i/\tilde{S}_{01,i}, & if & \tilde{S}_i/\tilde{S}_{01,i}<3\wedge\tilde{S}_i/\tilde{S}_{02,i}\geq 3\wedge\tilde{S}_i/\tilde{S}_{03,i}\geq 3;\\ \tilde{S}_i/\tilde{S}_{02,i}, & if & \tilde{S}_i/\tilde{S}_{01,i}\geq 3\wedge\tilde{S}_i/\tilde{S}_{02,i}<3\wedge\tilde{S}_i/\tilde{S}_{03,i}\geq 3;\\ \tilde{S}_i/\tilde{S}_{03,i}, & if & \tilde{S}_i/\tilde{S}_{01,i}\geq 3\wedge\tilde{S}_i/\tilde{S}_{02,i}\geq 3\wedge\tilde{S}_i/\tilde{S}_{03,i}<3;\\ \tilde{S}_i/\left(\tilde{S}_{01,i}+\tilde{S}_{02,i}+\tilde{S}_{03,i}\right), & if & \tilde{S}_i/\tilde{S}_{01,i}\geq 3\wedge\tilde{S}_i/\tilde{S}_{02,i}\geq 3\wedge\tilde{S}_i/\tilde{S}_{03,i}\geq 3,\end{vmatrix} \quad (6)$$

Also, in order to find the values of conditional critical factors, based on the general expression (3), we can obtain the following formulas:

$$\tilde{K}_{b1,i}=\mathrm{M}_{b,1}\cdot\frac{1+V_S\tilde{\gamma}_{S,i}}{1+V_{bs1}\tilde{\gamma}_{bs1,i}}; \quad (7)$$

$$\tilde{K}_{b2,i}=\mathrm{M}_{b,2}\cdot\frac{1+V_S\tilde{\gamma}_{S,i}}{1+V_{bs2}\tilde{\gamma}_{bs2,i}}. \quad (8)$$

The magnitudes of the expected value and the coefficient of variation of the critical factor, when calculated by formulas (7) and (8), will acquire a new presentation:

$$m_{b,i}=\mathrm{M}_b\cdot\frac{1}{\alpha_{b,i}+\beta_{b,i}\eta_{bs}}; \quad (9)$$

$$V_{bs,i}=\frac{\sqrt{\alpha_{b,i}^2V_{bs1}^2+\beta_{b,i}^2V_{bs2}^2\eta_{bs}^2}}{\alpha_{b,i}+\beta_{b,i}\eta_{bs}}, \quad (10)$$

where $\eta_{bs}=\dfrac{m_{bs2}n_2}{m_{bs1}n_1}$ – the ratio of the characteristics and the number of bolts.

The values of the coefficients $\alpha_{b,i}$ and $\beta_{b,i}$,which appear in (9) and (10) will be determined from the following conditions

$$\alpha_{b,i}=\begin{vmatrix}1, & if & \tilde{K}_{b1,i}<2\wedge\tilde{K}_{b2,i}<2;\\ 1, & if & \tilde{K}_{b1,i}<2\wedge\tilde{K}_{b2,i}\geq 2;\\ 0, & if & \tilde{K}_{b1,i}\geq 2\wedge\tilde{K}_{b2,i}<2;\\ 1, & if & \tilde{K}_{b1,i}\geq 2\wedge\tilde{K}_{b2,i}\geq 2.\end{vmatrix} \quad (11)$$

$$\beta_{b,i}=\begin{vmatrix}1, & if & \tilde{K}_{b1,i}<2\wedge\tilde{K}_{b2,i}<2;\\ 0, & if & \tilde{K}_{b1,i}<2\wedge\tilde{K}_{b2,i}\geq 2;\\ 1, & if & \tilde{K}_{b1,i}\geq 2\wedge\tilde{K}_{b2,i}<2;\\ 1, & if & \tilde{K}_{b1,i}\geq 2\wedge\tilde{K}_{b2,i}\geq 2.\end{vmatrix} \quad (12)$$

To illustrate the calculation procedure for the search of the coefficient of the critical factor of a three-element bolted joint, we will do a practical example of calculation with the following graphical presentation. The calculation will be performed accordingly to the following initial characteristics of the system − the bolted joint receives the action of a random force, with the expected value $m_S=200$ kN and with one of the three values of the coefficient of variation $V_S=0{,}2\vee 0{,}4\vee 0{,}6$. The distribution density of the probabilities is assumed as a double exponential law. The bearing capacity of all three bolts is determined by a normal law, but with different statistical characteristics: for the first bolt − $m_{S,01}=250$ kN, for the second one − $m_{S,02}=200$ kN, and for the third one − $m_{S,03}=150$ kN. The range of the coefficients' changing of the bearing capacity of all bolts is accepted as $V_S=0{,}05\vee 0{,}1\vee 0{,}2$.

The reliability assessment will be performed on a special coordinate plane – the critical probability scale, on the y-axis of which we put the values of a random variable of a critical factor, and on the x-axis – the double natural logarithm of the probability of the failure-free work

$$y=-\ln[-\ln(F_\gamma)], \quad (13)$$

where F_γ – the probability of a failure-free work, which is calculated using the formula [14, 15]

$$F_\gamma=\exp\left[-\exp\left(\frac{\alpha_B-\sqrt{\alpha_B^2-4\alpha_A^2(1-\alpha_CV_S-1/m_K)}}{2\alpha_A}\right)\right], \quad (14)$$

where α_A, α_B and α_C – dimensionless coefficients, which consider the influence of the coefficient of variation of the bearing capacity and the law of maximums' distribution of a random load. For the load distribution, accordingly to the normal law of distribution $\alpha_A\approx -0{,}02$, $\alpha_B\approx 0{,}65$, $\alpha_C\approx 0{,}18$, and for the double exponential Humbel law $\alpha_A\approx 0$, $\alpha_B\approx 0{,}84$, $\alpha_C\approx 0{,}57$.

This allows comparing data across a wide range of a failure-free work.

The resulting curves of change of the critical factor are shown on the fig. 2, which have the graphs of three kinds on the coordinate plane.

The curves plotted with a solid thick line correspond to the reference functions of the critical factor.

The graphs, which are illustrated by dots, are the functions that are constructed by formula (6) without considering the calculations of conditional critical factors, i.e., it is considered the process of failure of one bolt in joint with the two others, but the mechanism of failure of one bolt of the joint is not taking into account.

Dotted curves are the values of the critical factor calculated in case of absence of any sequence of failure of the bolts in the joint.

Observing the nature of the change in the curves (fig. 2), we can see clearly an error, which arises in case of fully neglecting the sequence of the bolts' failure in the joint.

It is logical that the greatest differences occur between the solid and the dotted curves.

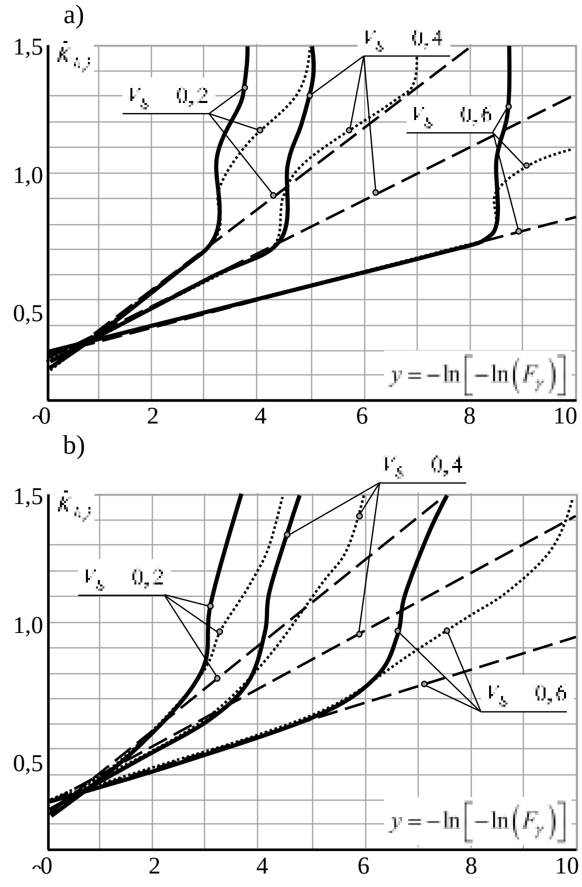

Figure 2 − Probability of a failure-free work of the joint of three bolts on a critical probability scale:

a – $V_{R,01} = V_{R,02} = V_{R,03} = 0,05$; $V_S = 0,2 \vee 0,4 \vee 0,6$;

b – $V_{R,01} = V_{R,02} = V_{R,03} = 0,2$; $V_S = 0,2 \vee 0,4 \vee 0,6$

The difference between solid and pointed curves is not so significant and does not increase in case of insignificant variability of the bolted strength, even if there are large values of the coefficient of variation of the external force. This is especially noticeable when comparing the behavior of the curves not in the entire coordinate plane, but only in the vicinity of the ordinate K_b = 1, for value of which the probability of a failure-free work is calculated.

Figure 2 also shows that the curves $\tilde{K}_{b,i}$ have three inflection points. However, the smaller the variability of the strength of the bolt is, the clearer the given point could be tracked and the greater is the angle of the curves to the horizontal axis. At large values of the coefficient of variation of the strength the curves of the critical factor are smoother in nature, and the points of inflection of the curves are almost imperceptible. Considering this, it can be assumed that the dependence of the critical factor from the logarithmic reliability index will always have as many inflection points as the bolts of different statistical characteristics, which are included in the joint. It should only be noted that the random values of the strength of the bolts must be independent.

Also, we consider that with small coefficients of variation of the force values and the different coefficients of variation of the strength of the bolts, the index of the probability of failure-free work of the joint is very close and differs only when V_s > 0,5. Of course, the biggest role plays also the ratio of expected value, but for highly reliable systems we can also use this tendency.

When there is the absence of variability of the strength values of the bolts on the critical probability scale, the inclined sections of the curves between the points of inflection turn into vertical segments (fig. 3).

Thus, we can state that the random value of the critical factor increases when there is the constant value of the reliability index.

If the expected value of the bolt strength is equal, then the curve of the critical factor must be constructed accordingly to the formula (3).

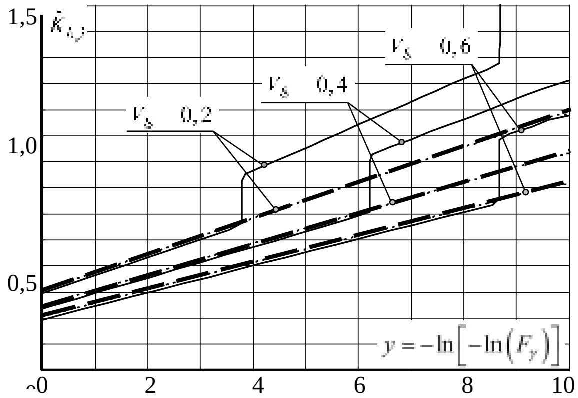

Figure 3 − Probability of a failure-free work of the joint of three bolts on critical probability scale in case of absence variability of the strength of the bolts

Conclusions

1. As part of the study, it was further developed the method for assessing the reliability of multi-bolt joints of the body sheets of steel cylindrical silos for situations when bolts in the mounting joint can belong to the three strength classes.
2. It was obtained the system of conditions and analytical dependencies for the calculation of the critical factor coefficient.
3. Using the example of graphical construction of the curves of the coefficients of the critical factor for different cases of failure sequences, it was illustrated that accidental presence of bolts of smaller strength classes in the mounting joint rapidly increases the risk of failure.
4. It was found that the index of probability of failure-free work of the homogeneous and quasi-homogeneous bolted joints begin to differ only when the coefficients of variation of the force value V_s > 0,5 increase.
5. Even if there is the absence of variability of values of the bolt strength, the random value of the critical factor will be increased accordingly to a constant value of the reliability index.

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COMPARATIVE ANALYSIS OF DESIGN SOLUTIONS OF METAL SILOS

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Concept of bulk material and complexity of storing it are explained. The dependence between the physical and mechanical properties of the loose material and type of construction in which it is stored. Structural elements considered silos and silos with conical and flat bottom are described. The common characteristics of metal silos for bulk materials are given. The classification of metal silos by type of housing design is given. The advantages and disadvantages of silos are described. The possibility of storage of bulk materials in cylindrical shells is analyzed depending on the type of construction. The history of occurrence of structures of spiral-fold silos is considered. The set of equipment for the construction of the housing of the spiral-fold silos is given, the step-by-step process of formation of the folding lock and features of the installation process are presented. The analysis of the structure is made and the advantages and disadvantages of spiral-fold silos are determined.

Keywords: bulk material, steel silo, spiral-fold silo, folding lock

ПОРІВНЯЛЬНИЙ АНАЛІЗ КОНСТРУКТИВНИХ РІШЕНЬ МЕТАЛЕВИХ СИЛОСІВ

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Висвітлено поняття сипучого матеріалу, розглянуто дуальність сипучого матеріалу та пояснено складність його зберігання. Роз’яснено, що поведінка сипучої речовини підпорядковується різним закономірностям залежно від способу зберігання: у замкненому сосуді чи в умовно нескінченному просторі. Проаналізовано залежність між фізико-механічними властивостями сипучого матеріалу з типом конструкції, в якій він зберігається. Розглянуто поняття силосу, сферу його застосування та переваги використання порівняно з відкритими або закритими складами. Описано конструктивні елементи силосу: дах, корпус, дно. Проаналізовано силоси з конусним та плоским дном. Наведено загальні характеристики металевих силосів для сипучих матеріалів, їх класифікація за типом конструкції корпусу. Розглянутий кожен тип конструкції корпусу силосу, описано особливості їх монтажу. Проведено аналіз переваг та недоліків силосів, наведених у класифікації. Розглянуто можливість зберігання сипучих матеріалів у циліндричних оболонках залежно від типу конструкції. Вивчено історію виникнення конструкцій спірально-фальцевих силосів. Висвітлено можливість зберігання різних матеріалів у металевих силосах спірально-фальцевого типу. Приведено приклад елеватора зі спірально-фальцевими силосами, який експлуатується в Україні вже більше 40 років. Наведено комплект обладнання для зведення корпусу спірально-фальцевих силосів. Розглянуто основні функції кожного агрегату з комплекту обладнання. Описано поетапний процес утворення фальцевого замка й особливості процесу монтажу корпусу. Зроблено аналіз конструкції в цілому та визначено переваги і недоліки спірально-фальцевих силосів. На основі порівняльного аналізу обґрунтовано загальний висновок про перспективність застосування сталевих спірально-фальцевих силосів для сипучих матеріалів.

Ключові слова: сипучий матеріал, металевий силос, спірально-фальцевий силос, фальцевий замок.

Introduction

Many modern technological processes are related to the processing of various bulk materials. In the chemical and metallurgical industries, as well as in the building materials industry, not only materials as coal, ore, gravel, sand, etc., are extracted, and in the process of processing primary raw materials produce a large number of new bulk materials such as coke, cement, slag, concentrates and others. It is also necessary to name an agroindustrial complex in with almost all products are bulk materials. A large number of granular materials makes it necessary to use a wide variety of machines, mechanical devices and structures for a faster and easier recycling process. One of the essential steps in the process - material storage. Ordinary composition of friable material is not a rational decision, as needs the considerable expenses of human labour for loading and unloading of material. The decision of this problem became silos for bulk materials.

Review of the research sources and publications

A large number of domestic and foreign scientists are in research of metal silos for bulk materials [1-2]. J. R. McCalmont was one of the first who classified silos and considered their characteristics [4]. Irena Selamovych and Robertas Balevichus considered in detail the analysis of the effect of rolling friction on the pressure distribution on the walls and velocities inside the fluid material [6]. Dietmar Schulz in his work examines general descriptions of friable solids, beginning from properties of stream of hard parts to behavior of stream of powders and friable materials in bunkers and silos. [7] G. L. Rozenblit in his work thoroughly examines rigid and flexible bunker for the coal industry [9]. V. V. Kachurenko thoroughly researched and described in his work behavioral and physical characteristics of the bulk material in silos [11]. D.O. Bannikov in his monograph presented authorial theoretical conception of co-operation of friable material with the elements of capacity building constructions [12]. A number of research in this area is increasing every year.

Definition of unsolved aspects of the problem

Nowadays there is quite a variety of types of bulk materials, the total number reaches several thousand. For their storage at different times concrete or metal welded silos were used [1 – 10]. Application of such types of constructions is related to difficult physical and mechanical properties of bulk materials. Such characteristics as tightness, strength and smooth surface of the wall made it possible to store a large variety of bulk materials.

Despite a large number of the works dedicated containers for bulk materials, some of which are listed above, almost all of them are focused on studies of metal silos for agriculture. Here are prefabricated silos with corrugated panels in bolted joints have proved best in our country and abroad. But the reliability of these structures are not fully investigated, it is confirmed by elevator accident associated with this type of construction.

Problem statement

The objective of this article is to review various structures of metal silos for bulk materials, the analysis of their advantages and disadvantages depending on the type of construction and method of installation.

Basic material and results

Before to begin the review of constructions of metallic silos, it is necessary to understand what it is a bulk material and what is the complexity of its storage.

Bulk materials is a material that has a pronounced grain structure with grain sizes, enabling flow of material in a confined space.

The specificity and complexity of storing bulk material is its duality. The bulk substance has properties inherent on one hand to continuous media (eg, the ability to exert pressure on vessel walls) and, on the other, to discrete media (eg, pressure dependence on material structure). Also the duality of granular substance is manifested in another respect. On the one hand bulk medium has properties of solids (eg, the ability to resist external forces) and at the same time, on the other hand, the properties of a liquid (such as the ability to flow or fill given volume).

It should also be borne in mind that the behavior of solids is subject to different laws depending on the method of storage, in a closed vessel or relatively infinite space. It is necessary to separately analyze static and dynamic behavior of granular material. After all, it is two rather different in nature phenomena that are associated with different structuring bulk material and therefore require the development of various theoretical models. Thus, in every case appear just quite different initial suppositions, as a result, eventual mathematical dependences quite often conduce to obvious contradictions.

Due to the features listed granular materials, it becomes clear the large number of types of metal structures silos. Fig. 1 shows the classification of metal silos for bulk materials. Consider it’s each type in details.

General characteristics of silos. Silos are used to store a variety of bulk materials in various industries. Often, they are used in grain elevators, cement and coal depots, metallurgy and chemical industry. Silos can be storage for the raw materials, semi-finished and finished products and intermediate tanks in the process.

Widespread use of silage storages in different areas is explained by a number of advantages of storage of bulk materials in silos compared to storing them in bunkers and open or closed warehouses equipped with cranes. Silos are compact buildings, with a high payload ratio that allow for larger capacities in relatively small building areas. This significantly facilitates the placement of storages in master plan.

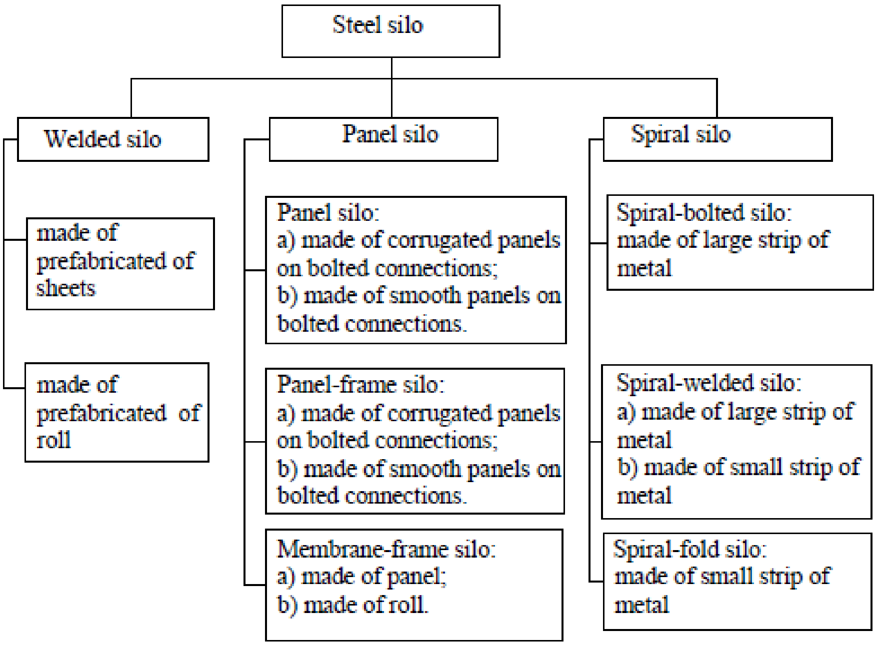


Figure 1 – Classification of metal silos

Thanks to the convenience of loading, unloading and high mechanical loading and unloading this type of closed storage facilities are the best in our time. Absence at the silos of free-surfaces, the methods of loading and unloading, that are used, provide reliable storage of materials and assist reduction of losses. In a silo there can be the organized drying of material that is kept, and also periodic moving for prevention of caking or spontaneous combustion.

Silo is a self-unloading capacitive structure in which the vertical height is 1.5 times greater than the diameter. For silos with a diameter of 18 meters or more, the ratio of the vertical part to the diameter may be smaller. The design is intended for long-term storage and overloading of bulk materials. Often, only the bearing capacity of the soil limits the height of the silos.

The main structural elements of the silo ar the roof, the housing, the bottom. The silo roof is a spatial conical structure, assembled from beam (rafters) and trapezoidal sectors. Trapezoidal roof sectors can have box-like rib edges along the edges, which give greater rigidity to the structure and have better protection against atmospheric moisture. The roof accepts snow load and is intended to protect against falling into the capacity of the silo of precipitation. Housing is a cylindrical shell, the construction of which depends on the type of silo. There are silos with flat and conical bottoms (fig. 2).

Welded silos (fig. 3 a, b) are a cylindrical shell, made of metal sheets welded together. According to the method of manufacture, these silos are divided into prefabricated sheets and rolls. The sheet silos are welded into a single housing made of individual sheets of metal. The roll silos are made of sheet metal, which is delivered to the construction site in the form of a roll whose height is equal to the height of the cylinder. During installation, the roll is positioned in a vertical position and unfolded around the perimeter of the annular foundation, forming a closed silo wall. The advantages of such silos are tightness and durability.

Due to this its are suitable for storage of various materials. Disadvantages of these structures are high material consumption and a large number of welds.

Panel silos (fig. 3 c) are a cylindrical shell is made of corrugated or smooth panels connected together with bolts. Corrugated panel profile helps save metal (due to thinner panels) and provides increased resistance to lateral load of silo. To compensate the loss of the bearing capacity of the panel through the corrugated profile on industrial silos establish additional ribs. The advantages of prefabricated silos are the opportunity to take large radial load on the material, no welds, high strength. The disadvantages of this option silos are a large number of bolted connections. This type of silo construction is most common in Ukraine and abroad, it is used in agriculture.

Panel-frame silos (fig. 3 d) have bent profile panels connected on bolts. In order to seal the silo, washers with gaskets are installed under the bolt heads. The roof of the silo is conical, it consists of annular and radial ribs, on which the flooring is arranged. The advantages of these silos are the strength and the absence of welded joints, the disadvantages are a large number of bolted connections, leaks, increased metal consumption and labor costs.

Membrane-frame silos [13] were developed at an institute in Moscow. The main structural element of these silos is a cylindrical membrane of membrane-frame type, made of tape 0.6 – 1 mm thick and 1250 mm wide. The tape is fastened in a spiral. The advantages of this option silos are covering membrane that takes only a tensile force, which allows full use calculated resistance; the disadvantages are large material consumption and complexity of installation.

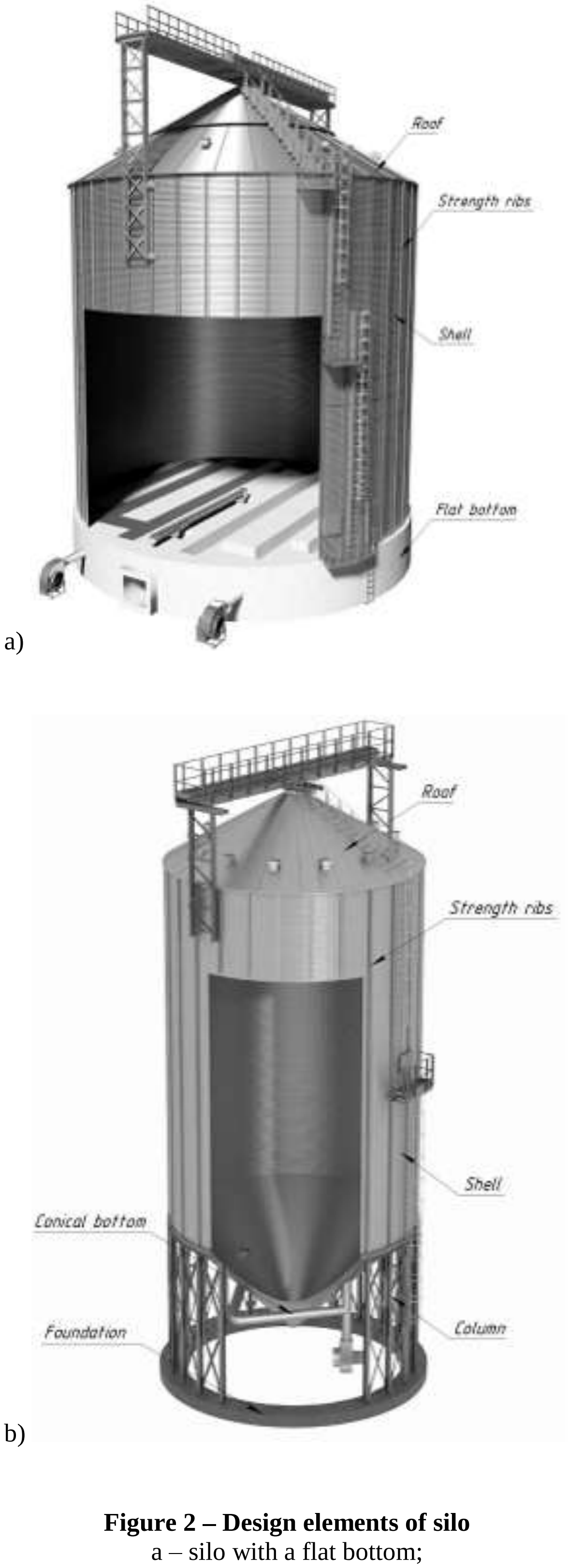


Figure 2 – Design elements of silo
a – silo with a flat bottom;
b – silo with conical bottom

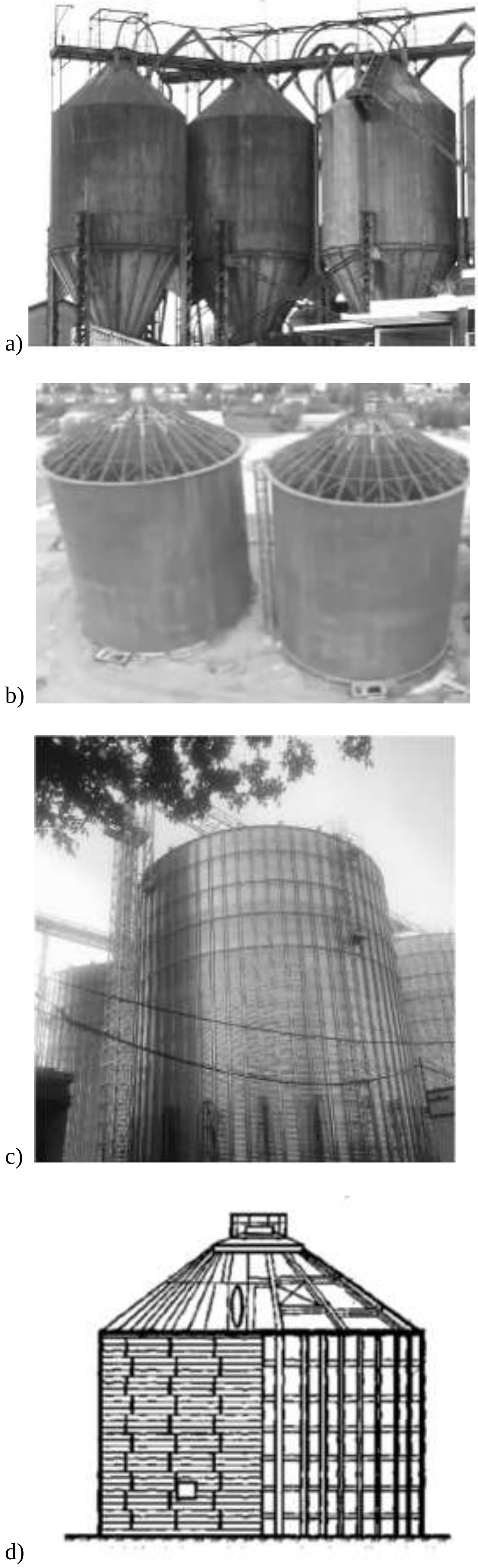
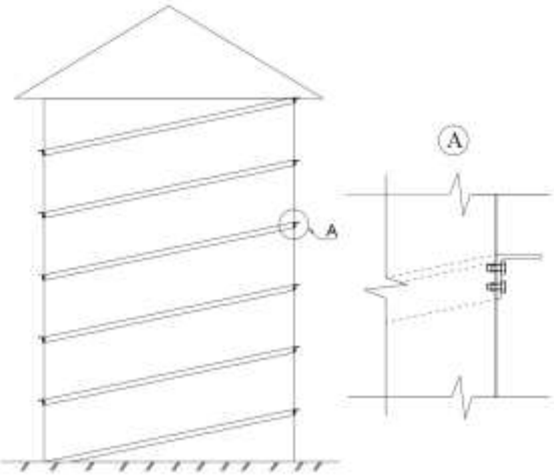


Figure 3 – Types of metal silos
a, b – welded silos;
c – panel silo; d – panel-frame silo

Spiral-bolted silo (fig. 4 a) includes a cylindrical shell made of a metal ribbon, curved in a spiral, the edges of which are connected by an ascending rib (corner or channel) by means of fixed-pitch bolts. The advantages of such silos are the absence of welded joints, the strength, the absence of additional processing of the edge of the roll blanks and the rising edges for the molding of the housing; the disadvantages are special equipment required for housing formation, additional equipment for bending and mounting of rising ribs, drilling of large number of holes for mounting of bolts; additional vertical ribs must be installed to ensure wall reliability.

Spiral welded silo (fig. 4 b) has a cylindrical shell made of a metallic strip, curved in a spiral, the edges of which are joined by welding. Geometric rolled billet thickness 1 – 4 mm, width 300 – 1250 mm. The joint of the workpiece edge must be tight. The advantages of silo are strength and tightness, disadvantages – a large number of welds made on the site, for their quality performance requires additional processing of the edge.

a)



b)

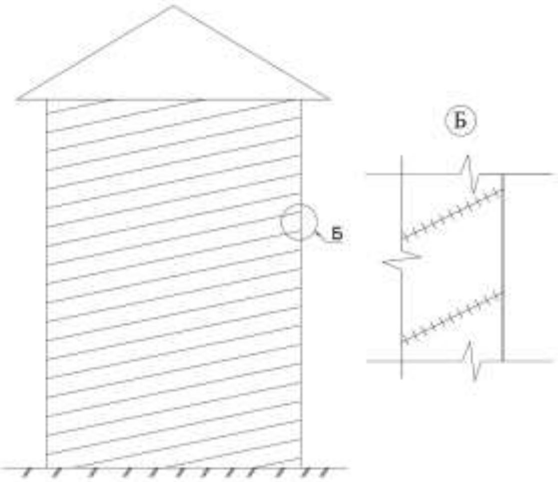


Figure 4 – Spiral silos:
a – the spiral-bolt; b – spiral welded

Spiral-fold silo (fig. 5) has a cylindrical body, which is a system of spiral connection of the steel strip by double folding. The design of the silo was developed in 1968 by a German scientist Xavier Lipp, who used the special equipment for processing of sheet metal and used it for the construction of silos spiral-fold [14]. The first silo was built in 1969 in Germany (fig. 5 a). Since the late 60s in Europe they began using the silos with this type of construction. Within ten years of study and research, in practice, this technology has proved successful, and since the early 70's large-scale production of galvanized steel spiral-fold silos has began.

a)

b)

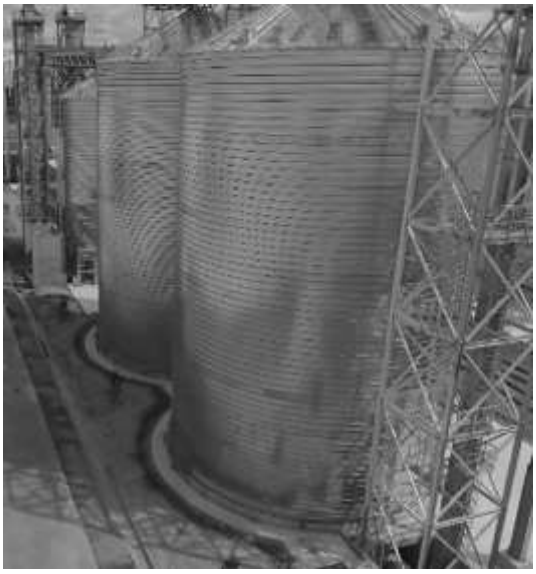


Figure 5 – Spiral-folded silos:
a – the first spiral-fold silo of Xaver Lipp, 1969;
b – spiral-fold silos (v. Korosten, Ukraine, 2018)

The advantages of these silos are:

– high precision, high degree of automation and speed of installation; reducing the time of installation and the required number of installers; minimizing the human factor in the installation, the whole process of silo installation is done automatically by a folding apparatus, the installation – from 4 to 6 days;

– when using the silo does not require any maintenance, can withstand a 7-point and larger earthquake, strong winds; warranty silo over 30 years; silo warranty for more than 30 years;

– good sealing, water resistance, complete absence of bolted connections and works related to waterproofing;

with a long shelf life can install a steam system for safe storage of grain, it is possible to store waste water, oil, petroleum products, cement and other materials; when using stainless steel it is possible to store food; alcohol, wine, flour, malt, molasses, sunflower oil, etc;

– saving steel, high strength, high grade zinc from 275 g/m^2 to 450 g/m^2;

– silo cost 30% lower then panel silos.

The disadvantages spiral-fold silos are:

– the maximum volume of silo may not exceed 10 thousand m^3;

– for installation, the silo assembly equipment must be transported to the construction site, which increases the cost of production.

One of the first in Ukraine in Yemylivka village, Kirovograd region in 1971 was commissioned the elevator with spiral-fold silos for storing grain (fig. 6). Despite its age, the company has been successfully operated to this day.

Figure 6 – Spiral-fold silos
(elevator in village Emylivka,
Kirovohrad region, 1971)

*Technology of installation of spiral-fold silos***.** The set of equipment for the construction of spiral silo (fig. 7) consists of: the un-coiler reel, a profiling machine, a folding machine and a support frame.

The main function of the profilling machine is the profiling and bending of a steel strip 495 mm wide and 2 – 4 mm thick by the silo diameter.

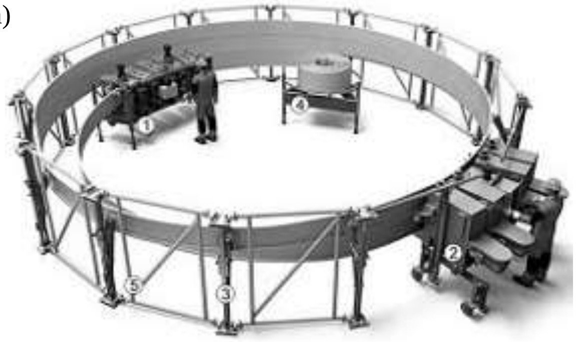
The folding machine is intended for rolling of steel tape and at the same time creation of a strong rolled seam 30 – 40 mm wide with a total thickness of 11 – 32 mm from the outside of the silo, each turn of which gives an additional rib of rigidity of the whole structure.

The speed of installation of the silo's shell is 3 – 5 m / min.

The installation process consists of the following stages: a circle is deposited on the foundation which has equal diameter of silo and on the contour on the outside supporting frames and folding machine are mounted. Inside the silo, there is a un-coiler rell and a profiling machine. At the center of the foundation at the appropriate height set a metal ring to which the roof rafters will later be attached. The metal tape is introduced into the profiling machine and the folding machine and further in a circle with the help of supporting frames, the winding process takes place with the formation of a fold lock (fig. 8).

Unique technology allows for compact and fast installation of high-strength and hermetical silos directly at the construction site, without the use of bolts and welded joints.

a)



b)

Figure 7 – Set of equipment for construction of spiral silo:
a – set of equipment in the working position, the process of coiling the silo's shell;
b – folding machine, profiling machine, support frame, un-coiler rell

a)

b)

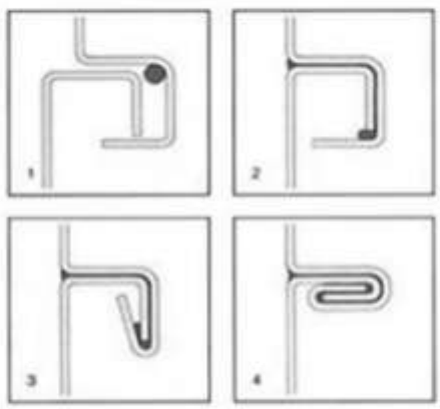


Figure 8 – The process of creating folding lock:
a – spiral-fold shell;
b – phased process of creating a fold lock

Conclusions

1. The physical and mechanical properties of bulk materials are analyzed.

2. The classification of metal silos for bulk materials is given.

3. The advantages and disadvantages of silos are considered, depending on the type of construction and method of installation.

4. Spiral-fold silos and method of their installation are discussed in detail.

On the basis of the comparative analysis, the general conclusion about the prospect of application of steel spiral-fold silos for bulk materials is substantiated.

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EXPERIMANTAL RESEARCHES OF THE CURRENT BURDENING COURSE PLATES ACHIEVEMENTS

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The article presents the experimental study results of flat intercolumn plates loading work peculiarities of the beamless overlapping developed system. The attention is paid to the design of the experimental designs of the bearing structure with the bevelled platforms of suspension on the outer perimeter of the span plates. The technique of performing experimental studies is presented. The conducted researches enabled to establish the nature of deformation and destruction of intercolumn plates as a separate element in the developed system of beamless overlapping. In this case, the magnitude of the compression deformations decreases to the point where the test specimens rest on the supporting blocks. It confirms the assumption of the transfer of load from the spacers to the intercolumnar on the principle of "linear hinge". Attention is paid to the fact that the achievement of the bearing capacity is not accompanied by the process of destruction, but is characterized by significant movements of the flying part of the plate in the vertical plane.

Keywords: displacement, bearing capacity, deformed state, experimental research, unbounded overlap

ЕКСПЕРИМЕНТАЛЬНІ ДОСЛІДЖЕННЯ МІЖКОЛОННОЇ ПЛИТИ ЗБІРНОГО БЕЗБАЛКОВОГО ПЕРЕКРИТТЯ

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У статті наведено результати експериментального дослідження особливостей роботи під розрахунковим навантаженням плоских міжколонних плит розробленої системи безбалкового перекриття. Приділено велику увагу конструюванню дослідних зразків несучої конструкції. Особливістю плити, що досліджується є скошені майданчики спирання по зовнішньому периметру пролітних плит та скошені опорні ділянки. Описано конструкцію та технологію виготовлення додаткового устаткування, що імітацує спирання міжколонної плити вищого рівня у вигляді залізобетонних опорних масивних блоків. Для неуможливлення зміщення блоки об'єднано сталевими прутами. Дослідні конструкції плити і допоміжного устаткування виготовлено на обладнанні та в умовах діючого заводу залізобетонних виробів. Наведено методику виконання експериментальних досліджень із вказуванням методів і засобів вимірювання геометричних та фізичних параметрів, що характеризують напружено-деформований стан та несучу здатність дослідної плити. Проведені дослідження дали змогу встановити характер деформування і руйнування міжколонних плит як окремого елемента у розробленій системі безбалкового перекриття. Аналіз характеру деформування свідчить, що деформації розтягу на нижньої грані дослідної плити наростають швидше ніж деформації стиснення (верхньоїьої грані). При цьому відбувається зменшення величини стискаючих деформацій до місця спирання дослідних зразків на опорні блоки. Це підтверджує припущення про передачу навантаження від пролітних плит на міжколонну за принципом «лінійного шарніру». Звернута увага на те, що досягнення несучої здатності не супроводжується процесом руйнування, а характеризується суттєвими переміщеннями пролітної частини плити у вертикальній площині.

Ключові слова: переміщення, несуча здатність, деформований стан, експериментальні дослідження, безбалкове перекриття

Introduction

Modern tasks of construction development are becoming new requirements for the production of building structures and their modernization on the basis of scientific and technological progress. A significant effect in the implementation of new modernized structural solutions can be achieved due to the optimal combination of physical and technical parameters of elements in the scheme "design material technology". Under this scheme, the bulk of the savings are formed, firstly, at the expense of the widest use of existing potential of precast concrete structures, in particular the use of circular hollow slabs, and secondly, the use of progressive reinforced concrete cones - structures that combine the advantages of steel and concrete and allow to reduce the structural height of the frame elements.

Nowadays, joists are used in the construction of joistless, ungrounded, and non-rigid overlapping structures. Such design systems allow the construction of arbitrary configuration buildings in the plan with different planning and planning solutions. A further step in the modification of prefabricated and prefabricated monolithic frames of buildings and structures is the combination of prefabricated circular hollow slabs, their modifications and steel-lizoboton beamless frame. Tasks aimed at finding rational parameters of such structures, investigating their strength and deformation properties and implementing the results in construction are appropriate and relevant.

Review of research sources and publications

Along with the existing types, new progressive structures of reinforced concrete have been developed to reduce the cost of installing console construction, to refrain from the formwork and additional racks and to increase the speed of installation. Various systems of non-beam overlaps have been widely considered [1]. They all have advantages, but not without disadvantages. Material and labour costs for their installation are significant.

The system of beamless overlapping with modified multi-hollow plates [2, 3], designed by the authors, is able to provide optimum due to the plates layout joints total length and strength and does not require large material and labour costs for the installation and replacement of joints between individual plates. This is achieved by the fact that precast, intercolumnar and span slabs are used in precast concrete precast slabs, with slabs along the perimeter having sloping lateral faces forming a platform for supporting adjacent slabs [4]. The overall stiffness of the flooring is achieved by welding together the mortgages that are pre-seen on all slabs. To put into practice the construction of such overlapping systems, the necessary proposals for their design and, in particular, the calculation of the strength of the individual elements and the whole system as a whole are necessary.

Definition of unsolved aspects of the problem

At present, there is virtually no data on the robot of individual plates in the composition of beamless overdigging. As a prototype, an over-sized slabless slab of slabless overlap was selected, which enabled obtaining the most complete information on the object of study.

Problem statemen

The purpose of the experimental researches is to establish a true stress-strain state and to determine the load-bearing capacity of the inter-column slabs of the flat beamless overlap.

Basic material and results

A program of their experimental studies was developed to identify the peculiarities of interconnector plates work under the influence of external loading (Fig. 1). According to the adopted program of experimental studies, a series of prototypes were made. Concrete formulations and reinforcing bars, which were available at the Svetlovodsk Plant of Reinforced Concrete Products, were used for the samples. The thickness of the test plates was taken 220 mm. Special markings were made on the surface of the specimens to accurately fit the design position. The experimental specimens were filled with C25/30 concrete in strength.

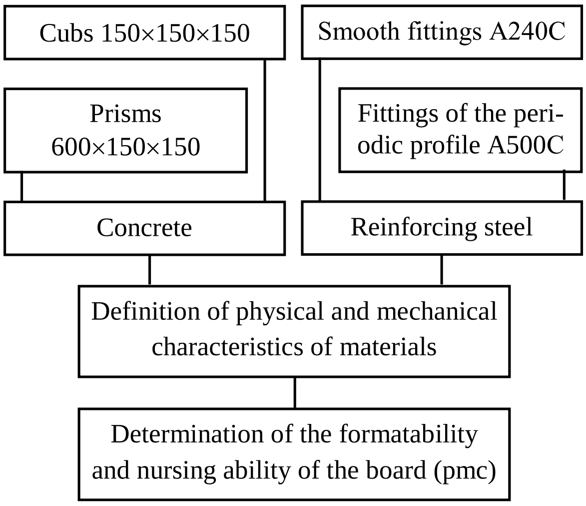

Figure 1 – Program of experimental research of inter-column slabs of the basal overlapping system.

Designs of samples and technology of their manufacture

Intercolumn plates were made and tested at the factory of reinforced concrete products in Svetlovodsk.

Figure 2 shows the test plates on which the measuring marks are glued using the BF-2 adhesive. Marks were used to improve the accuracy of the positioning of control points and characteristic points in photographs. The photographic method is interesting and promising in the first place because it is a non-contact method of determining the deformation of bodies. The essence of the method lies in the measurement of the prototype before and after deformation between point marks, which in our case were marking marks. In this method, photographing of marks was carried out at a distance, so there is no need for direct contact of the devices with the body, as well as the essential positive method is to measure the deformations of distant or complex body shapes.

Figure 3 shows a diagram of the location of the measuring devices on the surface of the test specimens of the intercolumn and flight plates of the series, respectively PMC and software.

In order to be able to mount in the working position of the test specimens of inter-column (PMK) and fly-in (SO) slabs, two concrete blocks with a cut one upper edge were made. To prevent movement during loading, these support blocks were joined by two steel rods. The latter were omitted in the pre-formed holes (Fig. 4).

Figure 2 – Experimental design of the intercolumn plate on the test bench.

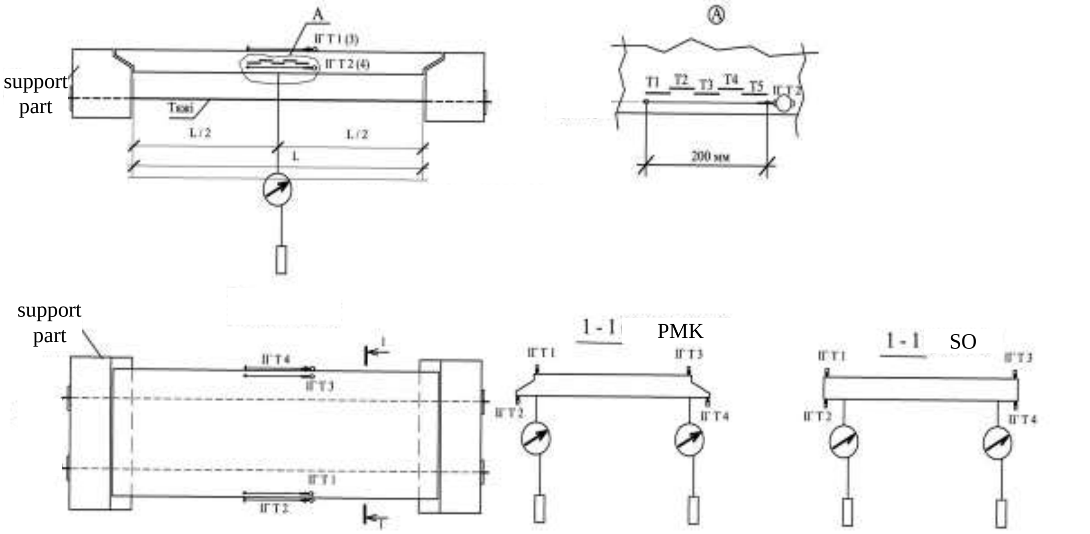

Figure 3 – Arrangement of measuring devices on the intercolumn plate

Figure 4 – Supporting blocks for testing of PMC series plates

Inventory cargo blocks with a calibrated weight of 1.96 tons were used as a useful load for the PMK test plates. The overall dimensions of such blocks were 600×600×2200 mm.

The results of the experimental pre-investigations of the developed PMC intercolumn plates, indicated a number of disadvantages of ordinary working reinforcement. This plate design had sufficient bearing capacity, but displacement and fracture toughness were not sufficiently secured, as evidenced by the magnitudes of the parameters that characterize them. In the practice of designing reinforced concrete structures, the practice of replacing conventional strength reinforcement with high strength is widely used, such replacement allows the use of pre-stressed working reinforcement.

To evaluate the efficiency of the use of pre-stress in the intercolumn slabs, prototypes of such slabs were made (Fig. 5). The following used: concrete of class C16/20; armature valve ∅14 A800, mounting – ∅6 AII.

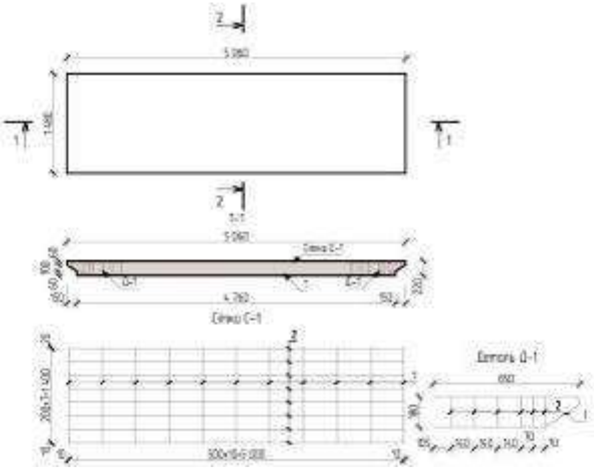

Figure 5 – Reinforcing frame and PMC parts

The laboratory tests of reference specimens of cubes and prisms obtained mechanical properties of concrete. The cubic strength of the control samples was $f_{ck,cube}$ = 26,4 MPa, prism – $f_{ck,prism}$ = 15,0 MPa, the initial modulus of concrete E_{ck} = 23,0 GPa, the coefficient of transverse deformation – ν = 0.19. The characteristics obtained correspond to the design class of C16/20 concrete.

The steel physical and mechanical properties determination was carried out by testing 3 standard samples in the form of rods.

On the sections of the armature were placed electro-strain gauges on two opposite sides. Adjusted cores were tested on a universal tensile test machine.

The stresses, which corresponded to the yield strength and the temporary resistance of the material, were determined according to the diagram σ – ε. The resistance of steel f_y and f_u was assumed to be the average of the minimum of the test specimens. The modulus of elasticity of the steel was determined by the diagram, E_s = 2,1×10^5 MPa

Figure 6 shows a graph of the defocation distribution on the concrete surface at the level of the upper compressed fibre and at the axis level of the lower working armature. Initially, up to 0,4 $F_{\max}$ the increment of deformations occurs in proportion to the increment of the load. It should be noted that the graphs have several points at which the deformation increment changes depending on the load value. This can be explained by the specific procedure of loading the test plate with the help of inventory loads. The weight of each such cargo amounted to 16,3 кN (Fig. 7).

The graphs can highlight the moment when a substantial increase in strain in the stretched zone is doubled by the same increase in deformation of the compressed zone. At this point, the sum of the total extracurricular effort amounted to 163 кН.

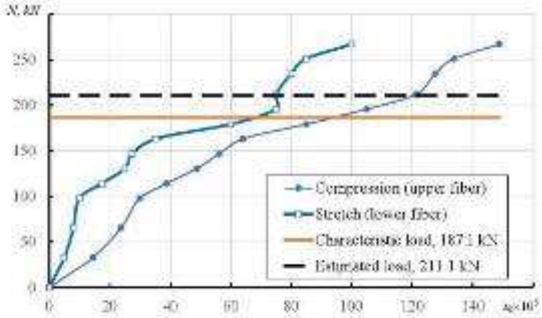

Figure 6 – Development of relative deformations of the upper fiber and at the level of the lower working reinforcement

During unloading, the test plate was deformed in the opposite direction. It led to the closure of the cracks in the stretched area, but some residual deformation was preserved. So in the cramped zone they reached the magnitude of 47×10^{-5}, and in the stretched – 30×10^{-5}.

It should be noted that this character of the development of longitudinal deformations is confirmed by the graph of the change in the magnitude of the vertical displacements of the middle cross-section (Fig. 8) of the intercolumn plate with the prestressing of the lower working armature.

Increasing the total load to 163 kN led to the appearance of "hair" cracks (Fig. 9). Crack propagation reaches 40% of the height of the cross section from the bottom face.

As the load increased, the cracks length increased and the compressed zone decreased by 5–7%. Cracks were also manifested on the lower face of the slab. From Figure 9 it can be seen that the cracks are evenly revealed along the entire cross-section.

Figure 7 – Experimental sample of inter-column slab with pre-stressed working fittings during in-test

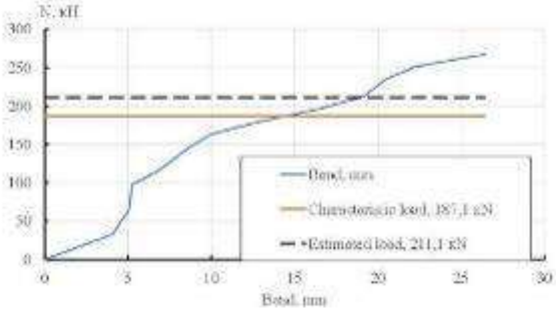

Figure 8 – Mean section deflections depending on the magnitude of the total load, such as pre-stressed PNC

The variation of the size of the compressed zone of the middle cross-section is shown in Fig. 10. According to these graphs, it can be seen that from the beginning of loading to loading of 0.4 F_{max} = 97.8 kN the height of the compressed zone was 68% of the height of the section of the plate. With the load increasing to 163.0 kN, the height of the compressed zone was 59%. Then it decreased to almost 50%.

The average value of the maximum load according to the test results for the interconnector plates was *F* = 267.3 kN (bending moment in the average cross section *M* = 151.8 kN·m).

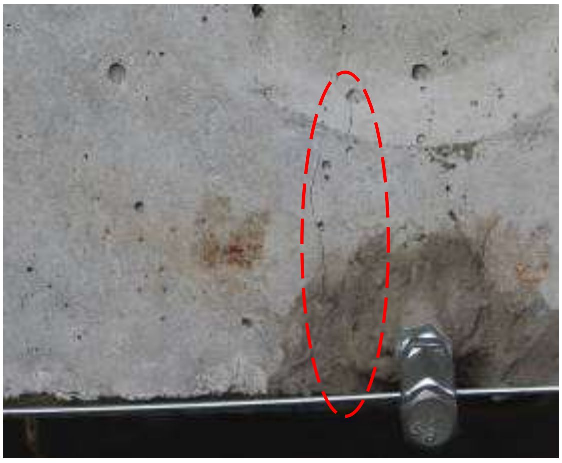

Figure 9 – Occurrence of the first cracks near the average cross section at a total load of 163 kN, such as pre-stressed PNC

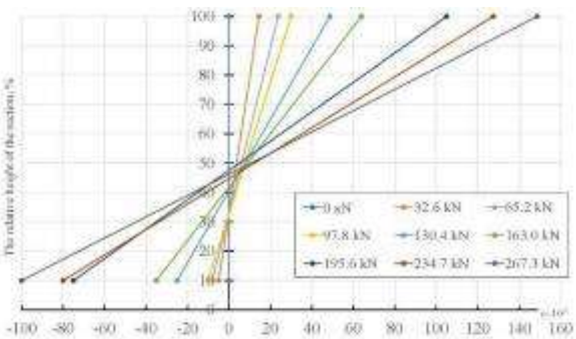

Figure 10 – Changing the height of the compressed zone depending on the magnitude of the total load, such as pre-stressed PNC

The results in Table 1 refer to the average cross-section with respect to the plate span. The forces *N* (kN), relative deformationї $\varepsilon_{cm}\times10^{-5}$ and $\varepsilon_{sm}\times10^{-5}$ and deaths *f* (mm) were measured during the experimental studies. The magnitudes of the outer bending moment *M* (кН·m) are calculated in accordance with the actual loading of inventory loads, taking into account their location and the corresponding quantity. The curvature was determined for the corresponding average cross-section depending on the deformation of the plate materials.

Dependences of changes in stresses in concrete of compressed and stretched zones at different loading stages were constructed according to the deformations measured during the test and according to the concrete diagram. The graphs are presented in Fig. 11.

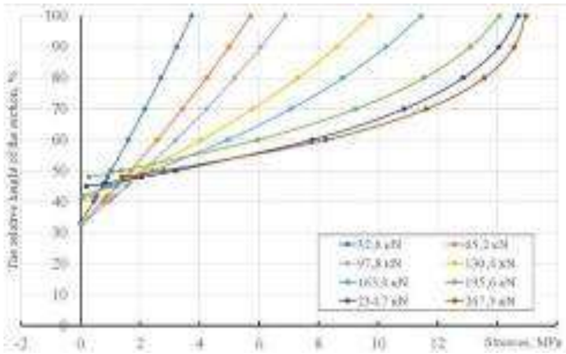

Figure 11 – The change in the stress state of the concrete in the average cross-section depending on the value F of the sample of PNA with prestress.

Table 1 – Results of experimental studies of the interplanar PMK plate

№	*N*, kN	*M*, kN·m	$\varepsilon_{cm}\times10^{-5}$	$\varepsilon_{sm}\times10^{-5}$	*f*, mm
0	0,0	0,00	0,0	0,0	0,00
1	32,6	32,09	14,5	5,0	4,07
2	65,2	52,14	23,8	8,0	5,07
3	97,8	60,71	30,0	10,0	5,24
4	114,1	77,36	38,8	17,5	6,71
5	130,4	86,78	48,8	25,0	7,72
6	146,7	91,14	56,3	27,5	8,73
7	163,0	101,09	64,0	35,0	9,99
8	179,3	117,13	85,0	60,0	12,81
9	195,6	130,12	105,0	75,0	16,38
10	211,9	137,07	121,3	75,0	19,12
11	234,7	137,84	127,5	80,0	20,47
12	251,0	138,82	133,8	85,0	22,25
F_{max}	267,3	151,80	148,8	100,0	26,42

Thus, the load-bearing capacity of the inter-column pre-stressed plate was 258.6 kN. The design of the plate during the experiments did not suffer destruction, but the value of the relative deflection exceeded the limit value in 1/200. Therefore, as a bearing capacity, an effort is taken that corresponds to the magnitude of the regulatory deflection. In this case, the theoretical bearing capacity of the inter-column plate with pre-tension was 211.1 kN. The difference was 21.3%. It can be stated that the theoretical non-shear capacity, which is calculated by the standard methodology, enables to establish with sufficient accuracy the load-bearing capacity of such plates.

Table 2 compares the bearing capacity, the boundary deformations of the extreme fibers of the median section and the deflection of the experimental design of the intercolumn plate with the pre-stressed working reinforcement with theoretical values.

The longitudinal deformation of the compression of the upper compressed fibre in the limit state is understressed by 8%, and the deformation of the lower tensile fibre exceeds the ultimate deformation of the concrete by 35%. The experimental pro-gins almost coincided with their theoretical values.

Table 2 – Comparison of experimental results and theo-theoretical data

	Carrying capacity, kN	Deformation, $\varepsilon\times10^5$		Bend, mm
		compression	stretching	
Experimental	267,3	149	100	26,4
Theoretical	211,1	162	65	26,0
Difference, %	+ 21,3	– 8,7	+ 35	+ 1,5

Conclusions

The experimental research program was designed to consider the possibility of building structures existing production material base using. It enables to design and manufacture life-size prototypes of real load-bearing structures of a flat, girderless floor. Materials of structures (steel and concrete are used in real load-bearing structures. The test rig is certified.

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METHODOLOGICAL ASPECTS OF ASSESSING THE STEEL FRAMES RELIABILITY

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The article highlights the proposed algorithm for evaluating the reliability of steel frames. In particular, it is possible to analyze the reliability of the most likely failure mechanism. Separate assumptions that determine the sequence of application of the limit equilibrium method are presented. A method for determining the reliability of statically indeterminate steel frames in the plastic stage is presented. This method provides an opportunity to determine the probable mechanism of destruction. The ultimate equilibrium method is used to calculate the forces at the final stage of destruction. In the work, the real mechanism of destruction is understood as a mechanism for which the work of external forces to create it is the least. It is revealed that the real mechanism of destruction is approaching the beam or floor elementary mechanism.

Keywords: reliability, failure mechanism, steel frames, calculation.

МЕТОДИЧНІ АСПЕКТИ ОЦІНЮВАННЯ НАДІЙНОСТІ СТАЛЕВИХ РАМ

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Стаття висвітлює запропонований алгоритм оцінювання надійності сталевих рам. Зокрема, приведена можливість аналізу надійності за найбільш ймовірним механізмом руйнування. Представлені окремі припущення, які визначають послідовність застосування методу граничної рівноваги. Представлено методику визначення надійності сталевих статично невизначених рам у пластичній стадії. Така методика надає можливість визначення ймовірного механізму руйнування. Методом граничної рівноваги розраховуються зусилля на кінцевій стадії руйнування. В роботі під реальним механізмом руйнування розуміється механізм для якого робота зовнішніх сил по його створенню буде найменшою. Виявлено, що реальний механізм руйнування наближається до балкового або поверхового елементарного механізму. В статті розрахунки на початковому етапі виконані в детерміністичній постановці для сталевих рам методом граничної рівноваги Отримані чисельні граничних моментів, зокрема для граничного етапу реального механізму руйнування. Розроблена програма, яка надає результати розрахунку сталевих рам за двома напрямами. Один напрям орієнтований на суто задані чисельні значення жорсткостей. Окремо визначаються граничні моменти та значення моментів у перерізах. Такий метод визначає реальну картину руйнування. Інший напрям розрахунку направлений на оптимізацію розрахунку та мінімізацію показників маси конструкції. Представлена можливість розгляду різних випадків руйнування сталевих рам. А головна перевага даного методу знаходження найбільш реального механізму руйнування. Такі методи є важливим елементом проектування нових сучасних конструктивних форм та визначення слабких місць уже існуючих конструкцій.

Ключові слова: надійність, механізм руйнування, сталеві рами, граничні моменти.

Introduction

Considering the general information about the reliability assessment of steel frames, it is noted that the distribution of forces during plastic destruction does not depend on the loading history, on the behavior of the structure before its complete plastic destruction. Therefore, for the calculation of steel statically indeterminate frames made of elastic - plastic material, we can only consider the phase of exhaustion of the bearing capacity of structures, their plastic destruction. This position is used in the calculation using the limit equilibrium method.

Review of research sources and publications

In general, norms in Ukraine [1] set the general principles for ensuring the reliability and structural safety of buildings and structures. These rules apply to the search, design, construction and disposal of buildings and structures, regardless of their purpose.

The issue of reliability of steel frames is presented in the works of various leading scientists of the world. In particular in the work [2] concerns the underpinning system reliability calibrations that enables the implementation of the next generation of system-based design-by-analysis method of steel rack frames, i.e., a design approach where analysis and capacity checks are carried out in a single step by using fully nonlinear analysis. The paper details the design framework of the new approach, referred to as the Direct Design Method (DDM), and derives system strength statistics for five typical configurations of rack frames using Monte-Carlo simulations, considering the randomness of geometric and material properties. The nominal models of rack frames are developed in accordance with the Australian Standard AS4084. The mean-to-nominal ratios (bias) and coefficient of variation of the system strengths are obtained, and is used in the companion paper to derive the system resistance factors consistent with a given structural reliability. In the second of two works [3] introducing the underpinning structural analyses and reliability studies that implement the system-based design-by-analysis method of steel rack frames, referred to as the Direct Design Method (DDM). The present paper presents the reliability analyses and derivation of system reliability index (β) versus system resistance factors (ϕ_s) curves. Results are presented for nominal system strengths as per Australian Standard AS4084 for several nominal models, including models that exclude sectional imperfections, and models without member and sectional imperfections. The effect of model uncertainty is also assessed. A detailed example of the DDM applied to the design of a rack frame is presented and the benefits of the DDM are demonstrated when compared to the traditional design approach which is based on elastic analysis. In the [4], the authors note that high strength bolted end-plate connection is main type of the connections used widely in industrial construction. There are two kinds of end-plate connections in steel portal frames: flush and extend end-plate connections. The main initial imperfection of bolted end-plate connection lies in which the thickness of end-plate and column flange can't meet the code provisions based on a field investigation. Considering the effect of initial imperfection, the actual behavior of end-plate connections in steel portal frames is seldom fully rigid. The true behavior of the connections is usually semi-rigid. Neglecting the real behavior of connections in the analysis may lead to unrealistic predictions of the response and reliability of steel portal frames. The paper [4] considers the effects of semi-rigid behavior of the connections in the finite element analysis and reliability analysis of steel portal frames. Assuming that the loads and the resistance of members are random variables, then the Monte Carlo simulation technique is used to estimate the failure probability of steel portal frame system. The results confirm that the thickness of end-plate has a significant effect on safety of steel portal frame. Integrated structural designs, with consideration of system reliability for steel portal frames comprising tapered members, are studied in the paper [5]. The reliability-based integrated design (RID) directly checks the structural system limit states and the corresponding system reliability, based on structural nonlinear analysis. The nonlinear integrated analysis model, the semi-analytical simulation method employed for system reliability assessment, the development processes of RID format and the design application of RID formula and curves are presented in this paper. Design examples and comparisons among three different design formats demonstrate that RID proposed in this paper is of certain and consistent system reliability levels, and provides a feasible way for structural engineers to improve the design quality and flexibility of steel frame structures. Progressive collapse is an important failure mechanism that must be considered in the design of critical and essential buildings [6]. For steel moment structures, beam-column joints, which act as transportation hubs of forces, are crucial members to resist progressive collapse. This research [6] investigated the effectiveness of beam-column joints with cast steel stiffeners (CSS) in steel moment frames for progressive collapse resistance. A computationally efficient macromodel that can be used for routine design of steel moment frame buildings with CSS was developed in this paper. The developed model, which considers the deformation of joints with CSS and the catenary action effects during progressive collapse, was validated using a 3D solid finite-element model. Subsequently, the macromodel was utilized to calculate the proper dynamic increase factor for steel moment frame structures with beam-column joints using CSS. The results show that the frame with CSS is less vulnerable to gravity-induced progressive collapse than frames with welded beam-column joints without stiffeners. The proposed macromodel is effective and a dynamic increase factor of 1.6 is suitable for dynamic progressive collapse analysis of steel moment frame structures using beam-column joints with CSS.

The work [7] reviews the state-of-art in progressive collapse studies on framed building structures. Such types of failure start with a local damage which extension increases, up to the whole structure. First emphasis is placed on the current techniques to study collapse propagation, i.e., numerical, experimental and analytical. In particular, the various numerical methods found in the literature are reported and discussed and the experimental studies and technologies involved in the laboratory tests are listed and compared. As reviewed, the method of analysis depends on the collapse mechanism and the triggering event. Thus, an in-depth review of the collapse typologies is proposed. Pure and mixed progressive collapse mechanisms are discussed and debated. The various triggering events, their modeling and their effects on the framed structures are examined. Details on the available literature on multi-hazard scenarios are provided. Finally, robustness techniques against progressive collapse are summarized, compared and contrasted. The paper [7] concludes with an ambitious comprehensive list of open questions and issues covering different aspects of future needs.

Definition of unsolved aspects of the problem

In the literature, there are not enough disclosed questions on the formulation of the term "survivability", not presented a single algorithm for calculating the survivability of building structures. Also, the literature does not consider the dynamic components of the load on steel redundant frame.

Problem statement

When determining the reliability of steel statically indeterminate frames by the method of limit equilibrium in this paper it is necessary to make some assumptions:

1. Application loads are of the quasi-static type. Dynamic defects and re-variable loading were not considered.

2. The construction Material is ideally elastic-plastic and obeys the Prandtl diagram. It can be noted that the ideal plasticity is the first approximation for the real behavior of the structure beyond the elastic limit and corresponds to this method of limiting equilibrium is quite suitable for solving problems of determining the load-bearing capacity. Considering the actual operation of a statically indeterminate steel frame, it can be concluded that it is close to an ideal elastic-plastic one. The strength distribution of the material was assumed to be normal, which corresponds to the experimental data obtained during the tensile testing of steel samples.

3. Deformations at destruction are small, so the equilibrium equations are made for an underformed scheme. It is known that this assumption is always accepted when elastic calculation of structures does not cause doubts about the insignificance of errors. It is assumed that when considering one-, two-, and three-story multi-span frames, horizontal deformations are small.

4. The sections of the elements have an ideal shape, for such a section, the plastic section occurs simultaneously over the entire area, as a result of which the zone of the plastic hinge is limited to a point. This assumption enables to assume that the destruction mechanism is a kinematic chain consisting of solid particles connected in certain places by hinges. This assumption makes it much easier to kinematically consider the system, the closer to reality the cross-sections of elements are closer to the ideal cross-section, which is most acceptable in metal structures, because thin-walled profiles close to I-beams are used.

5. The main acting forces are bending moments, and the basis for determining the bearing capacity is the strength criterion. The action of transverse forces in the formation of the destruction mechanism is not taken into account, since their influence is small. Accounting for the longitudinal force for columns is possible and is considered as a fraction of the maximum bending moment.

Basic material and results

When determining the reliability of structures, a reasonable approach should be taken. This approach takes into account all aspects that determine its load-bearing capacity and Express real work under current loads. Based on the results of probabilistic calculations and experimental studies, a number of the above-listed assumptions can be accepted for the reliability of the results. The reliability of statically indeterminate steel frames operating in the plastic stage can be assessed with sufficient confidence by examining one of the most likely mechanisms of structural failure. The proposed calculation method allows us to obtain this one most likely (true) mechanism of structural failure. To determine the forces in the final phase of destruction, the method of limiting equilibrium is used, which can be expressed as the equality of the virtual work of external A_{sx} and internal forces A_{in}:

$$A_{sx} = A_{in} \; ; \tag{1}$$

$$\sum_j P_j f_j = \sum_k M_{pl,k} V_k \; , \tag{2}$$

where P_j is the value of the *j-th* external load in the form of a concentrated force, distributed load, or moment;
$M_{pl,k}$ is the plastic moment in the *k-th* section when forming a plastic hinge;
f_j is the turns or moves of nodes;
v_k is the turns of rods in the *k-th* section.

For a static formulation of the problem of determining internal forces when the load-bearing capacity of the frame is exhausted, the one for which the work of internal forces reaches the lowest value is accepted out of all the statically acceptable ones. A mathematical model of the problem of calculating an elastic-plastic system, characterized by a single parameter *min* M_o, from a one-time simple load, can be expressed

$$\left.\begin{aligned} \mu_i \cdot M_o - M_i \ge 0 \\ \mu_i \cdot M_o + M_i \ge 0 \end{aligned}\right\} i = 1,2,...n \; , \tag{3}$$

where M_i is the moment acting in the I-th dangerous section;
M_o is the parameter of the maximum bending moment;
μ_i is the component of the vector of coefficients of ratios of the system's load-carrying capacity characteristics are set

$$\sum_{i=1}^{n} \propto_{ij} M_i = P_i \ , (j = 1, 2, \dots (n\text{-}k)), \qquad (4)$$

where $\alpha_{i,j}$ is the element of the matrix of equilibrium conditions;
P_j is the component of the external load vector;
k is the degree of static uncertainty of the system;
n is the number of suspected dangerous sections.

Condition (3) is considered as a linear programming problem that is solved by the simplex method. For the elastic-plastic calculation of flat frames, a program was written in the FORTRAN language, in which two calculation modes are performed.

One direction is focused on strictly specified numerical values of stiffness. Separately determined by the marginal moments and the values of the points in the sections. This method determines the real picture of destruction.

Second, when the frame is optimized for a given number of stiffness ratios ($\le n$) to obtain the minimum moment distribution by changing the stiffness ratio, the result is a minimum weight design. Let's focus on the first mode, which shows the true mechanism of destruction (the most likely). To perform calculations of statically indeterminate steel frames by the simplex method, it is necessary to create elementary equilibrium equations (in the static formulation of the equation together) for the specified geometric dimensions of the structure, the ratio of stiffness characteristics, the magnitude and direction of external loads. In connection with the different methods of composing equations in the literature, we present a generalized version of their Assembly (Fig. 1 - 2).

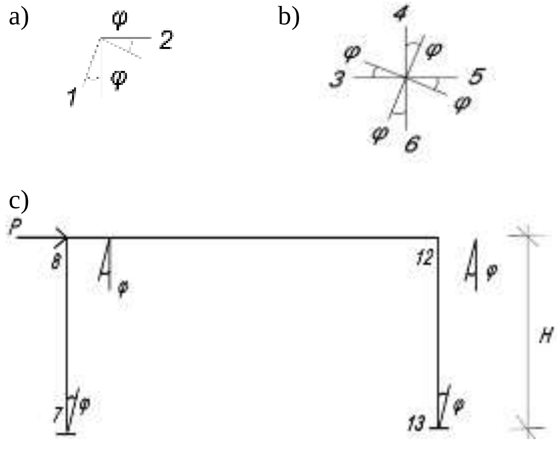

Figure 1 – Nodal (sum of moments in a node):
a) $M_1 + M_2 = 0$; b) $M_3 + M_4 + M_5 + M_6 = 0$;
c) surface (sliding),
(the shear force of the floors or stairs):
$M_7 – M_8 - M_{12} - M_{13} = P\ H$

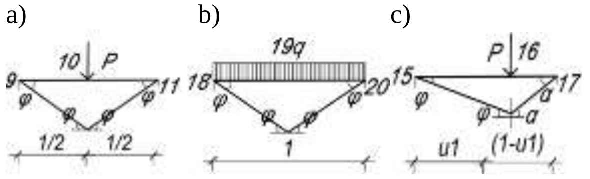

Figure 2 – Beam (the value of the bending moment for the cross section on the rod through the moments at the ends:
a) $M_9 + 2M_{10} + M_{11} = PL/2$;
b) $M_{18} + 2M_{19} + M_{20} = qL^2/2$;
c) $M_{15}+M_{16}+uM_{17}=PL(1-u)$

The calculation of steel statically indeterminate frames by the method of limiting equilibrium in the initial stage is performed in a deterministic setting. In the process of calculations, we obtain the values of the limiting moments in the frame sections for the boundary phase of destruction for the true mechanism. A true mechanism is a mechanism for which the work of external forces to create it is the least. For this calculation, the true mechanism is the one for which the value of the Mo limit moment is the smallest. Probabilistic characteristics of strength and load are introduced at the final stage of calculating the probability of failure of the system as a whole. Based on the joint solution, a method for calculating the reliability of statically indeterminate frames is obtained, in which the conditions for plasticity hinges have the form of equations describing hyperplasticity in $(k + 1)$ – dimensional hyperspace

$$\sum_{j=1}^{k} M_{ij} x_j + qM_{io} \le M_{i,pl} \ , (j = 1, 2, \dots) \qquad (5)$$

where $M_{i,pl}$ is the limit moment in the *i-th* section;
$M_{i,j}$ is the moment in the *i-th* section of the main system from the excess unknown $x_j = 1$;
M_{i0} is the moment in the i-th section from external loads q, whose parameter is assumed to be $q=1$.

The intersection of the hyperplanes defines the vertex of the polyhedron of conditions for which the maximum load value is determined

$$q = q_{max} \ . \qquad (6)$$

From the solution of $(k+1)$ linear equations (5) with substitution of the average limiting moments $M_{r,pl}$ in the right part and transition to the area of random parameters, we obtain the mathematical expectation of the frame strength as a whole in the space of the load parameter

$$\bar{q} = \sum_{r=0}^{k+1} \frac{A_{r,k+1}}{D} \bar{M}_{r,pl} = \sum_{r=1}^{k+1} \frac{A_{r,k+1}}{D} \mu_r \bar{M}_{0,pl} \ , \qquad (7)$$

where D is the determinant of the system of equations;
$A_{r,k+1}$ is the algebraic extensions of elements $M_{r,pl}$ of the determinant D;
$M_{o,\ pl}$ is the average value of the frame limit moment parameter;
μ_r is the component of the vector of coefficients of the frame limit moments ratios;
r is the number of the plasticity hinge.

The frame strength standard in the load parameter space is determined by

$$\hat{q}=\sum_{r=0}^{k+1}\frac{A_{r,k+1}}{D}\hat{M}_{r,pl}=\sum_{r=1}^{k+1}\frac{A_{r,k+1}}{D}\mu_r\hat{M}_{0,pl}\ . \qquad (8)$$

Expressions (7) and (8) determine the numerical characteristics of the random strength of the frame as a whole in the space of the load parameter, depending on the random characteristics of the random strength of individual elements when the frame is loaded once, when all loads and boundary moments are proportional to one parameter. The distribution of plastic moments in the frame $M_{r,pl}$ and the value of the limit plastic moments $M_{o,pl}$ are determined by the simplex method using the SIMPLEX program.

If the load parameter is a random value◦ $\tilde{Q}$, the frame strength reserve is equal to

$$\tilde{S}=\tilde{q}-\tilde{Q}>0\,, \qquad (9)$$

mathematical expectation of the strength reserve

$$\bar{S}=\bar{q}-\bar{Q}>0\,, \qquad (10)$$

the average square deviation of the strength reserve

$$\hat{S}=\sqrt{\hat{q}^2+\hat{Q}^2}\ . \qquad (11)$$

Dependence of the calculated load characteristics and limit plastic moments on the corresponding characteristics of random parameters:

$$Q=\bar{Q}+\gamma_Q\hat{Q}\ ; \qquad (12)$$

$$M_{0,pl}=\bar{M}_{0,pl}-\gamma_q\hat{M}_{0,pl}\ , \qquad (13)$$

where $M_{0,pl}$, $\bar{M}_{0,pl}$, $\gamma_q\hat{M}_{0,pl}$ are the calculation, mathematical expectation, standard of the limit moment value;

Q , $\bar{Q}$, $\hat{Q}$ are the calculation, mathematical expectation, standard of the limit moment value.

γ_Q , γ_q are the the number of standard deviations from the average for the design load and strength.

Conclusions

Determining the reliability of systems as a whole by one mechanism is numerically quite justified, but there may actually be mechanisms that have the probability of appearing close to the most likely mechanism. Therefore, the responsibility of the elements that make up these mechanisms is as significant as the elements that make up the true mechanism. In this regard, it is necessary to consider all the most likely mechanisms of destruction to better account for the load-bearing capacity of all structural elements in the design of new and reconstruction of existing buildings.

In general, it is possible to consider various cases of destruction of steel frames. And the main advantage of this method of finding the most real mechanism of destruction. Such methods are an important element of designing new modern structural forms and identifying weaknesses in existing structures.

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PLASTIC BEARING CAPACITY OF THE STEEL ELEMENT CROSS-SECTION BY INTERNAL FORCES COMBINATION AND RESTRAINT

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The article examines the features of determining the bearing capacity of the steel profile cross-section, rolled or composed from three sheets with arbitrary thickness, considering the development of plastic deformations in a complex combination of different power factors that may occur during spatial loss of stability. Based on a new approach to the perception character of internal forces, the goal is to investigate their ability to redistribute between separate linear elements, on which cross-section of the beam is broken. It is proposed to increase the savings of the material by detailing the calculation.

Keywords: bimoment, buckling, torsion, partial internal forces method, restraint.

НЕСУЧА ЗДАТНІСТЬ ПОПЕРЕЧНОГО ПЕРЕРІЗУ СТАЛЕВОГО ЕЛЕМЕНТА В ПЛАСТИЧНІЙ СТАДІЇ ПРИ СПОЛУЧЕННІ ВНУТРІШНІХ ЗУСИЛЬ І РОЗКРІПЛЕННІ

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Розглянуто особливості визначення несучої здатності поперечного перерізу сталевого профілю, прокатного чи складеного із трьох листів довільної товщини, з урахуванням розвитку пластичних деформацій при складному поєднанні різних силових факторів. На основі нового підходу до аналізу характеру сприйняття внутрішніх зусиль було поставлено за мету дослідити їх здатність перерозподілятися між окремими лінійними елементами, на які розбивається поперечний переріз балки. Запропоновано збільшити економію матеріалу шляхом деталізації розрахунку. До розрахунків, у процесі виконання яких передбачається поява і потреба врахування додаткових внутрішніх зусиль обмеженого кручення, можна віднести: 1 – звичайний та альтернативний (за теорією другого порядку) розрахунок балок на загальну стійкість; 2 – розрахунок елементів на сумісну дію поперечного згину (в одній або двох площинах) і кручення в результаті ексцентричності прикладення навантаження; 3 – розрахунок балок із викривленнями у площині найменшої жорсткості. У розрахунках за теорією другого порядку, що альтернативно враховують втрату стійкості плоскої форми згину елемента, вводиться еквівалентне початкове згинальне відхилення в площині найменшої жорсткості профілю. Тоді балка починає працювати вже як просторовий елемент, і поряд зі звичайними напруженнями згину в ній виникають додаткові напруження косого згину та кручення. Для збільшення точності розрахунків із застосуванням початкових недосконалостей і наближення їх до дійсних умов роботи конструкції внутрішні зусилля, в тому числі й від закручування, потрібно визначати за цією нелінійною теорією. Вона враховує геометричну нелінійність і являє собою по суті розрахунок за деформованою схемою, в якому рівняння рівноваги записуються для деформованого стану системи. Крім цього, необхідно врахувати жорсткість конструкцій, що дискретно чи континуально розкріплюють стиснутий пояс балки в більшості практичних випадків і зменшують деформації його зміщення.

Ключові слова: бімомент, втрата стійкості, кручення, метод часткових внутрішніх зусиль, розкріплення.

Introduction

The calculations, in the process when the appearance and need of restrained torsion additional internal forces consideration are foreseen, may include: 1 – conventional and alternative (according to the second order theory) calculation of beams for overall stability; 2 – the calculation of elements for the compatible action of the transverse bend (in one or two planes) and torsion as a result of load eccentric application; 3 – the calculation of beams with curvatures in the plane of the smallest rigidity. Equivalent initial bending deviation in the plane of the most minimal rigidity is introduced in the calculations by the second order theory, which alternatively considers the element bending flat shape stability loss. Then the beam starts to work as a spatial element, and along with the usual stresses of the bend there are additional stresses of oblique bending and torsion. In order to increase the accuracy of calculations by applying the initial imperfections and approaching them to the actual working conditions of the structure, internal forces, including from twisting, should be determined by this nonlinear theory. It considers geometric nonlinearity and represents essentially the calculation by a deformed scheme, in which the equilibrium equations are recorded for the system deformed state. In addition, it is necessary to consider the rigidity of structures that discretely or continually restrain the compressed beam flange in most practical cases and reduce the deformation of its displacement. It is proposed to determine sectorial geometric properties by rigid restraint not relative to the shear centre, but relative to the point of lateral restraint, which is located on the beam rotation axis. This point can move up or down depending on the degree of restraining, i.e. stiffness and placement of attached structures.

Review of research sources and publications

The works of representatives of the German classical technical school [1, 2, 3] can be attributed to the main sources, where there are the position of bearing capacity calculation of the rolled and composite beams in a plastic stage in a random combination of eight internal forces (longitudinal force, two transverse forces, two bending moments, two torque moments of primary and secondary torsion, flexural-torsional bimoment) by the partial internal forces method. Theoretical foundations of the theory of thin-walled rods are laid and presented in the books of the Soviet classical science school [4, 5]. The work of I-beams under the action of bending moment and bimoment is considered in the article [6]; however, in these studies, in contrast to the second order theory, there is a significant drawback, thus they do not consider the influence the rotation angle influence the value of bending moments in two planes. Experimental research of steel I-beams by a compatible bend and torsion operation was conducted under the direction of Tusnin A.R. [7]. The approbation of the second order theory in the works [8 – 10] demonstrates that insufficient restrained structure in an elastic stage is very sensitive to the load and curvature change, therefore, by determining the bearing capacity consideration of braces rigidity and plastic work of steel possibly can be effective, but a greater curvature is accepted (25 – 50 %).

Definition of unsolved aspects of the problem

The problem of rigid restrained beams calculation for building model compliance with real work of beams at complex resistance had not been solved before. As a research task, it has been decided to analyse the method of plastic bearing capacity determination and make in it adjustments that relate to the restraining and type of rolled profile.

Problem statement

Based on the acquired experience in the analysis of the distribution of internal forces in cross-section, considering the degree of restraining, it was decided to find and describe the differences in the rod work by the combination of forces from the transverse bend in two planes and restrained torsion.

Basic material and results

To calculate the limiting bearing capacity of the steel beam (non) symmetrical cross-section (Fig. 1) in the plastic stage in an arbitrary combination of internal forces it should use a separate case of the partial internal forces method (PIFM) [3]. In the case of cross-section, which can be decomposed into three rectangular elements with arbitrary thicknesses (two flanges and one web), the method is to put the origin not in the centre of gravity of the general cross-section, but in the centre of web gravity; further definition of geometric properties of general cross-section (position of the centre of gravity, angle of rotation of main axes, sectorial coordinates in case of longitudinal force, position of shear centre or by beam restraint of the centre of rotation) relative to the centre of web gravity; moving the internal forces into the coordinate system with the beginning at this point; decomposition of common internal forces into partial for each element in relation to its centre of gravity with the use of equilibrium conditions between them; checking of certain conditions of calculation for flanges bending from bending moment in a plane of smaller rigidity and bimoment; determination and comparison of residual bearing capacity of not included in the work part of the general cross-section for the bending moment with the active bending moment in the plane of greater rigidity with consideration of longitudinal force if it is necessary and reduction of bearing capacity from possible presence in place of inspection of the internal forces, leading to the appearance of tangential stresses, namely the transverse forces and to a lesser extent torque moments, but for the beams influence of the latter can be neglected without appreciable loss of accuracy.

By the example of a light slope roof purlin, it is considered in detail each of the stages of calculation, adapting it to the restrained by profiled flooring beam of the channel cross-section, hinge supported on the ends and loaded evenly distributed load (design sketch of purlin).

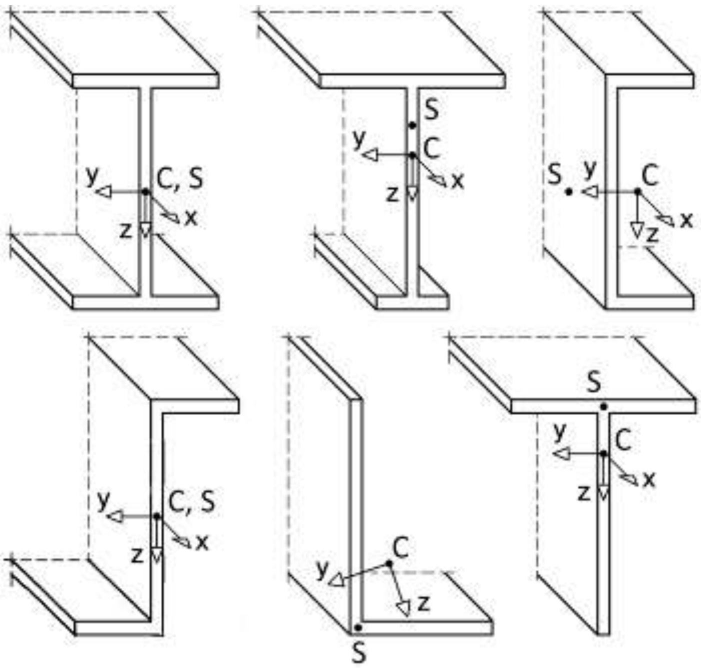

Figure 1 – Types of cross-sections that can be calculated with PIFM

It is known that with sufficient shear stiffness *S*, it is suggested that the compressed beam flange is completely fixed from the transverse displacement and rotation axis, which in this case is called restrained, passes on the beam top. Due to the high convergence of the calculation results and modelling it was discovered that for the channel with sufficient stiffness the axis of rotation is located above the shear centre at the level of the upper flange, where the slope component of load is applied that almost does not have an eccentricity in relation to that point.

Bending moments in the system with initial point in the web centre of the web in the absence of turning the main axis relative to central axis do not change ($M_{\bar{y}} = M_y$; $M_{\bar{z}} = M_z$). The formula for bimoment determining by the transition from the centre of rotation *D* for restrained cross-section (the shear centre *S* for unrestrained cross-section) to the centre of web gravity *O* is going to look (the equation in brackets is fair for enough restrained channel cross-section – Fig. 2)

$$M_{\bar{\omega}} = M_\omega + M_y\left(y_D - y_O\right) + M_z\left(z_D - z_O\right); \quad (1)$$

$$(M_{\bar{\omega}} = M_\omega + M_y e + M_z\left(-0{,}5h_s\right)). \quad (1^*)$$

Design bending moments M_y, M_z regarding *y*-axis and *z* can be determined according to the second order theory, since the consideration of lateral-torsional buckling in the partial internal forces method does not involve the use of the stability coefficient. It is necessary to consider the angle of the rod rotation and the equivalent load influence, which can be considered close due to the adoption of the bending moment diagram in the plane of smaller rigidity from the external and the stabilizing equivalent load in the form of sine wave (or parabola) with the maximum value, equal to the production of shear stiffness *S* and beam displacement, in the middle of the span.

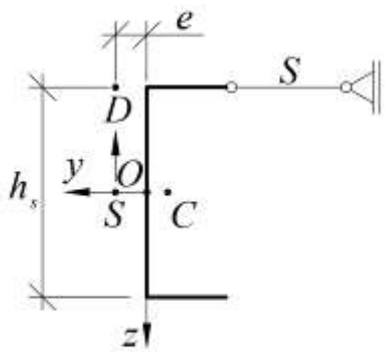

Figure 2 – Determination of the bimoment for the restrained channel beam

The actual work of the channel purlin in the inclined roofing, despite its prevalence, is often interpreted incorrectly due to the complexity of the description, which leads to analytical errors in the course of its calculation. It is especially proper for the definition of the bimoment, which depends on the function of the angle of rod rotation. The distribution of these deformations by element length is analyzed. It is important to note that in reality, the calculated bimoment depends not only on the load, its eccentricity, elastic flexural-torsional constant of cross-section and beam span, but also on the stiffness of attached to the beam structures.

Even when it is fastened the profiled flooring through the wave, its shear stiffness often reaches a sufficient value to take the restrained axis of beam rotation by the criterion of reduced necessary shear stiffness. Modelling by the finite elements analysis has shown that in this case the bending moment in the plane of the slightest rigidity decreases considerably (at the average angles of roof slope 10 – 30º more than 10 times) and its influence over the total stressed-strained state is practically levelled. The angle of rod rotation function is often described in the literature via a single-member expression using half of the sine wave, which usually leads to more or less accurate results. Because the application boundaries are rarely defined clearly, negative deviations can occur. Thus the presence of profiled flooring with considerable torsional stiffness and slope component of loading can cause tangible distortion of the rotation angle distribution curve and especially its derivatives by beam length. It, in turn, causes noticeable changes in the magnitude of the bimoment.

The deformed state of the beam by torsion is better described using an expression for the angle of rotation with seven parameters, chosen for convenience of differentiation and integration, three of which are zero. A smaller number of parameters leads to too high error in bimoment determining at the beam span middle in comparison with the FEA modelling in software module FE-LTB of complex Dlubal RSTAB 8.13 (Fig. 3), although it gives the satisfactory result in the determination of deformations and equivalent stabilizing load (the possibility of applying an expression with three parameters, one of which is zero, to describe the deformation of the beam in torsion with sufficiently high accuracy proved in the work [9]).

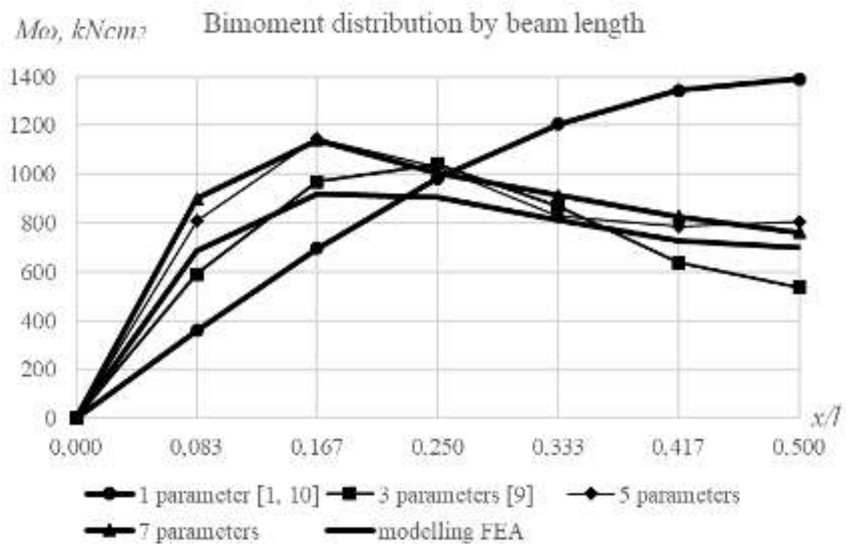

Figure 3 – Distribution of the bimoment by beam length

The partial bending moments in the bending of the upper and lower flanges with the action line at the centre of web gravity (point O in Fig. 4) are equal (the equations in brackets are valid for the rolled cross-sections)

$$M_{wao} = \frac{M_{\bar{z}}\bar{z}_u - M_{\bar{\omega}}}{\bar{z}_u - \bar{z}_o}; \quad (2)$$

$$(M_{wao} = \frac{M_{\bar{z}}}{2} - \frac{M_{\bar{\omega}}}{h_s}); \quad (2^*)$$

$$M_{wau} = \frac{-M_{\bar{z}}\bar{z}_o + M_{\bar{\omega}}}{\bar{z}_u - \bar{z}_o}; \quad (3)$$

$$(M_{wau} = \frac{M_{\bar{z}}}{2} + \frac{M_{\bar{\omega}}}{h_s}). \quad (3^*)$$

Formulae for determination of bearing capacity for longitudinal force and bending moment for upper and lower flanges with different sizes (in brackets – with the same sizes) in a plastic stage of work by the absence in the cross-section of tangential stresses are

$$N_{plo} = b_o t_o f_y; \; N_{plu} = b_u t_u f_y; \quad (4)$$

$$(N_{plo} = N_{plu} = b_f t_f f_y); \quad (4^*)$$

$$M_{plo} = 0,25 N_{plo} b_o; \; M_{plu} = 0,25 N_{plu} b_u; \quad (5)$$

$$(M_{plo} = 0,25 N_{plo} b_f = M_{plu} = 0,25 N_{plu} b_f). \quad (5^*)$$

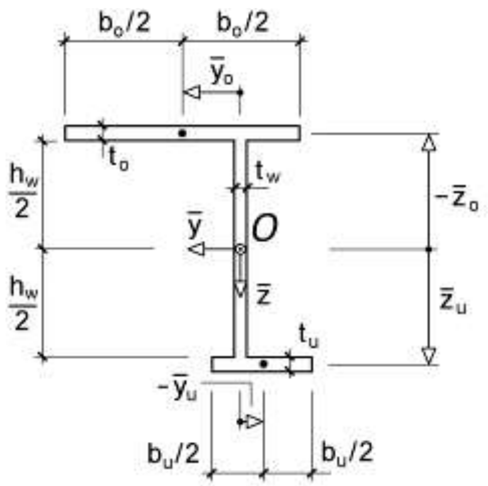

Figure 4 – Idealization of cross-section with arbitrary dimensions of elements

The equation (2) and (3) can be converted into the partial bending moments when bending the upper and lower beam flanges with the action line at the centre of the flange gravity

$$M_o = M_{wao} + N_o \bar{y}_o ; \quad (6)$$

$$M_u = M_{wau} + N_u \bar{y}_u , \quad (7)$$

where N_o, N_u – partial longitudinal forces in the flanges;

$\bar{y}_o$, $\bar{y}_u$ – coordinates of gravity centres for flanges in relation to the centre of web gravity.

These relationships are used in the equation of internal forces interaction for the rectangular cross-sections (for $i = o,u$):

$$\left(\frac{N_i}{N_{pli}}\right)^2 + \frac{|M_i|}{M_{pli}} \le 1 \Rightarrow \left(\frac{N_i}{N_{pli}}\right)^2 + \left|\frac{M_{wai}}{M_{pli}} + 2\delta_i \frac{N_i}{N_{pli}}\right| \le 1 , \quad (8)$$

where $\delta_i = 2\frac{\bar{y}_i}{b_i}$ – conditional relative coefficient.

Marginal partial longitudinal forces in the flanges are found as roots as a result of equation (8) solution:

$$N_{fi\min} = N_{pli}\left(-\delta_i - \sqrt{\delta_i^2 + 1 - \frac{M_{wai}}{M_{pli}}}\right) \text{ for } \frac{M_{wai}}{M_{pli}} \ge 2\delta_i ; \quad (9)$$

$$N_{fi\min} = N_{pli}\left(\delta_i - \sqrt{\delta_i^2 + 1 + \frac{M_{wai}}{M_{pli}}}\right) \text{ for } \frac{M_{wai}}{M_{pli}} < 2\delta_i ; \quad (10)$$

$$N_{fi\max} = N_{pli}\left(\delta_i + \sqrt{\delta_i^2 + 1 + \frac{M_{wai}}{M_{pli}}}\right) \quad \text{for} \quad -\frac{M_{wai}}{M_{pli}} \ge 2\delta_i . \quad (11)$$

$$N_{fi\max} = N_{pli}\left(-\delta_i + \sqrt{\delta_i^2 + 1 - \frac{M_{wai}}{M_{pli}}}\right) \quad \text{for} \quad -\frac{M_{wai}}{M_{pli}} < 2\delta_i . \quad (12)$$

From the sub root expressions contained in equations (9) – (12), the conditions for calculating of sufficient bearing capacity of flanges are received:

$$\frac{|M_{wao}|}{M_{plo}} \le 1 + \delta_o^2 ; \; \frac{|M_{wau}|}{M_{plu}} \le 1 + \delta_u^2 . \quad (13)$$

The active bending moment in the plane of greater rigidity at zero longitudinal force is compared with the minimum and maximum bearing capacity of not included in the work part of the general cross-section for the bending moment (for rolled cross-sections $\bar{z}_u = 0,5h_s$; $\bar{z}_o = -0,5h_s$)

$$M_{\min} \le M_{\bar{y}} \le M_{\max} ; \quad (14)$$

$$M_{\min} = N_{fu\min}\bar{z}_u + N_{fo\max}\bar{z}_o - \\ -\left(N_w^2 - \left(N_{fo\max} + N_{fu\min}\right)^2\right)\frac{h_w}{4N_w} ; \quad (15)$$

$$M_{\max} = N_{fu\max}\bar{z}_u + N_{fo\min}\bar{z}_o + \\ +\left(N_w^2 - \left(N_{fo\min} + N_{fu\max}\right)^2\right)\frac{h_w}{4N_w} , \quad (16)$$

where N_w – bearing capacity of a web for the longitudinal force; considering the radius rounding R in place of the flange connection with the web for the channel cross-section $N_w = h_w t_w f_y + 0,43r^2 f_y$; for I-rolled cross-section $N_w = h_w t_w f_y + 0,86r^2 f_y$;

h_w – height of the beam web.

Developed on the basis of partial internal forces, method technique of determination of the limiting bearing capacity of steel element cross-section with the compatible action of transverse bending and torsion for rolled and composite beams (I-beams or channels cross-section) differs from the original technique and from the principles described in the work [11]; the proposal consideration the full degree of profile restraining is by replacing the shear centre on the centre of rotation, as well as radius rounding at the flange connection with the web. The technique was implemented in the software environment MathCAD and tested on examples. Verification of the method under the same conditions of calculation (internal forces, geometric dimensions of cross-section and strength) in a file with macros QST-TSV-3Blech study program RUBSTAHL-Programme implemented by author of the partial internal forces method J. Frickel and C. Wolf showed almost identical similarity. It reflects the correct understanding and application of the partial internal forces method, which enables for the light slope roof purlins to rationally increase the coefficient of cross-section use on average by about 25 % compared to the elastic work due to available strength reserves.

Conclusions

Consideration the factors specified in the article describing the peculiarities of the steel element operation at the complex resistance enables to determine the values of internal forces that affect the overall stressed-strained state of the structure and regulate calculated ratio more precisely and accurately. Determination of existing reserves of the plastic work of steel can be carried out by calculating the bearing capacity of beams in a plastic stage by the partial internal forces method in the action of bending moments and bimoment and making more the maximal initial curvature. The verification of mathematical apparatus for determination of bearing capacity of imperfect elements that are affected by bending with torsion as a result of presence of geometric nonlinearity is conducted.

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THE OWN STRESSES INFLUENCE ON SCALE EFFECT IN CONCRETE

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The experimental technique and the experiment results on the influence of own stresses unevenly distributed over cross section on materials strength and on scale effect are presented. The reasons for the occurrence and distribution of such influence are analyzed. Based on the experiments results on fragile duralumin samples, the inevitability of influencing on scale effect in concrete during compression stresses, which are caused by unevenly shrinkage over cross section was proved. Possible reasons for the different (sometimes opposing) results of experimental studies of various authors on scale effect in concrete in compression are explained. The influence of own stresses unevenly distributed over cross section on scale effect in concrete during compression, depending on samples size and concrete age, is analyzed in detail.

Keywords: scale effect, own stresses, concrete strength, shrinkage, creep

ВПЛИВ ВЛАСНИХ НАПРУЖЕНЬ НА МАСШТАБНИЙ ЕФЕКТ В БЕТОНІ

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У роботах різних авторів результати експериментальних досліджень масштабного ефекту при випробуванні бетонів, розчинів та інших пористих матеріалів на стиск носять різний, іноді досить суперечливий, характер. Спроби пояснення отриманих результатів досліджень масштабного ефекту привели до необхідності врахування різноманітних факторів, до яких, безперечно, можна віднести вплив нерівномірно розподілених по поперечному перерізу власних напружень. Наведено методику проведення експерименту та результати дослідів по впливу власних напружень, нерівномірно розподілених по поперечному перерізу, на міцність матеріалів та на масштабний ефект. Проаналізовано причини виникнення протилежних за знаком полів власних напружень, нерівномірно розподілених по поперечному перерізу зразка, та їх можливий вплив на масштабний ефект при випробуванні зразка на стиск. Спираючись на результати дослідів з крихкими дюралюмінієвими зразками доведено неминучість впливу на масштабний ефект у бетоні при стиску напружень, викликаних нерівномірною по поперечному перерізу усадкою. На основі результатів експериментів інших дослідників по зміні міцності бетону повітряно-сухого зберігання у часі обґрунтовано зміни нерівномірності розподілу власних напружень по поперечному перерізу внаслідок нерівномірної усадки і повзучості бетону в поперечному перерізі. Поява різних за знаком полів нерівномірно розподілених по поперечному перерізу власних напружень у різні проміжки часу не тільки впливає на міцність бетонних зразків при стиску, а й проявляє в різній мірі масштабний ефект. Пояснені можливі причини різноманітних (іноді навіть протилежних) результатів експериментальних дослідів різних авторів щодо масштабного ефекту в бетоні при стиску. Детально проаналізовано вплив нерівномірно розподілених по поперечному перерізу власних напружень на масштабний ефект у бетоні при стиску в залежності від розмірів зразків і віку бетону.

Ключові слова: масштабний ефект, власні напруження, міцність бетону, усадка, повзучість

Introduction

The scale effect is substantiated by the statistical theory of strength: the larger sample size, the greater probability of a destructive defect [1-6]. At the same time, various results of experimental studies were obtained (especially for concretes, mortars and other porous materials): in some experiments in larger sizes samples obtained less strength; in other experiments in larger samples there was greater strength; in some cases, in samples of different sizes there was almost no difference in strength [7, 8, 9, 10, 11].

Such experimental results led to necessity to consider various factors influencing scale effect. One of such factors can be considered as influence of unevenly distributed own stresses over section.

Review of the research sources and publications

The first experiments on own stresses influence (unevenly distributed over cross section) were performed on samples of epoxy resin [1]. Cylindrical samples with a height of 60 mm and a diameter of 30 mm were made of epoxy resin in layers in three stages. Samples each series consisted of two groups, which differed from each other in the sign of their own stresses fields. Samples of the first group were made starting from the inner layers. First, the first inner layer of sample was filled, and then it hold time for 10 days. After attainment of sufficient strength inner layer, it was loaded with compressive force and the next second layer was filled around it. After 10 days, both layers (inner and middle) were loaded with a larger compressive force. Thereafter, last third outer layer was filled. Subsequently, first series samples had gained strength for 10 days, and then central compression was tested.

Second group samples were made in reverse order. The first was filled with outer layer of cylindrical sample. After 10 days, after reaching outer layer of sufficient strength attainment, it was loaded with compressive force and then filled middle layer. After 10 days, both layers were loaded with compressive force and last inner layer was filled.

After unloading, own stress field was created in samples of first group where sample internal part (core) was compressed and outer layers were stretched. The field of own stresses arose with opposite signs in second group samples: outer layers were compressed, and inner, on the contrary, stretched. It should be noted, strength of all layers in each sample was different due to different terms of their hardening during manufacture.

The disadvantages of the experiment, which are caused by manufacturing technology of samples and could insignificantly affect experiments results, include stepwise (layer-by-layer) own stresses field creation. In addition, epoxy resin is an aging material, and in different layers there were different hardening periods of epoxy resin.

Despite these disadvantages, experiments results confirmed the effect of their own unevenly distributed cross-sectional stresses on material strength. First group samples strength was larger than strength of the second group samples, despite the fact that outer layers of first group samples have less strength than second group samples due to different hardening periods [1].

Later, experiments were performed on effect of unevenly distributed cross-sectional stresses on second series samples strength, which completely eliminates first series disadvantage. Experiments to determine influence of own stresses on material strength were performed on cylindrical samples made of fragile (silicate) aluminum alloy with a diameter of 30 mm and a height of 60 mm. Samples of one group were made by ordinary technology: cast in steel molds and their cooling began with outer layers. Thus, after equalizing temperature with complete cooling in cross section of samples, outer layers were compressed and inner stretched.

Second batch samples were made in steel molds of same size as the previous ones. Steel tubes with a diameter of 6 mm were mounted on samples axis. The molds were located in cylindrical asbestos thermal insulators in the middle of which were electric heating coil mounted. Before a duralumin was filled, an electric heating coil had turned on and the steel mold had been heating to the duralumin melting point. During casting of duralumin and further sample cooling, water was supplied through the tube and, therefore, sample cooling had begun from middle layers. Thus, after complete sample cooling, its inner layers were compressed and outer stretched.

The experiments confirmed own stresses effect on the strength of duralumin: samples strength cooled from the central layers was on average larger than 20% of samples strength of ordinary (natural) cooling [12]. Considering that in large-sized samples own stresses (unevenly distributed over cross section) would have larger values than in smaller ones (and, as a consequence, the decrease in strength is more intense), it could be drawn conclusions about influence of own stresses on scale effect.

To confirm own stresses influence on the scale effect, experiments were performed on different sizes samples with opposite signs fields of own stresses. The experiments were performed on cylindrical samples with diameters of 30 and 50 mm and a height of 60 and 100 mm, respectively, made of fragile aluminum alloy.

After filling molds were cooled from middle, which provided own stresses field, which is opposite to stress field in samples of ordinary cooling. The experiments results confirmed logical justifications for own stresses influence on the scale effect: in samples with opposite natural distribution of own stresses over cross section, samples with a larger diameter showed larger strength [3].

Definition of unsolved aspects of the problem

Thus, the influence of unevenly distributed cross-sectional own stresses on scale effect in concrete has not been studied.

Problem statement

The goal of the research is to determine own stresses effect (unevenly distributed over cross section) on scale effect in concrete.

Basic material and results

The appearance and change of own stresses (unevenly distributed over cross section) in concrete are much more difficult than in metals.

The difficulty of unevenly distributed cross-sectional stresses caused by shrinkage on concrete strength studying effect is that intensity and shrinkage distribution in cross-section largely depends on concrete sample's storage conditions.

When storing samples in an air-dry environment, shrinkage begins and flows much more intensely on sample surface and gradually spreads to its inner layers. With greater reduction due to more intense concrete shrinkage in sample's outer layers the on its surface there are shrinkage tensile stresses, which in turn compress concrete in sample's inner layers (core) and create conditions for cracks on the sample surface.

In initial period of shrinkage when sample is loaded with compressive force, sample outer layers are not compressed until the external load reaches a value that can compensate for their own tensile stresses.

The sample internal part, compressed by its own stresses until an external force is applied, has higher total stresses when loaded with compressive force than outer layers, since self-stresses in inner layers consist of stresses from the action of the compressive external load with same sign. Therefore, strength of whole sample should determine its more intense inner part. However, inner part, although overloaded, is located in a clip made by underloaded outer layers.

As a result, in general, such sample should withstand more external compressive load than the sample without own stresses field. Well-known and confirmed by numerous experiments, there is the effect of increase in strength during concrete drying, associated with a more intense manifestation of above own stress distribution over sample cross section.

In the stressed state of concrete sample, in the above, there is a concrete creep in tension outer layers, which leads to their elongation and in inner layers of creep from compression, which reduces them. In parallel, the shrinkage gradually moves to sample's inner layers and eventually reaches the sample central part. During this period, shrinkage intensity in outer layers decreased compared to initial, and the concrete surface of sample increased in size due to creep from tensile stresses.

Sample inner layers (core), continuing to shrink from shrinkage in size, compress the outer layers, and themselves gradually move to a stretched state. Characteristically, this process is enhanced by the resulting reduction of sample's inner part due to creep during compression in shrinkage initial period.

During this period, when loading such a sample with compressive force, outer layers are overloaded, and total load in them consists of its own stresses and stresses caused by action of compressive external load. The outer layers have no clamps and, therefore, naturally, sample collapses at a lower load than sample without its own stresses.

Based on the changes in distribution of own stresses in the cross section over time due to the shrinkage and creep of concrete, it is possible to draw a conclusion about the possible decrease in concrete strength over time. This conclusion is confirmed by the experimental results by S.A. Mironov, shown on the Figure 1. A temporary decrease in strength of concrete is observed for concretes of both natural hardening and steaming [13].

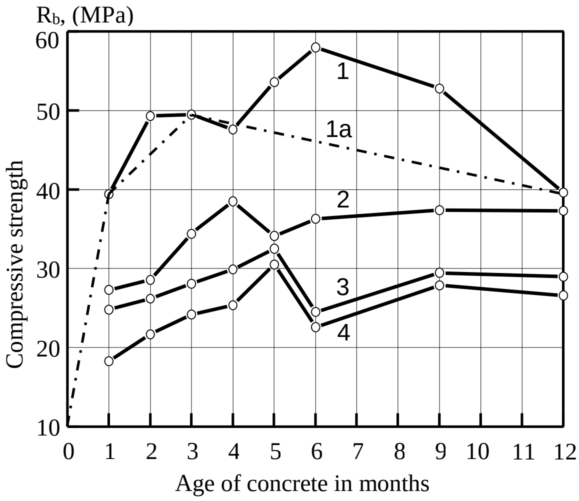

Figure 1 – Changes over time concrete strength in the experiments of S.A. Mironov

After some time, concrete shrinkage attenuates throughout cross section, after which relaxation of its own tensile stresses in the sample middle (core) and compressive stresses in its outer layers begins to appear. Thus, the difference between stresses values in outer and inner layers is reduced and, as a consequence, negative effect of unevenly distributed own stresses over cross section on compressive concrete strength is reduced. During this period there is increase in concrete strength.

Diagrams of the change in cubic compression strength of some samples concrete are shown in Figure 1. Repeated decrease in concrete strength can also be additionally caused by relaxation of structural own stresses [14]. Such a decrease may not be at all or it may be insignificant depending on tested samples concrete properties: gravel size, water-cement ratio, hardening conditions, storage conditions, etc.

Summarizing the analysis stresses shrinkage effect unevenly distributed over cross section on concrete compressive strength, it can be concluded that the intrinsic stresses first increase the strength of concrete, then there may be temporary decrease in strength, followed by increase in compressive strength. After the attenuation of its own stresses and the gradual attenuation of deformations caused by creep, there may be a gradual decrease in the concrete strength, over time. These conclusions are fully confirmed by the experiments of S.A. Mironov and others.

The diagrams could have a different form depending on the timing of determining concrete strength during experiments. When constructing diagrams on four points (and not on eight points, as in the experiments of S.A. Myronov) according to the results obtained in the experiments of V.I. Sytnyk and Yu.A. Ivanov [15] repeated increase in strength concrete is not observed in

Figure 1 (1a). Such conclusions are confirmed experimentally. In the experiments of V.I. Sytnyk and Yu.A. Ivanov, in all considered concretes, mortars and cement stone, the strength gradually decreases over time after reaching the maximum (Fig. 2).

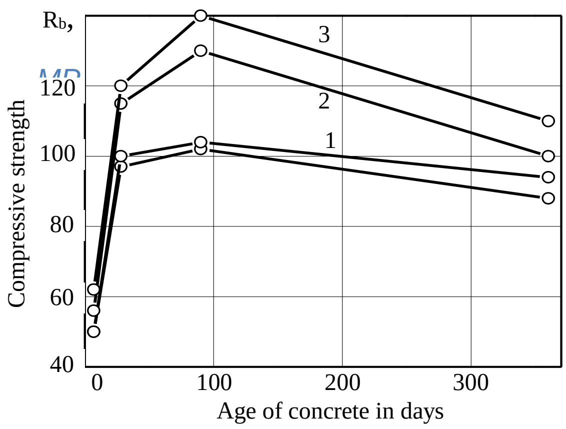

Figure 2 – Changing concrete strength (1), mortar (2) and cement stone (3) over time in the experiments V.I. Sytnyk and Yu.A. Ivanov [15]

In almost all experiments, when the concrete reaches maximum strength, a temporary slight decrease in strength is observed. In the future, strength increases slightly, then slowly and gradually decreases over time [13].

The opposite effect will occur at water saturation of concrete samples, because process begins with moisturizing outer layers. Swelling of concrete causes, respectively, appearance of compressive stresses in sample's outer layers and tensile stresses in inner layers. During this period, when compressing such a sample, outer layers that are not in holder will be overloaded and therefore sample will begin to collapse from outer layers. As a result, strength of the water-saturated concrete sample will be less than strength of same sample without its own stresses caused by swelling.

When storing samples in an air-dry environment, the gradual decrease in concrete compressive strength after reaching the maximum value; in smaller sizes samples begin earlier than in larger sizes samples due to the fact that shrinkage in samples of smaller sizes is more quickly aligned over the cross section.

The graph of the change in concrete strength over time in different sizes samples is shown in Figure 3.

Figure 3 clearly shows different variants of the strength ratio in the different sizes samples depending on concrete age in which the tests were performed.

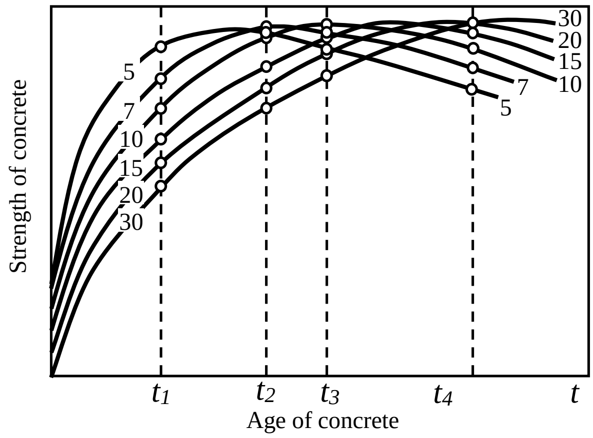

Figure 3 – Characteristic graphs of changes in concrete strength over time

The numbers that indicate curves in graphs correspond to edges size of the tested cubes in centimeters. The age numbering of concrete in which test sample were tested corresponds to the numbering given in Table 1 and Figure 3.

Table 1 – Test results of different sizes cubes for strength in MPa

№	Authors of researchers	Edges of the cube size (cm)					
		5	7	10	15	20	30
1	Skramtaev B.	13	12	11,5	–	10,0	–
2	Kvirikadze O. (1 series)	24,6	25,2	24,4	23,3	22,8	–
3	Kvirikadze O. (2 series)	16,8	17,9	18,1	17,5	16,9	–
4	Lermit R.	–	–	19,8	21,2	21,2	19,3
5	Tsiskreli G.	–	–	–	32,0	32,5	30,0

The own stresses influence on scale effect under compression is confirmed by numerous experiments. Some results of experimental studies are shown in table 1 and for clarity are duplicated in Figure 4.

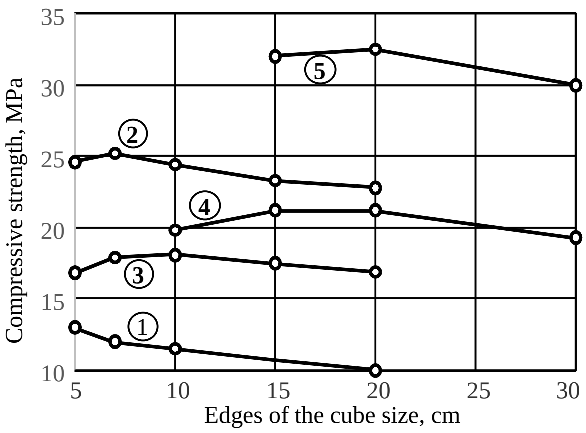

Figure 4 – Experimental data on the influence of scale effect on the concrete compressive strength

At the age of concrete t_1 (Fig. 3) a large-scale effect is maintained: with increasing sample size decreases concrete strength. When testing concrete at the age of t_2, t_3 and t_4 there are strength ratio different variants in samples of different sizes: in some cases, with increasing sample size concrete increases strength, in some there is no effect of sample size on concrete strength). In the age of concrete exceeding the term t_4, the opposite scale effect is observed – with increasing sample size, concrete compressive strength increases.

Some authors have obtained the results of experiments in which concrete strength in cubes with an edge of 5 cm or 7 cm is less than in larger sizes cubes. This phenomenon was most often explained by insufficient compaction of concrete in manufacture of samples or that small samples dry quickly and there is not enough moisture to hydrate cement. At the same time, there are the results of experiments in which strength in cubes with an edge of 10 cm and 15 cm (in the table, the experiments of R. Lermit and G.D. Tsiskreli) is less than in larger samples. These deviations can be explained by own stresses influence on concrete strength (scale effect).

It should also be noted that tabular results do not cover all the ratios of concrete strength in samples of different sizes. For example, in the experiments of I.S. Karol’ and others [16] in four series (430 cubes with edge sizes of 20, 15, 10 and 7 cm) of different manufacturing conditions (storage) obtained the lowest strength in samples with edge size of 10 cm. Graphs of concrete strength over time, while maintaining the general shape, can also change the ratio depending on concrete preparation technology, storage conditions, and more.

The various factors influence on the experiments results of scale effect in concrete was analyzed in detail by O.P. Kvirikadze [6]. The analysis was conducted on our own experiments, as well as on different researchers’ experiments results. The author gives recommendations on experiments performance on determinate of scale effect of concrete at compression. The influence of own structural stresses (caused by shrinkage or creep) on concrete strength largely coincides with influence of unevenly distributed cross-sectional stresses [14]. It is almost impossible to separate (determine) the influence degree of both stresses.

Conclusions

The influence of unevenly distributed over cross sectional compression stresses on scale effect in concrete is analyzed and substantiates and explains the variety of experimental studies results of concrete, mortars and other porous materials by different authors.

The influence of the scale effect in concrete tensile strength has not been studied enough. The experiments were most often made by bending or splitting. Samples were most often made in a horizontal position, which also negatively affects test results.

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ENGINEERING METHOD FOR CALCULATING STEEL-REINFORCED CONCRETE ELEMENTS WITH FLEXIBILITY

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In the article presents an engineering method for calculating compressed flexible reinforced concrete elements with sheet reinforcement over a steel cross section. The results of the calculation are compared with the experimental data. Calculation of load-bearing capacity of reinforced concrete flexible elements with sheet reinforcement is based on the method of boundary states. The work of specimens under load and the nature of the load-bearing capacity depending on the height and eccentricity of the effort were investigated. The proposed method of calculating compressed elements with sheet reinforcement on a steel-cross-section allows to take into account their flexibility in both axial and out-of-center application of load.

Keywords: steel reinforced concrete, bearing capacity, sheet reinforcement, eccentricity, flexibility

ІНЖЕНЕРНИЙ МЕТОД РОЗРАХУНКУ СТАЛЕЗАЛІЗОБЕТОННИХ СТИСНУТИХ ЕЛЕМЕНТІВ З УРАХУВАННЯМ ГНУЧКОСТІ

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Наведено інженерний метод розрахунку стиснутих гнучких сталезалізобетонних елементів з листовим армуванням за зведеним до сталі перерізом, а також порівняння результатів розрахунку з експериментальними даними. Розрахунок несучої здатності сталезалізобетонних гнучких елементів з листовим армуванням ґрунтується на методі граничних станів. Як граничний стан за міцністю прийнято зусилля, в результаті дії яких у листовому армуванні в найбільш напруженому волокні поздовжні деформації досягають значення, що відповідають напруженням плинності сталі. У розрахунок за методом зведених перерізів покладено передумови, що бетон, метал листового та стержньового армування вважаємо ізотропними пружно-пластичними матеріалами, а поперечний переріз сталезалізобетонного елемента з листовим армуванням залишається плоским аж до моменту руйнування. При розрахунку центрально стиснутих сталезалізобетонних елементів із листовим армуванням несуча здатність визначено шляхом зведення комплексного поперечного перерізу до односкладового – сталевого. Це дає змогу використовувати у розрахунку табличні коефіцієнти поздовжнього згину φ. Несуча здатність позацентрово стиснутого сталезалізобетонного елемента з листовим армуванням також визначається шляхом зведення комплексного поперечного перерізу до односкладового – сталевого. При цьому виникає можливість використовувати при розрахунках табличні значення коефіцієнта поздовжнього згину φ_e, які містяться у ДБН. Експериментально було досліджено роботу зразків під навантаженням та характер втрати несучої здатності залежно від висоти й ексцентриситету прикладених зусиль. Запропонована методика розрахунку стиснутих елементів з листовим армуванням за зведеним до сталі перерізом дозволяє враховувати їх гнучкість як при осьовому, так і позацентровому прикладанні навантаження.

Ключові слова: сталезалізобетон, несуча здатність, листове армування, ексцентриситет, гнучкість

Introduction

When calculating the elements working on compression, the problem is to consider their flexibility. This issue remains unresolved for steel reinforced concrete elements reinforced with sheets.

Review of the research sources and publications The engineering method for calculating steel-reinforced concrete elements with sheet reinforcement for a steel-cut cross-section was presented in DBN [1]. Prior to this work, compressed elements with reinforcing sheets of 4 mm thickness were investigated and height up to 1 m [3] and elevation to 2.4 m [4].

Definition of unsolved aspects of the problem

This paper addresses the issue of calculating compressed flexible elements reinforced with sheets by cutting the cross section to steel.

Problem statement

The purpose of the article is to compare the results of theoretical studies with the engineering method of calculation of central and noncentrally compressed flexible reinforced concrete elements with sheet reinforcement in a steel-cross section.

Basic material and results

The calculation of the bearing capacity of steel reinforced concrete flexible elements with sheet reinforcement is based on the method of boundary states. As the ultimate state of strength it is taken the effort that as a result of which the sheet reinforcement in the most intense fiber longitudinal deformations reach values that correspond to the stress of steel flux.

The following assumptions are based on the calculation of the consolidated sections:

– concrete, metal sheet reinforcement and rod reinforcement are considered as isotropic elastic-plastic materials;

– we believe that the cross-section of steel-reinforced concrete element with sheet reinforcement remains flat until the moment of destruction;

– geometrical and physical-mechanical characteristics of steel and concrete we accept steel-length steel-reinforced concrete element;

– during the entire work of the integrated section, concrete, sheet reinforcement and core reinforcement deform jointly;

– in the limiting state with centrifugal compression, the ambiguous string of stresses for concrete, sheet reinforcement and rod reinforcement in compressed and stretched zones has the shape of a rectangle;

– concrete of a stretched zone is not considered in the section of steel-reinforced concrete element with sheet reinforcement;

– rod reinforcement, if it is in conditions of one-piece compression or stretching.

When calculating the centrally compressed steel-reinforced concrete elements with sheet reinforcement, the bearing capacity can be determined by the construction of a complex cross-section to a single-component - steel. In addition, it enables to use of tablewise coefficients of longitudinal bending in the calculation φ. In this formulation the question of the bearing capacity of steel-reinforced concrete element with sheet reinforcement is determined by the formula

$$N = \varphi R_s A_{zved}, \tag{1}$$

where A_{zved} – cross-sectional area, reduced to sheet metal reinforcement area (Fig. 1).

In this case, the aggregate area is calculated by the formula

$$A_{zved} = A_b \frac{R_b}{R_s} + A_s + A_{s1} \frac{R_{s1}}{R_s}, \tag{2}$$

Coefficient φ is determined by the addition of K to DBN [1] depending on flexibility λ and the strength of sheet steel reinforcement R_s.

Flexibility of the element is calculated by the formula

$$\lambda = \frac{L_0 \mu}{i_{zved}}, \tag{3}$$

where L_0 free length of steel reinforced concrete condensed element with sheet reinforcement;

μ – coefficient of estimated length;

$i_{звед}$ – the radius of inertia of the cross-section of steel-reinforced concrete element with sheet reinforcement, reduced to steel sheet reinforcement, determined by the formula

$$i_{zved} = \sqrt{\frac{I_{zved}}{A_{zved}}}. \tag{4}$$

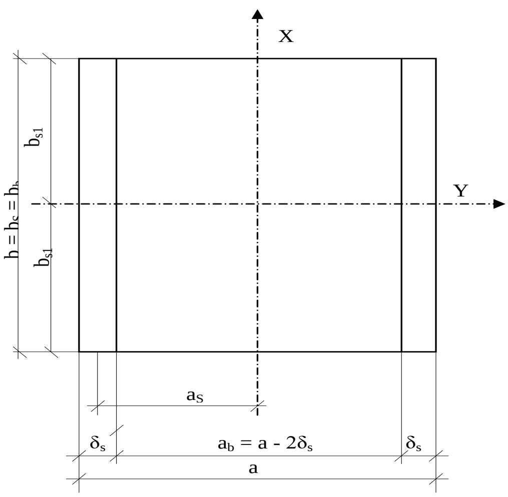

Figure 1 – Cross section of a steel concrete element with sheet reinforcement in the calculation for a steel-cut cross-section

With A_{zved} is calculated by the formula (2), and the consolidated moment of inertia is calculated by the formula

$$I_{zved} = I_b \frac{R_b}{R_s} + I_s + I_{s1} \frac{R_{s1}}{R_s}. \tag{5}$$

Thus, in accordance with the proposed method, it is possible to determine the bearing capacity of centrally compressed steel reinforced concrete elements with sheet reinforcement using the instructions given in the DBN [1] tabular coefficients of longitudinal bending.

By analyzing the stress-strain state and the nature of the destruction of non-centered compressed steel reinforced concrete elements with sheet reinforcement, it can be concluded that they are highly dependent on eccentricity e_0. In this case, it is distinguished between two cases of the boundary state of the bearing capacity with centrifugal compression (e_0>0):

– the load is applied with eccentricity, but within the core of the cross-section (case of small eccentricities). In this case, the cross-section is completely compressed, but the destruction occurs in the tensest zone due to the bending of sheet reinforcement. All this leads to the non-use of the strength of the less compressed part of the cross-section;

– the load is applied with large values of eccentricity, that is, beyond the core of the cross-section (case of large eccentricities). In this case, the longitudinal axis is significantly distorted and there is a stretched zone in the transverse section.

The bearing capacity of a noncentrally compressed steel reinforced concrete element with sheet reinforcement can be determined by the construction of a complex cross-section to a single-component - steel. In this case, it is possible to use in the calculations table values of the coefficient of longitudinal bending φ_e, which are contained in DBN [1]. In this case, the calculation formula for determining the bearing capacity of the centrifugally compressed steel reinforced concrete element with sheet reinforcement has the form

$$N = \frac{R_S}{\dfrac{1}{A_{zved}} + \dfrac{e_0 y_{max}}{I_{x(y),zved}}}, \qquad (6)$$

where A_{zved} – the area of the cross-section, reduced to the area of metal sheet reinforcement, is determined by the formula (2);

y_{max} – the distance from the center of gravity of the consolidated cross-section to the most stretched fiber;

$I_{x(y),zved}$ – the combined moment of inertia of the cross-section relative to the central axes, is determined by the formula

$$I_{x(y),zved} = I_{x(y),b}\frac{R_b}{R_S} + I_{x(y),s1}\frac{R_{s1}}{R_S}, \qquad (7)$$

where $I_{x(y),b}$, $I_{x(y),s}$, $I_{x(y),s1}$ – moments of inertia of concrete, metal sheet reinforcement and metal rod reinforcement relative to center of gravity axes (X_0 and Y_0) convex cross-section (Fig. 1).

The corresponding moments of inertia of the cross-sectional components areas are determined (Fig. 1) according to the formula

$$I_{X,b} = I_{X_b} = \frac{b a_b^3}{12}; \qquad (8)$$

$$I_{Y,b} = I_{Y_b} = \frac{a_b b^3}{12}; \qquad (9)$$

$$I_{X,s} = 2\left[I_{Xs} + a_s^2 A_s^1\right] = 2\left[\frac{b\delta_s^3}{12} + a_s^2 A_s^1\right]; \qquad (10)$$

$$I_{Y,s} = 2I_{Y_S} = 2\frac{\delta_s b^3}{12}; \qquad (11)$$

$$I_{X,s1} = 4\left[I_{X_{s1}} + a_{s1}^2 A_{s1}^1\right] = 4\left[\frac{\pi d_{s1}^3}{64} + a_{s1}^2 A_{s1}^1\right]; \qquad (12)$$

$$I_{Y,s1} = 4\left[I_{Y_{s1}} + b_{s1}^2 A_{s1}^1\right] = 4\left[\frac{\pi d_{s1}^3}{64} + b_{s1}^2 A_{s1}^1\right], \qquad (13)$$

where *(X_b, Y_b), (X_s, Y_s), (X_{s1}, Y_{s1})*, – own concrete axes, sheet reinforcement and rod reinforcement in the cross section of steel reinforced concrete element with sheet reinforcement, respectively;

A_s^1, A_{s1}^1 – cross-sectional area of one element of sheet reinforcement and one element of rod reinforcement in cross-section of steel-reinforced concrete element with sheet reinforcement.

The calculation for the stability of non-centrifugally compressed elements of the constant in length of the cross-section in the plane of action of the bending moment, which coincides with the plane of symmetry, should be performed according to the formula

$$\frac{N}{\varphi_e A_{zved} R_y \gamma_c} \le 1. \qquad (14)$$

In the formula (14) stability factor for off-center compression φ_e should be determined by table К.3 application К DBN [1] depending on the values of conditional flexibility $\bar{\lambda}$ and the reduced relative eccentricity m_{ef}, which is calculated by the formula

$$m_{ef} = \eta\, m, \qquad (15)$$

where η – coefficient of section shape influence of should be determined by table К.2 application К DBN [1];

$m = eA/W_c$ – relative eccentricity;

$e = M / N$ – eccentricity, when calculating the estimated values of internal effort M and N should be taken in accordance with the requirements DBN [1];

W_c – moment of resistance of the cross section, calculated for the most compressed fiber.

The value of the coefficient of longitudinal bending φ_e is set according to DBN [1] depending on the design resistance of steel sheet reinforcement R_y and flexibility $\bar{\lambda}$. The aggregate flexibility is determined by the formula

$$\bar{\lambda} = \lambda\sqrt{\frac{R_y}{E}}. \qquad (16)$$

Value of coefficients φ is shown in table К1 application К DBN [1].

The bends of the average height of the cross-section of flexible steel reinforced concrete structures with sheet reinforcement can be determined on the basis of the condition

$$M = N\left(e_0 + f\right) = \frac{Ne_0}{1 - \frac{N}{N_{gran}}}, \qquad (17)$$

where M – moment acting in cross-section;
N – longitudinal effort;
N_{gran} – conditional limiting force perceived by the design

$$N_{gran} = \frac{\pi^2 E_S I_{zved}}{L^2}. \qquad (18)$$

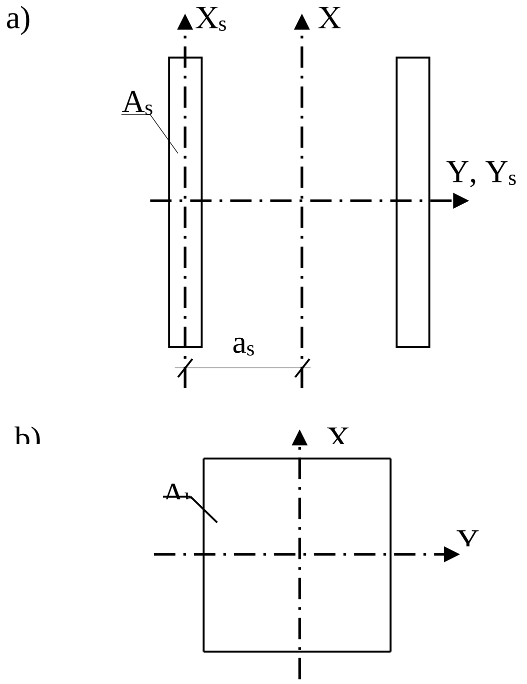

Figure 2 – Scheme of cross-section of steel-reinforced concrete element with sheet reinforcement, with defined moments of inertia of component parts:
a – sheet reinforcement; b – concrete

Substituting (18) in (17) and decided the equation relative f, it is obtained a formula for determining the complete bending of steel reinforced concrete structures with sheet reinforcement in compression:

$$f_{x(y)} = \frac{G\pi^2 e_0}{G\pi^2 - NL^2} - e_0, \qquad (19)$$

where G – rigidity of steel reinforced concrete structure with sheet reinforcement:

$$G = E_S I_{x(y),zved}, \qquad (20)$$

where E_s – modulus of elasticity of sheet metal reinforcement.

In the table 1 the research experimental data of steel reinforced concrete elements with sheet reinforcement bearing capacity are given [4]. For the study samples were made in different heights and different applied eccentricity. The process of manufacturing steel reinforced concrete elements consisted of two parts: the manufacture of steel frames and concrete concreting.

Table 1 – Results of comparison of calculations of bearing capacity

Sample series	Lengt hL, mm	Eccentricity e_0, mm	Bearing capacity N1, kN	Theoretical value, kN	Deviation, %
SB-PD-10-1	1000	0	238	219	8.0
SB-PD -10-2	1000	25	154	146	5.2
SB-PD -10-3	1000	50	105	101	3.8
SB-PD -17-1	1700	0	234	223	4.7
SB-PD -17-2	1700	25	144	132	8.3
SB-PD -17-3	1700	50	93	86	7.5
SB-PD -24-1	2400	0	211	198	6.2
SB-PD -24-2	2400	25	148	136	8.1
SB-PD -24-3	2400	50	102	93	8.8

For the production of experimental samples, section of which is given on Fig. 3, steel sheet was used t = 4 mm, cross-section valves of a class A-I ∅6 mm. The sample height was at 1000, 1700, 2400 mm, section 100×100 mm. To find out the effectiveness of the work of steel-concrete elements, a sample of steel without concrete height was tested 1000 mm. Standard concrete concrete cubes were tested for determination of physical and mechanical properties of concrete aggregate 150×150×150 mm and prisms 150×150×600 mm, made from the same concrete as the prototype.

When tested, experimental specimens had a different bearing capacity, which depended on the constructive solution of the load application eccentricity (Fig. 4).

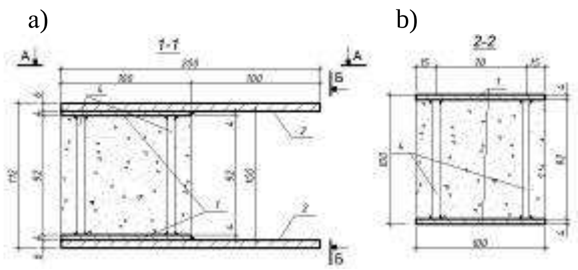

Figure 3 – Breakdown of experimental samples:
a – in the supporting part;
b – in the middle of the sample

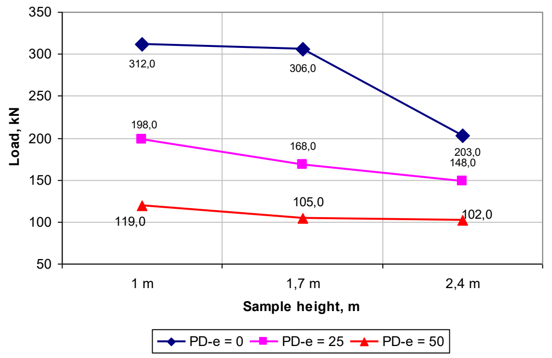

Figure 4 – Dependence of tested steel-reinforced concrete samples with sheet reinforcement carrying capacity on element height

The work of the samples under load and the nature of the bearing capacity loss depending on the height and eccentricity of the applied efforts have been experimentally investigated.

In the course of experimental studies, longitudinal and transverse deformations were measured at different distances from the ends of the element, as well as the displacement of the average height at the cross-section. All this enabled to get a complete picture of the work under the load of steel concrete racks with sheet reinforcement both at central and centripetal compression.

In the table 1 the comparison results of calculated values of load bearing capacity of compressed flexible steel reinforced concrete elements with results of experimental research are given.

Conclusions

The proposed method of calculation of compressed elements with sheet reinforcement in a steel-cut cross-section enables to consider their flexibility both in axial and centrifugal loading. The theoretical values of the bearing capacity are satisfactory with the results of the experiment, the difference is no more than 8.8 %.

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TRAY RESEARCH OF THE STRAIN STATE OF SOIL BASES REINFORCED BY SOIL-CEMENT ELEMENTS UNDER THE STRIP STAMP

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Experimental research of weak bases reinforced by vertical soil-cement elements are discribed in this article. Tray research of weak clay bases and bases with percent of reinforcement from 2,1% to 7,1% under strip stamp were conduct. As a result of the experiment settlements for each of soil base models were obtained. Characteristic curves of the influence of the reinforcement percentage on the bearing capacity, deformability are given, character of settlements of the investigated soil mass in time at step-increasing loading are shown. It was found that depending on the percentage of reinforcement the critical pressures increased linearly and the values of settlement at critical pressures were determined increased with the base reinforcement percentage. Empirical dependencies were obtained to determine the critical pressures depending on the percentage of reinforcement for the investigated soil conditions. In conclusion, the effectiveness of reinforcing the base of the strip foundations by vertical soil-cement elements was proved.

Keywords: loam, rigid strip stamp, settlement, soil base, tray research, vertical soil-cement element.

ЛОТКОВІ ДОСЛІДЖЕННЯ ДЕФОРМОВАНОГО СТАНУ ОСНОВ, АРМОВАНИХ ҐРУНТОЦЕМЕНТНИМИ ЕЛЕМЕНТАМИ, ПІД СТРІЧКОВИМ ШТАМПОМ

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Наведені останні дослідження вітчизняних вчених щодо впливу різних факторів на міцність ґрунтоцементу. Виявлено, що відсутні експериментальні дані про деформований стан глинистих ґрунтів під навантаженням від стрічкового штампу. Проведено лоткові випробування жорстким стрічковим штампом слабких глинистих основ без елементів армування та аналогічних основ, посилених ґрунтоцементними елементами з різним відсотком армування. Описано методику виконання досліджень, характеристика системи навантаження, фізико-механічні властивості елементів армування та ґрунтової основи. В результаті досліду отримано дані осідань по кожній із моделей ґрунтової основи та побудовано відповідні графіки осідань. Наведено залежності впливу відсотка армування на несучу здатність та деформативність основи. Проаналізовано залежності осідань дослідного ґрунтового масиву в часі при ступенево-зростаючому навантаженні. Визначено критичні тиски при різних відсотках армування основи штампу для фізико-механічних характеристик ґрунту, що розглядається. За результатами побудови графіків встановлено, що критичні тиски на основу залежно від відсотку її армування підвищуються за лінійною залежністю, а величина осідання, при якій визначалися перший та другий критичні тиски, зі збільшенням відсотку армування основи також зростає. Отримано емпіричні залежності для визначення критичних тисків на основу залежно від відсотку її армування з використанням класичних розв'язків Пузиревського та Прандтля для дослідних ґрунтових умов. Обчислені статистичні коефіцієнти варіації та кореляції для наведених залежностей. На підставі проведеного дослідження доведено ефективність посилення основи стрічкових фундаментів вертикальними ґрунтоцементними елементами.

Ключові слова: вертикальний ґрунтоцементний елемент, ґрунтова основа, жорсткий стрічковий штамп, лоткові дослідження, осідання, суглинок.

Introduction

In recent decades the method of reinforcing weak bases with vertical soil cement elements (SCE), which can be fabricated by using of jet-grouting or drilling-mixing technology, has become popular in the world. The effect of such reinforcement of the bases is that the mechanical characteristics of the soil (modulus of deformation, specific cohesion, etc.) are improved. This increases its load-bearing capacity and reduces its deformability. Reliability, cost-effectiveness, low energy consumption and material consumption are the main factors that substantiate the relevance of researches on the use of soil cement in the arrangement of bases and foundations [1 – 6].

However, it should be noted that the question of the effectiveness of reinforcing the bases of the strip foundations by SCE and the dependence of the settlement of the building foundations reinforced in this way have not yet been studied enough.

Review of the research sources and publications

From the practice of geotechnics experimentally and with the use of numerical modeling by the finite element method the effectiveness of the improvement of weak clay soil bases with the help of vertical SCE reinforcement has been proved [1 – 10].

It has been determined, in particular, from a number of researches conducted by the geotechnical school of Poltava National Technical Yuri Kondratyuk University [5 – 7, 9, 10] that over time the strength of the SCE and, consequently, the strength of the reinforced bases increase. Dependences of deformation modules of soil-cement mixture on the percentage of cement content and water-cement ratio, soil density, etc. were obtained [5, 6].

Based on this, tray stamp-loading tests were conducted with a round rigid stamp and the dependences on the characteristics of the soil base (density of dry soil, depth of arrangement of reinforcing elements, percent of reinforcement, etc.) were determined [7].

Definition of unsolved aspects of the problem

At the same time, there are no tray tests of bases, which are strengthened with vertical SCE of reinforcement under the rigid strip stamp with the identification of patterns of improving the strengthened by this way weak soil base at steady-growing load.

Problem statement

Therefore, as the purpose of this work, it was presented the research of the soil bases strain state with variable parameters of the reinforcement in the tray and the analysis of the obtained results of the soil base deformation, which is reinforced with vertical soil cement elements.

Basic material and results

To achieve exploration objective the following tasks were solved:

– to investigate the strain state of the unreinforced clay base under the rigid strip stamp;
– to investigate the development of deformations of the clay base reinforced by vertical SCE;
– to compare the strain state of the reinforced and unreinforced bases.

Tray tests were conducted in a metal tray of 3 mm thick steel sheet with stiffeners made of steel corners (Fig. 1). The front wall is made of transparent organic glass 40 mm thick. The internal dimensions of the tray are 580 × 530 × 560 mm. The test was performed with a rectangular steel stamp measuring 420 × 35 mm, weighing 67 N. A total of 4 tray experiments were performed. Clay paste was used as the basis for all tray researches.

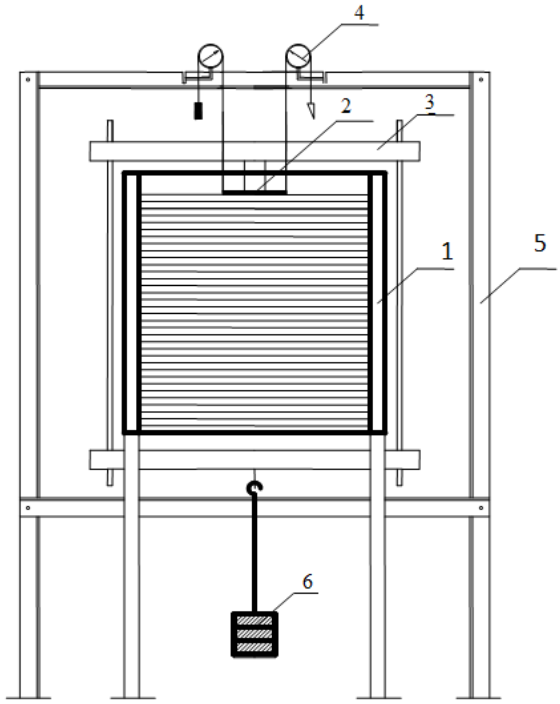


Figure 1 – Representation of the system for laboratory tray tests:
1 – metal tray; 2 – rigid strip stamp;
3 – load system; 4 – deflection indicator;
5 – metal bar for binding of indicators;
6 – cross-beam

In the first case the tests of the clay bases were carried out without reinforcing the CEE. Other tests were performed at different percentages of reinforcement *i* (ratio of the total volume of the SCE to the total volume of the reinforced array) with a depth of reinforcement of 100 mm.

Based on the experience of reinforcing the bases with vertical SCE [5 – 7] was accepted the variation of the percentage of reinforcement at three levels i = 2,1%; 4.4% and 7.1% (Fig. 2).

Clay paste was used to make the basis. The name of the paste corresponds to the heavy floury low-plasticity loam according to the classification of soils. For the preparation of the paste was used a natural soil, that selected from a foundation ditch in Poltava from a depth of 4 m.

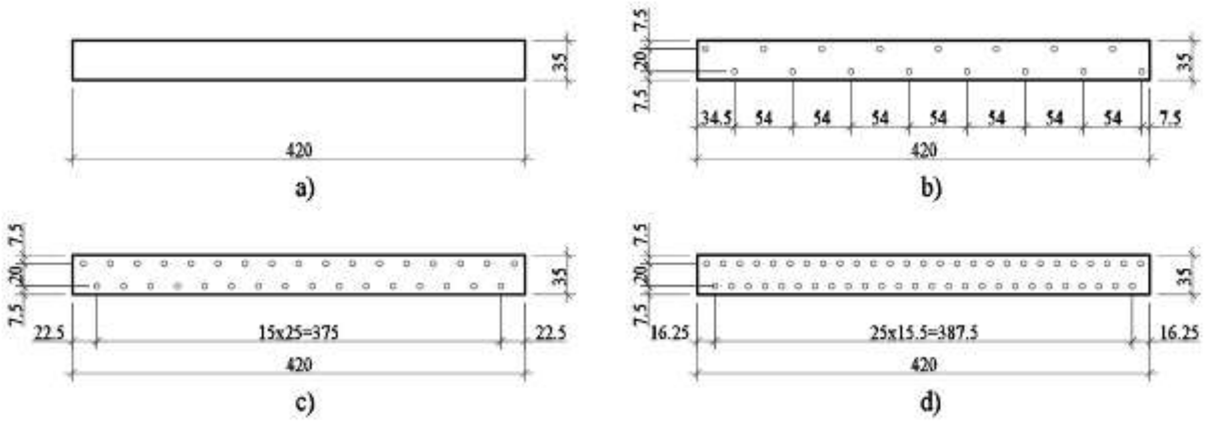


Figure 2 – The different schemes of SCE placement at the basis of the strip stamp depending on the percentage of reinforcement:
a – unreinforced base, b – at a percentage of reinforcement of 2.1%,
c – at a percentage of reinforcement of 4.4%, d – at a percentage of reinforcement of 7.1%

The following soil characteristics were determined by laboratory tests: solid particles density ρ_s = 2,68 g/cm^3; natural moisture of soil W = 0,205; moisture of soil at the liquid limit W_L = 0,369; moisture of soil at the plastic limit W_P = 0,224; index of plasticity I_P = 0,145. Based on the required dry soil density and water saturation conditions, the corresponding density and humidity values of the soil samples were calculated for the tests.

The following values of soil physical characteristics are calculated: dry soil density ρ_d = 1,45 g/cm^3; natural density ρ = 1,85 g/cm^3; natural moisture of soil W = 0,27; moisture of soil at the liquid limit W_L = 0,369; moisture of soil at the plastic limit W_P = 0,224; index of plasticity I_P = 0,145; index of liquidity I_L = 0,317; water saturation coefficient S_r = 0,85; void ratio e = 0,85.

To obtain the required soil characteristics for the experiment, taking into account the required soil moisture and density, an appropriate amount of soil (ground to a homogeneous state) and water for its additional moisture was determined. All components of the soil paste were calculated and selected by weight. The wetting of the required mass of soil powder was carried out by a spray of water with constant stirring of the mixture. The resulting paste was kept in plastic bags for three days to distribute the soil water evenly. In the next phase the paste was placed layer by layer into a tray. The thickness of each layer of clay paste was 15±2 mm. It was controlled by lines drawn on the glass of tray, and each layer of soil near the front wall of the tray was covered with a layer of chalk 1 mm thick. Soil compaction was carried out by tamping. During the filling, after completion of the test and before the next filling of the tray with clay paste, soil samples were taken from it to control its physical and mechanical characteristics.

To create a reinforced soil massive in the base of the rectangular strip stamp in the middle of the tray was arranged SCE reinforcement, based on the condition of the uniform placement of elements over the area of the base of the stamp.

Figure 3 – The tray filled with soil paste for stamping tests during the experiment

Soil-cement elements was made by filling pre-arranged wells with soil cement solution of such composition: soil (heavy floury loam); cement – 20% form the weight of dry soil; water from the condition of water-cement ratio – 3:1; solution hardening accelerator – calcium chloride ($CaCl_2$) in the amount of 1% by weight of water – which allowed to gain seven days' strength of soil-cement after 3 days. To prevent direct contact between the reinforcement elements and the stamp, a 10 mm thick layer of crushed stone of 2-4 mm fraction was poured over the reinforced massive.

The load on the stamp was transmitted through a loading system consisting of a steel channel and a steel pipe. Channel was mounted on top of the stamp (the load was transmitted through a steel prism of dimensions 250×35×70 mm). The pipe was hung to the channel using two levers as a suspension. A traverse was attached to it, on which crate cargoes were loaded. The own weight of the loading system together with the rectangular strip stamp was 282 N.

The pressure on the base was applied in stages: the initial degree was equal to the weight of the load system; each subsequent was 200 N. Each degree of pressure was maintained until the conditional stabilization of the basis settlements, the criterion of which was taken not exceeding the settlement of the stamp 0,1 mm in the last 2 hours of observations. Deformation fixation was performed after 1 min, 5 min, 15 min, 30 min, 60 min. and every 60 minutes after applying the next degree of pressure. The tests were stopped after reaching the basis of the limit of bearing capacity, which was manifested in the rapid increase of deformations without stabilizing.

In the course of conducting a series of stamp tests of the unreinforced and reinforced soil cement bases, the values of the average settlements of the basis of the stamp from the step-increasing pressure were obtained (Table 1). During the fixation of deformations in time the characteristic curves of settlements development of the basis of rigid strip stamp at each degree of pressure at different percentages of reinforcement of the basis are constructed.

Fig. 4 illustrates the dependence of the settlement of the basis of the rigid rectangular strip stamp at step load on pressure. Curve (1) shows an increase in settlement of the unreinforced soil massive from pressure, others show dependencies for the reinforced soil bases with percentages of reinforcement: 2.1% (2); 4.4% (3) and 7.1% (4). As we can see from the graphs, the strength of the soil base increases significantly as the reinforcement parameters increases. The designations F_1 and F_2 in Fig. 4 are characteristic points corresponding to the first and second critical forces, respectively.

Table 1 – Development of settlements of the basis of the stamp depending on the reinforcement percentage

The degree of step-increasing loading	Load, kN	Pressure, kPa	Average settlement at the percentage of reinforcement, mm			
			0,0%	2,1%	4,4%	7,1%
Initial conditions	0,000	0,0	0,00	0,00	0,00	0,00
Loading system	0,172	11,7	0,36	0,27	0,08	0,11
1 degree	0,372	25,3	1,42	0,73	0,34	0,29
2 degree	0,568	38,6	4,50	1,68	1,16	0,96
3 degree	0,768	52,2	6,27	2,36	1,61	1,42
4 degree	0,979	66,6	10,27	4,30	2,40	2,08
5 degree	1,178	80,2	11,53	6,06	3,76	2,57
6 degree	1,378	93,7	26,29	9,11	4,40	3,04
7 degree	1,578	107,3	-	10,98	6,75	4,06
8 degree	1,773	120,6	-	12,25	6,97	5,24
9 degree	1,971	134,1	-	17,03	9,19	7,77
10 degree	2,170	147,6	-	-	10,98	9,74
11 degree	2,370	161,2	-	-	13,90	11,86
12 degree	2,570	174,8	-	-	15,20	13,08
13 degree	2,765	188,1	-	-	19,72	15,69
14 degree	2,965	201,7	-	-	-	17,05
15 degree	3,165	215,3	-	-	-	19,44
16 degree	3,365	228,9	-	-	-	24,41

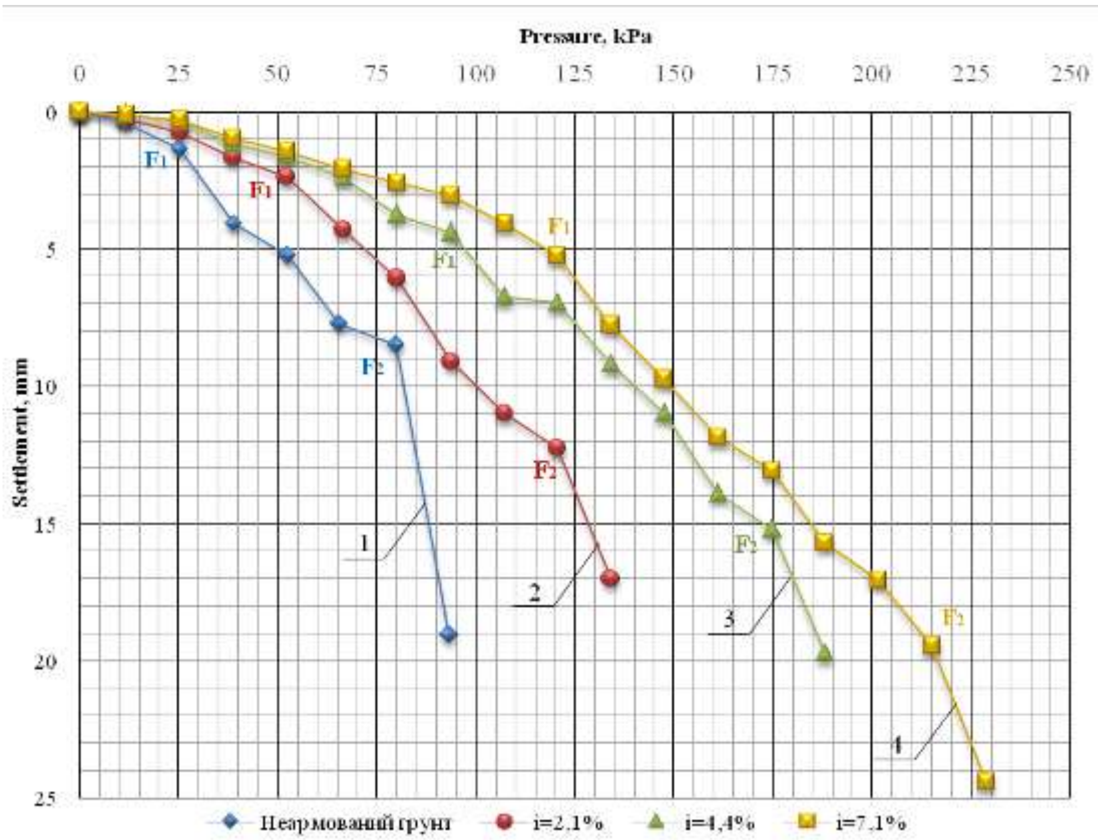
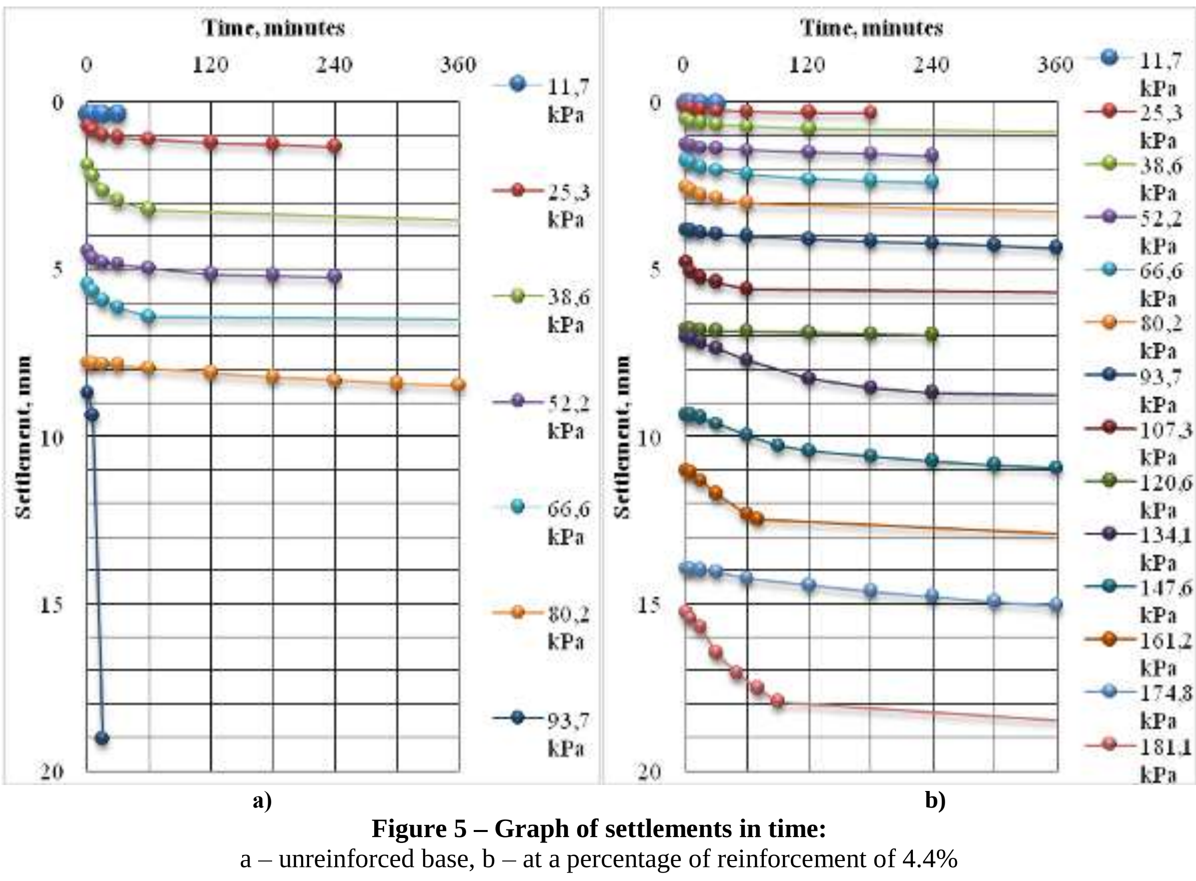


Figure 4 – Graph of dependence of settlements of the rigid strip stamp base on the pressure at step load:
a – unreinforced base, b – at a percentage of reinforcement of 2.1%,
c – at a percentage of reinforcement of 4.4%, d – at a percentage of reinforcement of 7.1% ;
F_1 – the first critical force; F_2 – the second critical force



a) b)

Figure 5 – Graph of settlements in time:
a – unreinforced base, b – at a percentage of reinforcement of 4.4%

Fig. 5a shows a graph of the development of settlements of the unreinforced stamp base in time at step load. In Fig. 5b we can see a graph of the development of settlements of the reinforced soil massive in time at step load on the example of settlement of the basis reinforced by SCE with a percentage of reinforcement i = 4.4%. The stabilization of the reinforced base at the same pressure occurs faster than the unreinforced one and increases with the increase of the percentage of reinforcement. Fig. 6 shows a graph of the settlements of the soil base at i = 4.4% at step-increasing load, taking into account the unloading of the system.

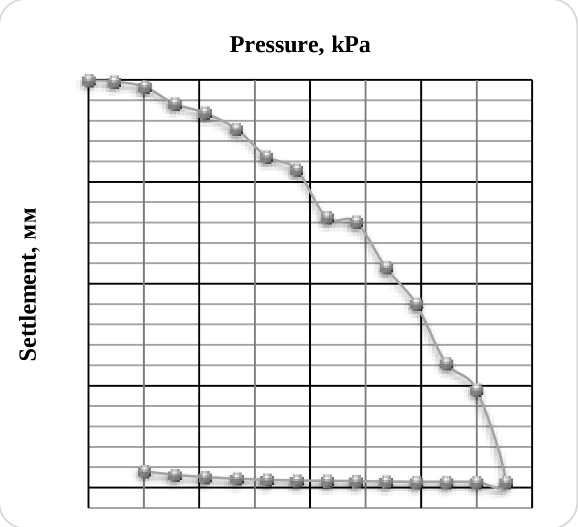


Figure 6 – Graph of settlements in time the base at a percentage of reinforcement of 4.4% with the unloading of the system

From the graph of the dependence of the base settlements under the rigid strip stamp on the pressure, certain regularities have been identified for its improvement by the SCE. Analyzing the experimental dependences of the settlements of the reinforced bases at stepping-increasing pressure (Fig. 3), it can be found that the bearing capacity (strength) of the bases increases with the percentage of soil reinforcement.

To identify the influence of the percentage of reinforcement on the change in bearing capacity on each of the graphs found characteristic points – the critical forces F_1 and F_2, which distinguish the transition from a linear relationship between stresses and deformations of the base (first and second critical pressures) [3]. It is known that the first critical pressure p_1 corresponds to the end of the compaction phase when no boundary condition is yet emerging at any point of the soil base and the second critical pressure on the soil is considered the maximum boundary pressure p_2 at which the full load carrying capacity of the soil takes place.

At characteristic points, the least-squares method found approximate dependences of the first critical pressure on the percentage of soil reinforcement (Fig. 7). According to the obtained values of the first critical pressure by the simplified Puzyrevsky formula, the specific clutch values for the soil under consideration were determined. This formula for perfectly connected clay soils ($\varphi \approx 0$; $c \neq 0$) has the form

$$p_1 = \pi c + \gamma d\,. \tag{1}$$

The Prandtl solution was adopted to determine the specific cohesion by 2 critical pressure

$$p_2 = 5{,}14c + \gamma d, \tag{2}$$

where c – specific cohesion; γ – soil density; d – foundation depth.

With the given parameters of the soil base, the dimensions of the stamp and the height of the reinforcement, the bearing capacity of the base increases with the percentage of reinforcement (Table 2).

Table 2 – Determined critical pressures with varying parameters of the percentage of base reinforcement

The percentage of reinforcement i, %		The first critical pressure	The second critical pressure
0,0	P, kPa	25,3	79,9
	s, mm	1,35	8,48
2,1	P, kPa	52,2	120,3
	s, mm	2,36	12,25
4,4	P, kPa	93,7	174,8
	s, mm	4,40	15,20
7,1	P, kPa	120,6	221,3
	s, mm	5,24	19,44

The first critical pressure for unreinforced weak soil was approximately 25.3 kPa, for a reinforced base with a reinforcement percentage of 2.1%; 4.4%; 7.1% is 52.2 kPa, 93.7 kPa and 120.6 kPa, respectively. The second critical pressure was 79.9 kPa, 120.3 kPa, 174.8 kPa and 215.3 kPa, respectively. In this case, the critical force F_1 corresponds to settlement of 1.4-5.2 mm, and F_2 – 8.5-19.4 mm, depending on the percentage of reinforcement of the base.

Table 3 – The results of the approximation of the experimental data of the dependence of the critical pressures on the base

The first critical pressure, kPa		The second critical pressure, kPa	
$p_1 = 25{,}97+13{,}81i$		$p_2 = 81{,}57+19{,}43i$	
correlation coefficient	variation coefficient	correlation coefficient	variation coefficient
$r = 0{,}995$	$v = 0{,}054$	$r = 0{,}996$	$v = 0{,}031$

As the percentage of base reinforcement increased, the settlement value at which the critical pressures p_1 and p_2 on the base were determined increased. The values of the statistical indices (correlation coefficient *r* and variation coefficient *v*) of the obtained dependences indicate an increase in the value of the first critical pressure with increasing percentage of reinforcement by linear dependence (Fig. 7). In the table 3 shows the empirical equations of these dependencies and the statistical accuracy of the approximation of the data for these equations.

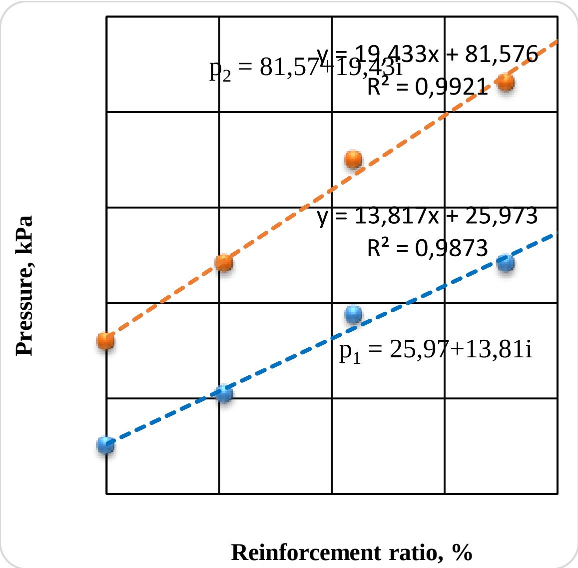


Figure 7 – Dependences of the critical pressure of the soil base bearing capacity on the ratio of reinforcement

The results of determining the specific cohesion for the reinforced and unreinforced bases of the rigid strip stamp are summarized in Table 4. It should be noted that the obtained value of the specific cohesion on the second critical pressure is higher, and as the percentage of reinforcement of the base increases the difference in values between the solutions of Prandtl and Puzyrevsky formulas decreases.

Table 4 – Specific cohesion, determined by the critical pressure on the base

Soil base		Specific cohesion, kPa (determined by critical pressures)	
Dry soil density, г/см^3	Reinforcement, %	the first critical pressure	the second critical pressure
1,45	0,0	8,3	15,8
	2,1	17,5	23,8
	4,4	27,6	32,5
	7,1	39,5	42,7

Conclusions

From the analysis of tray researches of pressure-settlement dependence, we can conclude that the soil base under the strip foundations is being improved by the method of reinforcing by SCE, in particular:

1. From the graph of settlement on load it can be observed that the parameters of the soil base and the height of reinforcement of the first and second critical pressures of the system "reinforced base – the strip foundation" increase in the parameters specified in this work.
2. The first and second critical pressures, depending on the percentage of reinforcement of the base, increases linearly.
3. With the percentage of base reinforcement increment, the amount of sedimentation at which the first and second critical pressures on the base were determined increased.
4. The determination of the specific cohesion of the soil from the obtained dependences on the first and second critical pressures due to the Puzyrevsky and Prandtl formulas for reinforced soil differs, but within the tolerable error.

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RESIDENTIAL BUILDING'S DEFORMATION ON PILE FOUNDATION

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A considerably distinctive deformed five-storeyed three-section brick residential house erected over 40 years ago on driven prismatic piles combined by a strip grid foundation by a standardized project is under investigation. The geotechnical monitoring results of the building's technical condition are presented. Methods and results of the instrumental control analysis of the piles' actual length and visual evaluation of their integrity are presented. The causes of the foundations base's excess deformation were determined: the inability of the piles' tips to reach the designed mark; the piles' destruction during their immersion in dense sands; rupture of the primary thermal network, which led to "negative friction" effect on the piles' lateral surface, etc.

Keywords: soil base, sand, soft soil, driven prismatic pile, grid foundation, settlement, crack, tension bar, engineering status of the building, geotechnical monitoring.

ДЕФОРМАЦІЇ ЖИТЛОВОГО БУДИНКУ НА ПАЛЬОВИХ ФУНДАМЕНТАХ

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Подано результати геотехнічного моніторингу технічного стану достатньо характерного деформованого п'ятиповерхового трисекційного цегляного житлового будинку, зведеного понад 40 років тому за типовим проектом на забивних призматичних палях, об'єднаних стрічковим залізобетонним ростверком. Виконано візуальне та інструментальне обстеження несучих будівельних конструкцій. Доведено, що конструктивну схему будівлі не можна вважати жорсткою. Виявлено дефекти й пошкодження, що впливають на несучу здатність та довговічність окремих конструктивних елементів і будівлі в цілому, зокрема вертикальні тріщини із шириною розкриття до 20 мм у зовнішніх та внутрішніх стінах, тріщини в збірних перемичках у підвалі, обрив попередньо напружених тяжів та ін. Визначено фактичні параметри основ і фундаментів. Описано методику та викладено результати аналізу інструментального контролю фактичної довжини паль і візуального оцінювання їх цілісності. Розраховано проектну та фактичну несучу здатність паль і осідання їх основ. Установлено причини наднормових деформацій основ фундаментів, зокрема: неможливість досягнути вістрями паль проектної позначки, що призвело до значного зменшення їх несучої здатності; руйнування паль у процесі їх занурення у щільні піски намиву; порив магістральної теплової мережі, котрий призвів до появи ефекту «негативного тертя» за бічною поверхнею паль; дія інерційних сил від вибухів у кар'єрі. Технічний стан пальових фундаментів визначено як незадовільний. Розроблено рекомендації щодо подальшої експлуатації будівлі. Обрано конструктивно-технологічне рішення посилення її фундаментів, зокрема посилення пальових фундаментів підведенням під існуючі ростверки монолітної залізобетонної плити товщиною не менше 400 мм, захватками та поетапно. Для включення плити «в роботу» одразу після її влаштування передбачено ущільнення основи під плитою.

Ключові слова: ґрунтова основа, пісок, слабкий ґрунт, забивна призматична паля, монолітний залізобетонний ростверк, осідання, тріщина, металевий тяж, технічний стан будівлі, геотехнічний моніторинг.

Introduction

The group of deformed structures [1] includes buildings that have undergone unacceptable settlement and deformation during the period of their construction and especially exploitation, which, however, do not interfere with its operation but limiting the possibility of its strengthening in time.

The absolute and specific settlement of the foundation's base, heeling and other deformations of the buildings and structures occur out of fairly typical errors, which are usually combined into four groups [1–7], in:

– geotechnical surveys (insufficient field and laboratory investigation, wrong determination of soil properties, neglecting of strata, layers, soft soil lenses within the base of a building, incorrect estimation of spatial heterogeneity of soil massif, neglecting of annual and seasonal fluctuations in groundwater level, other dangerous geotechnical processes, etc.) [1 – 9];

– poor design (in particular, the failure to account for the specific properties of soils within the compressible strata of the base, variation of the soil base's physical and mechanical properties, underestimation of the composition and essence of the load transfer on the foundation and base, the form complexity and deficient building's rigidity, cancellation or decreasing the number of pile static tests at the site, etc.) [1–14];

– violations of the building operations rules (such as the use of so-called “critical technologies” of soil structure distortion within its cyclic wetness change, partial freezing of foundation soil, poor arrangement of sand and soil beds, excess soil extraction from the pit, pile driving above the designed mark, under-compaction of the backfill soil in the pit hollows, etc.) [1–8];

– violations of the buildings and structures operation rules (for example, prolonged and accidental leaks from water-bearing communications, unreasonable increase in loads, construction works near the facility in the context of dense urban development, etc.) [1–4].

Constructive and technological solutions for the strengthening or reconstruction for each such object is a complex and relevant task of modern geotechnics, which is made by the investigation results of [1–3, 7]:

– regularities of geotechnical conditions change (GTC) of the site and stress-strained state (SSS) of building base;

– design features and operating conditions of the facilities;

– nature and causes of buildings' deformation;

– buildings foundations' stability under operation while its reconstruction (restoration).

Review of the research sources and publications

Prof. B. Dalmatov considered the settlement of each foundation base to be the sum of five components: settlement due to the compaction of the natural structure soils at the increasing stresses from the foundations' weight; settlement associated with decompression of the upper soil strata that lies below the bottom of a foundation pit, due to the reduction of stresses during excavation; settlement due to the soil squeezing (extrusion) from beneath the foundation caused by the progression of plastic deformations; settlement of the disruption, which progresses due to the soil compressibility increase at its natural structure distortion during execution; settlement caused by changes in the stress state or deformation of the soil base during the building (structure) operation [3, 15].

At the same time, prolonged geodetic surveys of the building's foundations settlement on pile foundations [7–10, 14, 16, 17] showed that both absolute and relative stabilized values of settlement are in most cases smaller than its calculated and normative limit values. Naturally, the deformed structures group includes buildings on pile foundations (foundations that are constructed without soil excavation) slightly less frequently than similar objects on a natural base (foundations built with soil excavation [7]).

In addition to the above, the causes of excessive absolute and relative settlement of pile foundations base (and occurrence and propagation of cracks and other noticeable deformations in load-bearing structures as a consequence) are most often [1 – 10, 14]:

– unjustified application of increasing correction coefficients to the results of compression tests of soft (highly compressive) soils (long geodetic surveys of settlement on soft, including wetted loess soil, indicate that the calculations of settlement should be performed [7, 10, 13, 14]);

– catching the bottom tips of piles into strata (layers) of soft soil (furthermore, under these conditions stabilization of settlement can be realized more than 10 – 15 years of building operation [7, 10, 14]);

– pile driving above the design mark, a characteristic problem is determining the depth of pile driving and the presence of the defect in it;

– overestimation of the piles bearing capacity due to failure to observe the optimum time of its “rest” after immersion or misinterpretation of “load on the pile - its subsidence “graphs;

– excessively close placement of adjacent piles in planar view, which, when driven especially in the sands, leads to “ejection” upwards of previously driven ones, and under the construction load the structure undergoes respective settlement;

– nonuniform loading of piles in the grid's structure, etc.

Definition of unsolved aspects of the problem

It is problematic to account for the influence of these factors on the magnitude of pile foundations base deformation, especially by methods of classical soil mechanics [6, 12]. Therefore, the choice of structural and technological solutions for the reinforcement (reconstruction) of each deformed building on the pile foundation is made only after a careful assessment of the technical condition of the object's load-bearing structure and the investigation of the parameters of its base and foundations, by engineering inspection and defining the causes of excess deformations of the foundations' base.

Problem statement

So, the research aims to evaluate technical condition of load-bearing structures of a sufficiently deformed characteristic (with numerous, mostly vertical, force cracks) full-scale object, five-storeyed three-section brick residential building on driven prismatic piles combined by a strip grid foundation, and determine the parameters of bases and foundations by engineering inspection and defining the causes of excess deformation of the foundations' bases.

Basic material and results

The research algorithm, the so-called geotechnical monitoring [3, 5], included the following steps:

– visual and instrumental inspection of load-bearing structures;

– measurements in the scope required for the verification of these structures' calculations ;

– verification calculations of load-bearing structures;

– assessment of the technical condition of load-bearing structures based on inspection and verification calculations;

– determining the parameters of bases and foundations, including the actual length of piles;

– defining the causes of excessive deformation of the building foundations' base;

– development of recommendations for the further operation of the building and the choice of structural and technological solutions to strengthen its foundations.

The five-storeyed residential building with a basement and a technical floor is located in Horishni plavni, Poltava region. It was erected in 1977 according to a typical project, consists of three-block sections: 87-049 / 71 – the left end, which has undergone significant deformations (Figs. 1 and 2); 87-046 – ordinary; 87-048 / 71 – the right end.

In structural terms, the building is a structure with longitudinal bearing walls. The height of the floors is 2.8 m, and the basement (from the floor to the bottom of the slab panel) is 2.1 – 2.2 m. The walls are made of solid one and a half silicate brick on a cement-sand mixture. The thickness of the outer longitudinal bearing and end walls without facing is 510 mm. The thickness of the inner longitudinal load-bearing walls with the plaster layer is 400 mm. The load-bearing walls in the basement of the house from the -2.800 to -1.600 mark are made of 2 rows of FBS 24.5.6-T blocks, and of brick masonry higher. Spatial rigidity is provided by the transverse staircase walls and inter flooring discs. The building's structural design can't be considered rigid. The class of consequences of the building – SS-2. The main thermal network with a diameter of 2×150, 125, 100 mm laid through the basement. Its inlet to the basement is located below the left block section (axis 1-2) from the side of the staircase wall. Below the section, there is a 90° rotation of the pipeline that passes through the entire basement and comes out on the opposite end of the house.

There were defects and damages during the construction and operation of the building affecting the load-bearing capacity and durability of individual elements and the building as a whole, such as:

– vertical cracks with opening width up to 20 mm in the outer and inner walls;

– pile's destruction due to deviation from the design position;

– cracks in precast spandrel beam in the basement;

– wetting of the basement structures of the building at the exit point of the main thermal pipeline due to its rupture;

– destruction of paving on the perimeter of the building;

– break of pre-stressed tension bars.

Thus, vertical cracks (Fig. 3) were detected at the entire height in the outer load-bearing walls, mainly at the places of the spandrel beam supporting and over (under) the openings.

Cracks are typical for buildings with reinforced concrete spandrel beams that cause deformation of the window unit by turning it as a single rigid element. Cracks indicate significant deformation of the foundations' base of the left end section in the direction of separation from the main part of the building. The cracks propagation significantly reduces the spatial rigidity of the section because, for separated parts of the outer and inner walls, the spatial rigidity is no longer provided by the transverse walls of the stairwell. There are no cracks on the outer and inner walls of the staircase, which confirms the previous statements.

Figure 1 – Fragment of the left block section on the axis B

Figure 2 – View of the left side block section in the axes A-B and on the axis A

a

b

c

d

Figure 3 – Vertical cracks with opening width:
a – 7 mm; b – 15 mm; c – 12 mm; d – 4 mm

Minor cracks in the walls and between the floor slabs occurred immediately after the house was occupied in 1977, but it suffered the greatest deformations in 1993 under the left block section I-II, in the axes 1 – 2 after the rapture of thermal pipeline in the spot of 90° turn. At the same time, to reduce deformations, the pre-stressed tension bars (Figs. 1, 2) of 36 mm diameter reinforcing bars were arranged with struts to create tension in it. Horizontal beams of the rectangular cross-section of two welded channels No. 24 are installed along the end walls, and the attachment of the weights to the beams is through an equal angle 140x10, which is installed in the corners of the building. Given the fact, that the free length between the points of attachment should not exceed 15 – 20 m (in our case it was 67.2 m), the cracks were opened in the future, since at considerable length the elastic absolute deformations in the steel of tension bars are very large. At the level of overlapping of the third and fifth floors, there was a break in the metal tension bar (Fig. 4), indicating a poor design of the reinforcement. Gypsum tell-tales were placed on the inner load-bearing wall, starting from the third floor, in the area of the stairwell. Cracks in it indicate the further development of deformations (Fig. 5). On the end wall of the left block section cracks are missing.

Therefore, the technical condition of the outer and inner bearing walls is classified as unsatisfactory.

Figure 4 – Break of the upper tension bar along axis A at the level of the fifth floor overlap

Figure 5 – Crack in the tell-tale (fifth floor)

Overlappings are the precast concrete void slabs 5.4 m long. There were cracks in the joints between the slabs in the basement and on the fifth floor, some joints unfilled with the mixture, which reduces the spatial rigidity of the building. The technical condition of the overlaps is classified as satisfactory.

The foundations are driven prismatic piles SNIP 9–35 (9 m length, 350×350 mm section), combined by a strip grid of 400 mm height. Under the inner load-bearing wall, the grid width is 400 mm, and under the outer wall, it is 500 mm. The increment of the piles under the inner wall is 1100 mm, under the outer wall is 1360 – 1530 mm, and under the end walls – 1590 – 1610 mm.

Lithologically, within the boundary of the site under the bulk layer (EGE-1a) and fine sands (EGE-2c and EGE-2p, respectively, of medium density and dense with modulus of deformation, respectively, E = 19.5 and 35 MPa) with a total depth of about 7 m, there is a layer of buried soils (EGE-3) - layered sandy loams, with layers of silt and clay, fluxional (E = 6.5 MPa, organic matter content – 8%), which are underlined by medium-sized alluvial sands of 9 – 10 m deep (EGE-4, E = 45 MPa, and from a depth of about 18 m -- by the clay of the Kharkiv formation. A sand wash on the site for construction was carried out in the the 1970 – 1973 period. From the graphs of static probing, it is possible to conclude that these sands have stabilized and self-compacted by the time of the investigation. Groundwater level (WL) at the time of the survey was 6.8 – 7.3 m from the earth's surface. Its annual and seasonal fluctuations reach 1.5 m from this level.

Unfavorable engineering-geological processes have been identified within site: dynamic impact on sandy soils from career explosions, which can lead to their dynamic liquefaction; mechanical suffusion during the operation of water-bearing communications; rather thick (up to 2.3 m) soil with impurities of organic matter.

The project provided for penetrating the buried soil with СНпр 9 – 35 piles, that reached IGE-4. In this case, the calculation revealed that: the load on the pile under the inner and outer bearing wall is 404.5 and 390.6 kN, respectively; pile's bearing capacity is Fd = 1334.8 kN; allowable design load N = 953.4 kN; the subsidence of the base of such foundation is S = 1,44 cm.

Considering the above and the worst possible scenario, under which accompanied by the rapture of the thermal pipeline could have the effect of “negative friction” on the lateral surface of the pile, the magnitude of which can reach 317 kN (soil layers EGE-2c and EGE-2p), allowable calculated load on the pile N = 499.5 kN which still exceeds the pile load from the structures 404.5 kN. Therefore, the actual length of the piles in the foundation was checked. It should be added that during static probing, the probe was not always able to pass the thickness of the washed sands, in particular EGE-2p, which is dense. It is problematic to penetrate these layers with the hammering with a diesel-hammer pile with a cross-section of 350x350 mm.

Thus, continuity and length control of the piles was performed acoustically using the software package Pile Integrity Tester PIT – W (Pile Dynamics in the USA). The PIT complex – W (US production) has a marginal relative error in determining the linear dimensions of structures of ± 5%. The relative error of determining the propagation time of the signal in the frequency range is f = 1.9 kHz is ± 1%. The complex underwent metrological verification in the center of standardization, metrology, and certification. The RIT™ Solestog is based on the Pulse Reflection Method (PRM). Initially, it sends an impulse that strikes lightly on the surface of the pile. To do this, a special hand hammer is utilized. The impulse-induced acoustic wave propagates along the pile. The pile's shape and its material's quality affect the reflected waves, which are recorded as they return to the surface. All surface vibrations are recorded to obtain information on all major reflected signals. Next, to judge the integrity of the pile, the reflected waves are analyzed, accounting for its nature and intensity.

6 exploratory shafts were made for these tests, and the pile body was cleared by 20 cm, 3 of them located in the damaged left block section (axis I – II), 2 – in the row block section (axis II – III) and 1 – in the right block sections (axis III – IV in Fig. 6). The pile length control process is shown in Figure 7.

To confirm the results of visual and instrumental control of the pile's integrity and its length, the pile heads “cutting” possibility was checked. For this purpose, the pile head's reinforcement design was compared with the actual one (the first 300 mm of the head were reinforced with 50 mm grids, and then a 700 mm spiral began with a 100 mm shear increments, going into a 200 mm shear cross spiral). The concrete protective layer was previously removed.

a

b

Figure 6 – The piles view as a part of the grid: a – №1 (pile №119); b – №2 (pile №105)

Figure 7 – Performing pile length control work

Compared to the nondestructive testing stage, during the investigation of the pile's reinforcement, the exploration shaft was deepened, revealing cracks in the pile body (Fig. 8). The pile was destroyed since the protective layer of concrete was split in a stretched zone. The concrete in the compressed area split and crumbled. The increment of the transverse reinforcement was 200 mm. It shows that the pile was not driven to the designed mark, it was “cut down”. The minimum length of the “cut” part was 600 mm. In this state, the pile does not withstand vertical compressive loads. The results of the instrumental control analysis of the pile's length and visual assessment of their integrity are summarized in Table 1.

Table 1 – Instrumental control results of piles' length and integrity

Shaft / pile №	Actual pile length, m	Notes
1/119*	Pile destroyed	The pile damage due to deviation from the design position while driving. The pile operates on eccentric compression due to the considerable eccentricity of loading and pile's body deflection from vertical. The crack opening in the pile - 40 mm.
2/105*	8.5	–
3/9*	4.5	–
4/77	8.5	–
5/171	6.0	–
6/33	7.5	–

* the selected shafts and piles in it, located in the left end block section, which was most deformed

The pile reinforcement check confirms the assumption that part of the pile failed to be driven to the design mark. The piles' lengths discovered in the instrumental survey are true. To establish the possible difference in settlement, a test calculation was performed for pile # 119 (shaft # 1) of a 4.5 m length with a cross-section of 350x350 mm. A vertical tie of this pile to the geotechnical column is shown in Fig. 9.

a

b

Figure 8 – The piles' view as grid component in pile reinforcement investigation:
a – pile №119 in shaft №1;
b – pile №105 in shaft №2 (the reinforcement increment in the survey was 100 mm, therefore, the pile's tip reached the design mark)

The following results were obtained in the calculation. Settlement of the pile is S = 2.2 – 2.5 cm based on the calculation method. Thus, the difference between the settlement of piles with a length of 9 m and 4.5 m is about 1 cm, which could not cause actual over-deformation.

However, given the occurrence of “negative friction” effect due to self-compacting and mechanical suffusion in the upper layers of the bulk sands after rupture of the main thermal pipeline, which was intensified by inertial forces from the explosions in the quarry, the bearing capacity of the pile decreased to Fd = 375.7 kN, it is up to N = 268.0 kN, which is less than the vertical force from the structures of the building 404.5 kN.

When the permissible load on the pile was exceeded, the pile's foundation base settlement was already in the nonlinear stage, which led to the appearance and development of existing deformations of the building. Provided that if all the piles' tips reached the design mark, then, of course, such deformation would not occur.

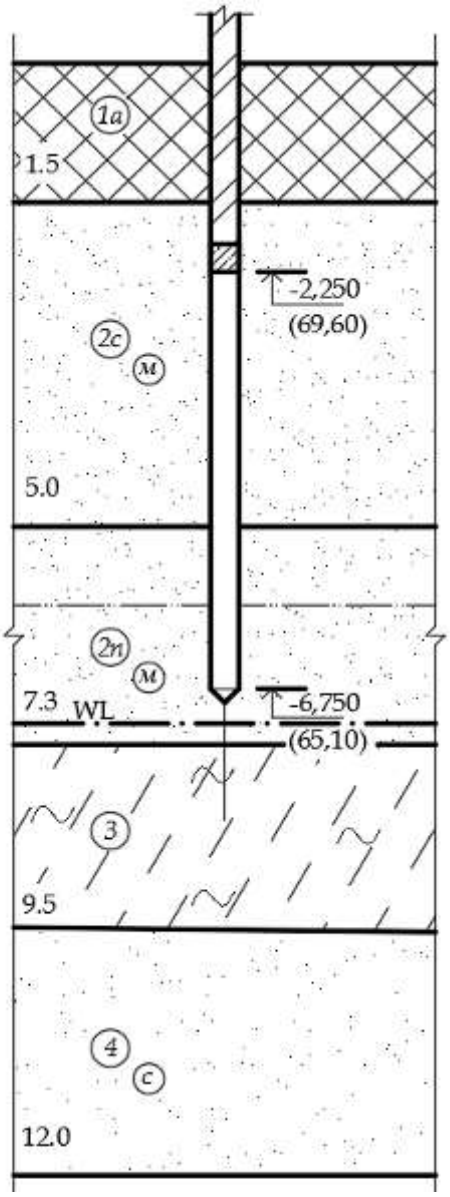

Figure 9 – The vertical tie of the pile to geotechnical column according to SNIP 4.5 – 35

Therefore, the technical condition of the pile foundations of the building was classified as unsatisfactory.

As the survey found that some piles were significantly damaged, the length of others did not correspond to the design, it is not possible to guarantee the absence of water-bearing communications for the future and to eliminate the negative impact of the explosions in the quarry on the wash sands, it was decided to develop a project to strengthen the pile foundations.

It's worth noting that if similar sections of the water supply network crashed in other sections of the building, it would get comparable damage. Therefore, to ensure the prolonged operation of the building for the period of reconstruction and afterward, the decision was made to develop an appropriate project, which would include a set of measures, that would include:

– reinforcement of the pile foundations of the left face block section in the I-II axes by underlaying at least 400 mm thickness of monolithic reinforced concrete slab under the existing grids. The underlaying of the slab must be done in four stages, starting from axis II, moving to axis I. To put the plate into operation as soon as it is performed, the base under the slab should be compacted by gravel and pounded with vibrating plates;

– strengthen the outer and inner bearing longitudinal walls to prevent the reduction of its spatial rigidity by dividing them into separate blocks (columns) by cracks. For this purpose it is possible to use already existing inefficient metal structures of reinforcement or to execute reinforcement with outer metal linings from rolling profiles;

– to chipper and rip cracks open with M 100 cement sand mixture (alternatively, injection of these cracks with M 100 solution);

– eliminate depressurization of the main thermal pipeline at the exit from the right end block section in axes III-IV;

– replace the metal of main thermal pipelines, which have been in operation for 37 years, since its standard service term is only 30 years and it is not possible to guarantee its hermiticity or to move the pipeline from the basement of the house;

– strengthen reinforced concrete spandrel beam in the inner bearing wall of the basement in axes I-II on axis B;

– restore the protective layer of concrete piles in which their reinforcement was investigated;

– to restore the design perimeter walk along the building and to organize vertical planning of the territory for surface water abstraction.

Finally, it should be noted that such a project was completed and successfully implemented. Now the building is operated normally.

Conclusions

Thus, according to the results of geotechnical monitoring of the technical condition of a considerably distinctive deformed five-storeyed three-section brick residential house erected over 40 years ago on driven prismatic piles combined by a strip grid foundation by a standardized project, the following conclusions are made.

1. In structural terms, the building is a structure with longitudinal bearing brick walls. Its spatial rigidity is provided by the transverse walls of the staircase, as well as the inter-floor disks. The structural design of the building cannot be considered rigid. The main thermal network passes through the basement of the building.

2. During the erection and operation years, defects and damages occurred in the building's structures affecting both the bearing capacity and the durability of the individual structural units and the whole building, that can be classified as follows:

– vertical cracks with opening widths up to 20 mm in the outer and inner walls;

– vertical cracks with opening widths up to 20 mm in the outer and inner walls

– destruction of the pile due to deviation from the design position;
– cracks in precast spandrel beams in the basement;
– wetting of the building's basement structures at the exit point of the main thermal pipeline due to its rupture;
– the destruction of the building's perimeter paving ;
– pre-stressed tension bars' breakoff.

3. The causes of the occurrence and development of excessive deformations in load-bearing units of the structure are a complex of factors, that includes:
– the inability to reach the design mark by the piles' tips during the construction, which led to a significant reduction of its bearing capacity;
– the destruction of piles during its driving in dense wash sands;
– rupture of the main thermal pipeline, which led to the occurrence of the "negative friction" effect (pile overloading due to the settlement of the surrounding soil, causing forces to act downwards on the pile) along the lateral surface of the piles;
– the impact of inertial forces from career explosions.

4. The technical condition of the pile foundations was estimated as unsatisfactory, and therefore, to ensure the further operation of the building, an appropriate project was developed to provide a set of measures, the main of which is the pile foundations' strengthening under the left end block section by extending at least 400 mm thick monolith reinforced concrete plate under the existing grids, by the divisions, and in stages. To incorporate the plate "into the framework" it is necessary to compact the base under the plate immediately after its completion.

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METHOD OF FORECASTING THE TERMS OF SETTLING OF STRUCTURES ON SOILS WITH SILT LAYERS

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The method of forecasting the size and timing of sedimentation of structures on soils with silt layers, based on the results of field observations, has been developed. It is suggested to use a set of exponential dependencies with a constant component. The algorithm and software for calculation of the envelope by experimental data by a stepwise approximation are developed. The process of consolidation of soils is considered as a combination of simultaneous and independent flow of phases of primary filtration consolidation and secondary consolidation of creep. According to the results of data processing of observations by direct iterative calculations by finding the minimum nonconnectedness by the method of least squares.

Keywords: base, silt, structure, precipitation, forecast, size, term, method, calculation

МЕТОДИКА ПРОГНОЗУВАННЯ ТЕРМІНІВ ОСІДАННЯ СПОРУД НА ҐРУНТАХ З ШАРАМИ МУЛУ

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Розроблено метод прогнозування величини та термінів осідання споруд на ґрунтах з шарами мулу, заснований на результатах натурних спостережень. Сформульовані вимоги до залежностей апроксимації процесу консолідації мулів. Запропоновано застосовувати для наближення сукупність експоненційних залежностей з постійною складовою. Розроблено алгоритм та програмне забезпечення для розрахунку огинаючої до експериментальних даних. Процес консолідації ґрунтів розглянуто як сукупність одночасного і незалежного протікання фаз первинної фільтраційної консолідації і вторинної консолідації повзучості. Апробація розробленої методології здійснено під час спостереження за осіданням масивів-гігантів хвилелому Карантинної бухти Одеського морського порту. Вимірювання поточних значень висоти марок, в якості яких були призначені точки в чотирьох кутах верхньої поверхні кожного з масивів-гігантів, здійснювалось тахеометром від площадки контейнерного терміналу з приведенням до висоти рівня моря в акваторії Одеського порту. За результатами обробки даних спостереження прямими ітераційними розрахунками шляхом пошуку мінімальної незв'язаності методом найменших квадратів отримані значення параметрів наближення осідання реперних точок, які відповідають потребам практичного застосування. В якості критерію завершення процесу консолідації запропоновано приймати момент стабілізації значень осідань з міліметровою точністю з урахуванням отриманих відхилень. Отримані експериментальні залежності для розрахунків швидкості осідання точок масивів-гігантів. Наведено приклади розрахунку прогнозів величини та термінів осідання огороджувального хвилелому контейнерного терміналу Карантинної бухти. Встановлені терміни завершення процесів консолідації мулоподібних ґрунтів основи волнолому в місці розташування масивів-гігантів і остаточне положення поверхонь масивів-гігантів. Представлена методологія прогнозу осідання основ з мулоподібними ґрунтами рекомендується для оцінювання впливу консолідації мулів на осідки елементів конструкції при будівництві та під час експлуатації споруд.

Ключові слова: основа, мул, споруда, осідання, прогноз, величина, термін, метод, розрахунок

Introduction

Artificial and natural soil deposits, as a rule, represent a combination of several materials, such as sand, silt, clay, gravel or mixtures of different proportions. In this case, the occurrence of individual geological layers of the soil is characterized by a variety of forms and variability of thickness, which is especially manifested for structures of significant size, such as dams and wave breams. Design and construction on soils containing layers of silt of a considerable thickness requires an additional increase in the height of the buildings by the size of the construction lift. At the same time, calculations of settling elements of structures, which will be based on such complex foundations, at the design stage are extremely complicated and are preliminary. Current construction norms [1-3] require their refinement on the basis of field observations of the state of the buildings both at the stages of construction and operation.

Review of research sources and publications

At present, the basic method for calculating the settling of layered bases is a layer-summing method [1, 4-10]. The method is based on the summation of sedimentation of elementary layers of the base within the compressed layer arising from the loads transferred by the structure. In order to take into account the influence of the geometrical features of the construction, the flat and spatial simulation of deformation processes in soils using PLAXIS, ANSYS, LS-DYNA, Midas, Z-Soil, Phase2 and others [11-15] is also used. In this case, the classical models of deformed solid media are accepted for calculations as well as the mechanical properties of materials of separate layers according to the results of laboratory or field tests [16-18] within the framework of accepted models.

Definition of unsolved aspects of the problem

Difficulties in taking into account parameters of deformation and viscosity of real soils, such as waterlogged sands, mules, complexity of designation in calculations of the depth of the compressive layer of the base in some cases do not allow to obtain with the necessary accuracy the value of complete settling of the structure and to estimate the development of precipitation in time. Thus, the deformation of the mules, accompanied by the consolidation with the extrusion of the liquid and gaseous phases and the plastic deformation of the solid residue, has so far no analytical description and model representation. Therefore, the development of methods for controlling sedimentation of building elements in the course of construction and methods for forecasting their development in time is an actual scientific task for the construction industry.

Problem statement

The purpose of the work is to develop a method for forecasting the size and terms of settling of structures on soils with mud layers on the basis of field observations at the stages of construction and operation.

Basic material and results

1. Characteristics of the consolidation of mulch-like soils

According to DSTU B B.2.1-2-96, multidimensional soils or muds include water-saturated modern sediments mainly in marine waters containing organic matter in the form of plant residues and humus. Usually in a mule, the content of particles less than 0.01 mm is 30-50% by weight. Upper layers of sludge have a fairly high value of the coefficient of porosity (more than 0.9) and flowing consistency. When compressed in muddy soils, water filtration takes place, and this consequently leads to filtration consolidation. However, accumulation of muddy soils (sealing) does not stop even after the process of filtration consolidation (when the water pressure is near zero), but continues for a long time due to the creep of the skeleton of the soil.

Consolidation is a process of development in time of fading deformation of the seal in compression conditions. Consolidation of soils refers to rheological properties, more precisely to voluminous creep. Bulk creep characterizes the process of time development of volumetric deformations of the soil, which arise in the general case under the influence of average effective stresses on the main axes.

Deformations of volumetric creep developing in the soil in time are:

– fading ones;

– nonlinear ones

as a result of viscous resistance of interbranch connections.

The term «fading deformations» characterizes the presence of the limit of the value of deformation for each value of the existing stresses. Compatible with the term «nonlinear» and «fading deformations» indicate that, for fixed load conditions, the deformation curve of the mules in time has the character of asymptotic approximation to the horizontal line, which is the strain point for a specific base containing layers of mules.

Load conditions are given:

– mode – the time of transfer of the load on the element of the soil and the time of its holding;

– conditions for squeezing porous water from the soil element – with the ability to squeeze porous water (open circuit) and with the conditions under which spin-off porous water is excluded (closed circuit).

During volumetric creep, due to the seal of the soil element, the volume of the soil particle itself decreases by squeezing out pores as liquids and displacement of porous air. Therefore, in general, volumetric creep can develop in both dry and wet soils, in lime and frozen, in rocky and dispersed soils. When compressing muddy soils there is a coherent flow of the following processes:

– filtration consolidation;

– formation of solid precipitation in time;

– creep of the skeleton of the mineral component;

– mineralization of organic impurities of mules,

– the manifestation of which must be considered together.

Consolidation is a partial type of volumetric creep of soil and it may be natural when soils are compacted under the weight of the layers located above as a result of natural accumulation during a certain geological time.

During the construction work to improve bearing capacity in the bases, form the conditions for the formation of artificial consolidation of the mules, which in the term proceeds much faster than natural (but significantly slower than in laboratory tests of samples).

Consolidation of muds as multiphase soils (Fig. 1) is related to the interaction of solid 1, liquid 2 and gaseous 3 soil constituents, changes in their ratio in space and time as a result of the course of a complex of physicochemical processes (thixotropic hardening, aging, etc.).

In the general case [19-22], the consolidation process can consist of four phases (Figure 2):

– phase *AB* – prefiltration consolidation, during which the pressure of the fluid begins to exceed the initial value. The phase ends with the formation of closed pores from liquid and gas (see Fig. 1);

– phase *BC* – primary filtration consolidation, which is carried out by pushing out the pores of liquid and gas from the deformed volumes (Fig. 3);

– phase *CD* – secondary filtration consolidation, which is carried out by squeezing the bound fluid;

– phase *DE* – consolidation of volumetric creep, which is carried out due to plastic deformations of the soil skeleton (silt).

a)

b)

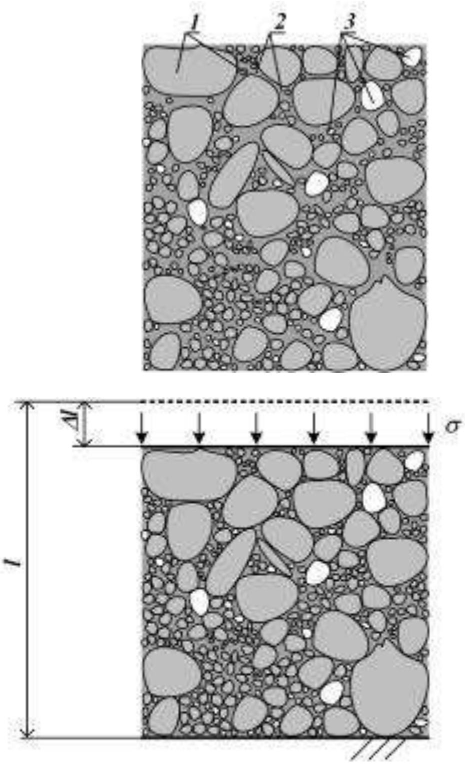

Figure 1 – The scheme of consolidation of mules at the micro level:
a – the components of mudlike soils:
1 – solid; 2 – liquid; 3 – gaseous;
b – the consolidation scheme at the micro level

When conducting laboratory tests of samples of moles for determining bearing capacity, the process of consolidation of soils is sometimes characterized by indicators of only two phases [22]:

– primary filtering consolidation – coefficient of filtration (primary) consolidation c_v;

– secondary consolidation of creep – coefficient of bulk (secondary) consolidation c_a.

Taking into account the simultaneous phases of the process, the description of the consolidation process is sufficiently fully disclosed by the application of two coefficients of consolidation – c_v and c_a.

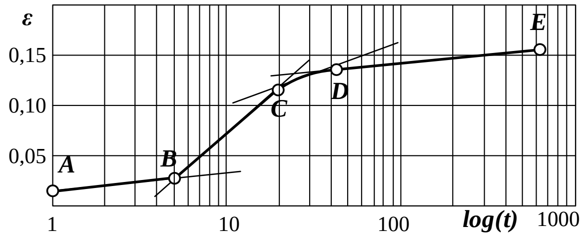

Figure 2 – Phases of consolidation of muds

For presentation of the results of the analysis of the process of consolidation (see Fig. 3), as laboratory samples of mules and structures of bases containing layers of mules, two forms of diagrams are used:

– diagrams of the deformation of the sample (design) in time

$$\Delta h = f_1(t), \tag{1}$$

where Δh – deformation of the sample (design), m;
t – time of observation (during laboratory tests time is measured in minutes, with observation of structures – in days);

– diagrams of change in height (length) of a specimen (construction) in time

$$h = f_2(t), \tag{2}$$

where h – the current value of the height of the sample (design), m.

Taking into account that during initial laboratory tests, the initial height of the specimen H is fixed, and the deformation of the specimen by definition

$$\Delta h = H - h,$$

then the diagrams (1) and (2) are equivalent.

In case of need the deformations are given in relative form, and the axis of time, taking into account the long-term processes, is laid out on a logarithmic scale, that is, in the form of diagrams on a semi-logarithmic scale. So, in Fig. 2 diagram of consolidation phases of mules is given on a semi-logarithmic scale.

2. Models of consolidation process of soil bases containing layers of mules

According to the characteristics of the process of consolidation of muds when approximating the results of monitoring of the settling of structures located on moldy soils, approximating dependencies must meet the following requirements:

– have a nonlinear appearance;

–wear fading character - the character of the asymptotic approximation to the horizontal line;

– have a significant change in the intensity of fading over time.

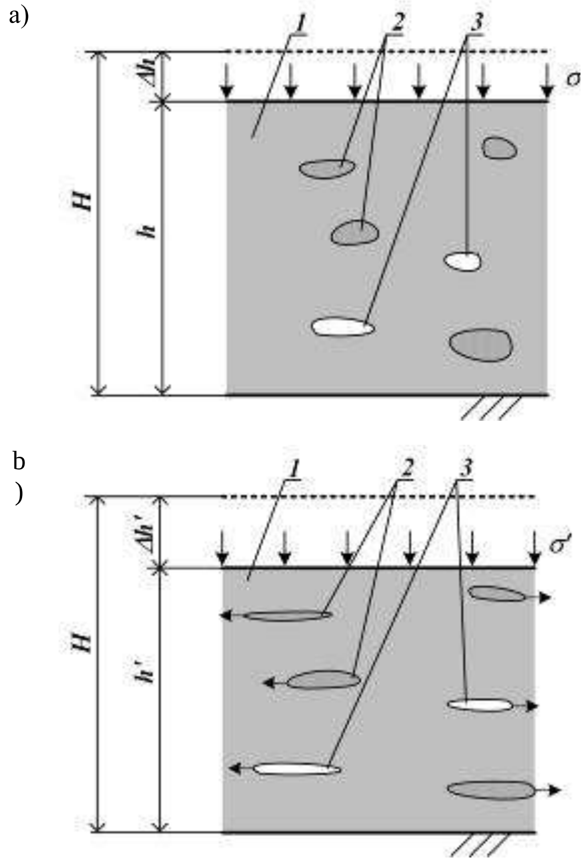

Figure 3 – The scheme of consolidation of mules at the macro level:
a – prefiltration consolidation;
b – filtration consolidation;
1 – a silt of a homogeneous structure;
2 – pores of liquid; 3 – gas pores

The specified requirements correspond to the form of a set of exponents with a constant component

$$h(t) = \sum_{i=1}^{n} A_i e^{-B_i t} + h_0 , \tag{3}$$

where they are constant A_i, B_i and h_0 determined when approaching experimental data sets.

The connection of the mathematical dependence (3) to the physical content consists in the distribution of the consolidation of the mules into processes that proceed rather quickly, intensively (components $\sum_{i=1}^{n} A_i e^{-B_i t}$), and on long-term processes ($A_i e^{-B_i t}$) so, that they coincide in time with the limit of deformation of a concrete basis, which contains layers of mules (component).

The number of exponents in the approximating dependence is established on the basis of experimental data in accordance with the nature of the processes occurring. In order to predict the processes of sedimentation of organo-mineral soils in the result of consolidation, it is expedient to adopt $n = 2$, that is, to take for the approximation of the results of the observation curve

$$h(t) = ae^{-bt} + ce^{-dt} + h_0. \tag{4}$$

At the same time, for processes that proceed fairly quickly, let us consider the phases of consolidation:
– the phase *AB* – prefiltration consolidation;
– the phase *BC* – the primary filtration consolidation.
The remaining phases:
– phase *CD* – secondary filtration consolidation;
– phase *DE* – consolidation of volumetric creep,
can be attributed to long-lasting consolidation processes of silt.

To the disadvantages of using the dependence (4) to approximate the consolidation processes of the mules should be attributed:
– a significant number of unknown approximation parameters – five constant a, b, c, d and h_0;
– transcendence of function (4).

The indicated disadvantages considerably complicate the process of calculation of approximation parameters and do not allow to obtain the result in an explicit form.

The calculation of the approximation parameters a, b, c, d and h_0 can be done by direct iterative calculations by finding the minimum irreconcilability – the method of least squares.

3. Algorithm for calculation of terms of development of settling elements of structures

The procedure for calculating the terms for the deposition of the elements of buildings will be considered on the example of the construction of a fencing container terminal at the Quarantine Mole of the Odesa branch of the State Enterprise «AMPU» (Fig. 4) in the unfavorable natural conditions of the site (the presence of a large thickness of silts) and the requirements of observance of measures to reduce sediment and compensate for their impact for construction

3.1 Situational plan for the location of the object. The indigenous bed of the district of the port facilities is represented by Tertiary meiotic and Upper Sarmatian deposits, composed of gray and red varieties of clay with layers and sand lenses.

At the base of the enclosing wavy container terminal there is light semi-solid clay 7 with layers of sand 8, 9, loams, and from the surface of the natural deposits are covered with different composition of the bulk soils of alluvial-sea genesis 6, and in its area – a complex of estuarine or alluvial-marine sediments. Seismicity of the construction area is 7 points. The category of grounds of the site for seismic properties is III. Gigantic massifs are located on stone pouring 3 from layers of crushed stone with a thickness of 40-70 mm and a stone with a mass of 15-100 kg and a layer of sand 5. To increase the strength of the structure reinforced by layers of geogrid 2 and nonwoven geotextile material 4.

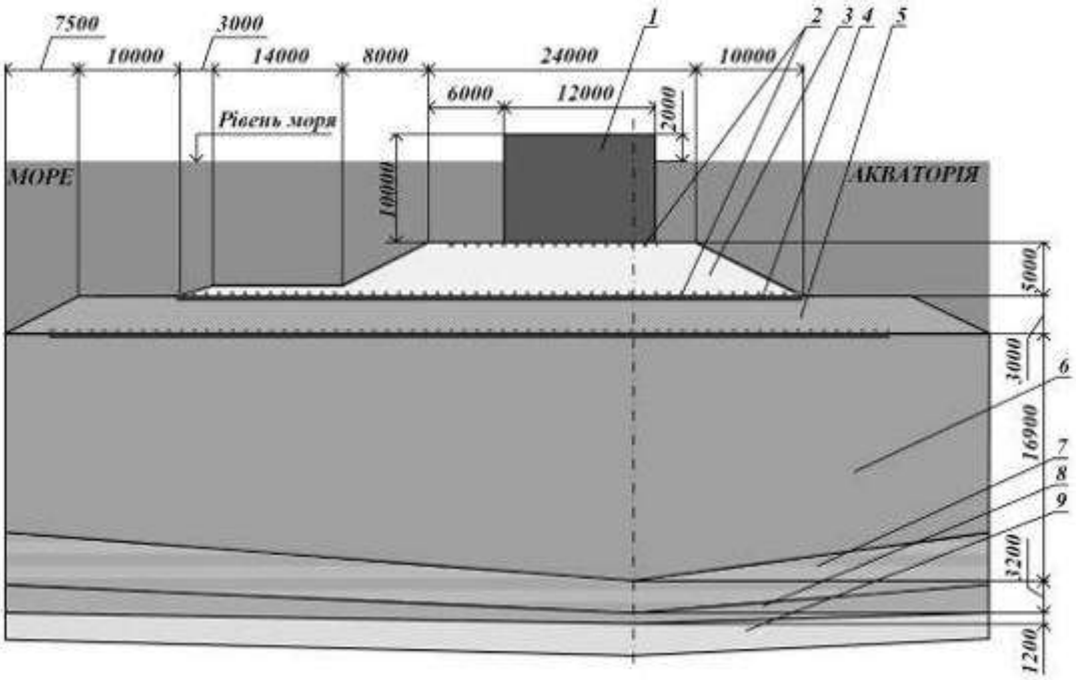

Figure 4 – Scheme of intersection of the waveguide in the zone of the massive-giant No. 3:
1 – massive giant; 2 – geogrid; 3 – stone mound; 4 – non-woven geotextile material;
5 – sand; 6 – gray silt; 7 – clay is light, fluid-plastic; 8 – dust of sand grain density;
9 – sand of medium size, medium density

3.2. Observation results of precipitation. After the completion of the first stage of construction, the control over the settling of massive-giants was carried out. Bottom line points are installed on each of the four corners of the giant array. The reference point number consists of a giant massive number and a corner number that is set from the right lower corner counterclockwise in accordance with the top view of the giant array from the container terminal's water area. Measurement of the height of the reference points was carried out by the tacheometer from the platform of the container terminal with bringing to the sea level.

Diagrams of subsidence of reference points 31 and 32 of the giant massif No. 3 relative to the sea level during the observation period are shown in Fig. 5 in normal $h_{ij} = f(t)$ (Fig. 5a) and semi-logarithmic $h_{ij} = f(\log(t))$ (Fig. 5b) scales. As a unit, the measured time we assign one day – «day». As follows from the charts in the semilogarithmic scale, the fixed subsidence of the reference points in time is sequentially grouped with respect to the set of straight lines 1, 2 and 3. Since the straight lines on the semilogarithmic diagram correspond to the exponential curves on the charts with normal scales, this provides a basis for the approach of the sedimentation process to dependences in the form aggregate of exponents.

3.3 Dependency choice for approximation. The choice of dependence for approximation is carried out on the basis of the analysis of sedimentation diagrams constructed on a semi-logarithmic scale by allocating sections with a homogeneous character of subsidence in the order, reverse to observation. So, for an example, we consider the dependence (4) as an approximating one. The areas of stable precipitation 3 (see Fig. 5b) will be approximated by the component ae^{-bt}, and the areas of the initial (intense) precipitation 1 and 2 – by the component ce^{-dt}. We assign the length of the vectors of intensive precipitation to M = 17, the vectors of stable precipitation N = 6 at the total length of the vectors in the 23 observations. In case of need for a more detailed description of the initial settling stage, we may consider sections 1 and 2 separately by adopting n = 3 and introducing an additional component le^{-mt}. However, this would require an increase in the measurement of sediment during observation so that the number of points in each plot to be approximated is not less than 4-5 (at areas 1 only three measurements are available).

3.4 Approximation of the stable subsidence area. We perform the approximation of the vector of the stable subsidence of plot 3 (see Fig. 5a) with the length N of the approximation dependence

$$hst(t) = ae^{-bt} + h_0, . \qquad (5)$$

According to the input data, we assign:
– vector of time $x_i = t_{i+N}$;
– vector of settlements $y_i = h_{i+N}$.

As a criterion for completing the approximation process, we assign the smallest sum of the squares of the difference between the values of the approximating dependence and the measurement data $y_i = h_{i+N}$.

Below is the text of the program for calculating the approximation coefficients of a direct iterative passage in the Matlab programming language:

```
Nx = length(x);
A = 1.0;
H = 0.10 : 0.00005 : 0.20;
Nh = length(H)
B = 0.0005 : 0.00000005 : 0.0006;
Nr = length(B)
KM = 20.0;
h0 = 0.0;
b = 0.0;
for ih = 1 : Nh
 for ir = 1 : Nr
  i = 1 : Nx;
   fs(i) = A*exp(-B(ir)*x(i))+H(ih);
    KK = 0;
     for i = 1 : Nx
      KK = KK + (y(i) - fs(i))^2;
     end
      K(ih,ir) = KK;
       if (K(ih,ir) < KM)
        a = A;
        b = B(ir);
        h0 = H(ih);
        Km = K(ih,ir);
       end
 end
end
```

As a result of the execution of the program, the solution is obtained in the form of values a, b and h_0 at which the minimum value is achieved. The results of calculations for reference points 31 and 32 are shown in Table 1.

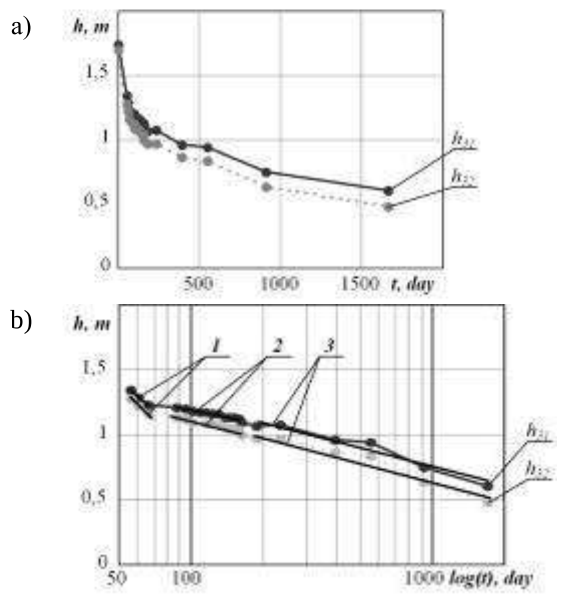

Figure 5 – Diagram of subsidence of reference points of massive-giant No.3:
a – normal scale; b – semilogarithmic scale;
h_{31} – reference point 31; h_{32} – the reference point 32.

3.5 Determination of the vector of intense subsidence of reference points. The procedure for determining the vector of intense settling is carried out for experimental data in the reverse order of time «from stable to intense». According to Dependence (4), the vector of intensive subsidence of reference points in length M can be calculated as

$$hin_i = h_i - hst(t) = h_i - ae^{-bt} + h_0, \qquad (6)$$

Since the approximation of the intensive deposition vectors will be exponential-dependent without a constant component

$$hin(t) = ce^{-dt}, \qquad (7)$$

then for the calculation of approximation coefficients it is expedient to apply standard experimental data processing programs, for example, Curve Fitting Tools Matlab software package. The results of the calculations of the coefficients c, d and the sum of the squares of the difference Km for the reference points 31 and 32 are shown in Table 2.

3.6 Diagram and forecast of subsidence of reference points. The result of the procedures performed are dependencies (4) whose approximation coefficients for the example under consideration are given in Table 1 and Table 2. Comparison of experimental data with approximations of reference points of massive-giant No. 3 is shown in Fig. 6a.

Table 1 – Parameters of the approaching plot of stable settling of reference points 31 and 32 of the massive-giant No. 3

The reference point	a, m	b, 1/day	h_0, m	Km, m^2
31	1.0	$0.5308 \cdot 10^{-3}$	0.1722	$3.893 \cdot 10^{-3}$
32	1.0	$0.5899 \cdot 10^{-3}$	0.0862	$2.659 \cdot 10^{-3}$

Table 2 – Parameters of approaching the intensive settling area of reference points 31 and 32 of the massive-giant No. 3

The reference point	c, m	d, 1/day	Km, m^2
31	0.5640	0.02131	0.004420
32	0.6089	0.02019	0.004737

Average-quadratic deviation of extrapolation dependencies with respect to experimental data calculated as

$$del = \sqrt{\frac{\sum_{i=1}^{2} Km_i}{N+M}} \qquad (8)$$

0.019 m and 0.021 m for reference points 31 and 32, respectively. The obtained mean-square deviation values fully correspond to the needs of practical application and indicate that the process of subsidence of the massif-giant can be approximated by the dependence in the form of a set of exponents with a constant component.

After substitution $T = 365\ t$ of dependence (4) can be used to predict the subsidence of reference points by extrapolation (Fig. 6b). As a criterion for completing the process, it is advisable to take a moment of stabilization of the values of sediment, for example, taking into account the deviations received (8) the stabilization of the third sign. Thus, it has been established that consolidation processes of mud-like grounds of the waveguide at the location and deposition of the massive-giant No. 3 for reference points 31 and 32 will end, respectively, for 35 years and 38 years from the commencement of operation (from 05/21/2014). The height of the location of the reference points 31 and 32 will then be 0.173 m and 0.088 m respectively above sea level.

a)

b)

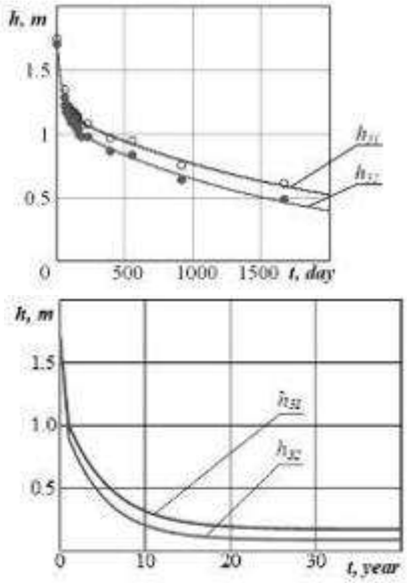

Figure 6 – The forecast of subsidence of the reference points of the massive-giant No. 3:
a – comparison of experimental data with approximations; b – extrapolation dependencies;
h_{31} – reference point 31; h_{32} – the reference point 32.

3.7 Speed of subsidence of reference points. Taking into account that dependences (3) link the subsidence of construction points with time, on their basis, by differentiation, equations can be constructed to calculate the rate of subsidence of points as

$$vh(t)=\frac{d\left(\sum_{i=1}^{n}A_i e^{-B_i t}+h_0\right)}{dt}=-\sum_{i=1}^{n}A_i B_i e^{-B_i t}, \quad (9)$$

or for a partial case of dependence (3) as

$$vh(t)=-\left(a\cdot b\cdot e^{-bt}-c\cdot d\cdot e^{-dt}\right). \quad (10)$$

Diagrams of velocity of subsidence of reference points of massive-giant № 3 are shown in Fig. 7

Numerical values of the rate of subsidence of reference points when connected with the thickness of the layers of sludge and loading on the soil basis can be applied during the observation of the state of the building as a control for assessing the nature of the development of sedimentation.

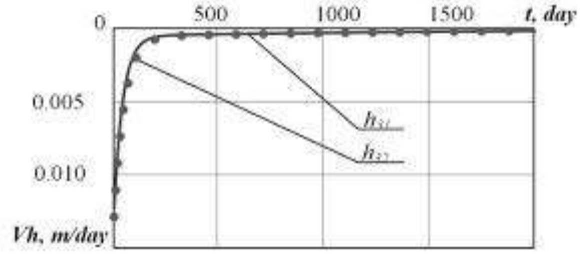

Figure 7 – The rate of subsidence of the reference points of the massive giant No. 3:
h_{31} – reference point 31; h_{32} – the reference point 32.

Conclusions

1. The development in time of processes of consolidation of muds, silty and organo-mineral soils and settling of structures constructed on bases with such layers, is exponential in nature and can be approximated by the dependence in the form of a set of exponentials with a constant component.

2. A method for predicting the size and terms of settling of structures on soils with mud layers, based on the results of field observations, has been developed. As an approximative it is suggested to apply a set of exponential dependencies with a constant component. The algorithm and software for calculation of enveloping to experimental data by a step-by-step approach are developed. An example of the calculation of the forecast of the size and terms of settling of the enclosing wavy container terminal on the Quarantine Mole of the Odesa branch of the State Enterprise «AMPU» is given.

3. Direct iterative calculations by finding the minimum non-relation by the method of least squares have established the parameters of dual exponential dependence with the constant for forecasting the subsidence of reference points of the massif-giant No. 3. It was established that the processes of consolidation of the mold-like soils of the basis of the waveguide at the location and deposition of the massive giant No. 3 for reference points 31 and 32 will be completed, respectively, for 35 years and 38 years from the beginning of operation (from 21.05.2014). The height of the location of the reference points 31 and 32 will then be 0.173 m and 0.088 m respectively above sea level.

4. The proposed methodology for forecasting the settling of bases with multidimensional soils is recommended for assessing the effect of consolidation of muds on the remains of structural elements during construction and during the operation of structures.

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SUBSTANTIATION OF SCHEMATIC AND STRUCTURAL SOLUTIONS OF THE MAIN ELEMENTS OF BIOGAS PLANT FOR THE DISPOSAL OF FALLEN LEAVES

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The peculiarities of biogas technology application in the falling leaves utilization plants are analyzed and the application of soil cement is proposed to create the basic elements of such plants. A new schematic and constructive solution of biogas fermenter (bioreactor) with application of soil-cement technology with a blending method for efficient functioning of technological chain of biogas fermentation of fallen leaves and other plant residues for energy supply systems has been developed. The possibility of reliable isolation of the space of the bioreactor to the excavation of the pit from the surrounding array with soil cement screen is substantiated. The corrosion resistance of the soil cement waterproof screen has been proved.

Keywords: biogas plant for the disposal of fallen leaves, pit, soil cement, waterproofing, aggressive medium, strength, coefficient of chemical resistance, soil cement waterproof screen.

ОБҐРУНТУВАННЯ СХЕМНО-КОНСТРУКТИВНИХ РІШЕНЬ ОСНОВНИХ ЕЛЕМЕНТІВ БІОГАЗОВОЇ УСТАНОВКИ УТИЛІЗАЦІЇ ОПАЛОГО ЛИСТЯ

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Проаналізовано особливості застосування біогазової технології в установках утилізації опалого листя та інших рослинних решток і запропоновано застосування ґрунтоцементу для створення основних елементів таких установок. Розроблено нове схемно-конструктивне рішення біогазового ферментатора (біореактора) опалого листя та інших рослинних решток із застосуванням ґрунтоцементної технології бурозмішувальним методом. Обґрунтовано можливість надійного ізолювання простору біореактора від навколишнього масиву ґрунтоцементним екраном до відкопування котловану. За наявності близького за глибиною водотривкого шару ґрунту екран занурюють у нього, а за його відсутності дно котловану ізолюють пошаровим укладанням литого ґрунтоцементу чи січними ґрунтоцементними елементами. Наведено результати експериментальних досліджень корозійної стійкості та міцності ґрунтоцементного водонепроникного екрану для умов використання лужних та кислотних модельних розчинів з концентрацією речовин, що можуть бути наявними в технологічному обладнанні біогазової установки. Установлено, що коефіцієнт хімічної стійкості ґрунтоцементних зразків поступово знижується при збільшенні періоду їх витримки в модельних розчинах, але він залишається в межах, що характеризують ґрунтоцемент як хімічно високостійкий. На основі результатів проведених досліджень відкривається практична можливість розроблення методів проектування та зведення основних елементів біогазової установки утилізації опалого листя із застосуванням ґрунтоцементної технології.

Ключові слова: біогазова установка утилізації опалого листя, котлован, ґрунтоцемент, водонепроникність, агресивне середовище, міцність, коефіцієнт хімічної стійкості, ґрунтоцементний водонепроникний екран.

Introduction

Burning of fallen leaves is still widespread in Ukraine in autumn and in spring – last year's grass and other plant residues, although this is prohibited by law [1]. Combustion of 1 ton of plant residues releases about 9 kg of smoke microparticles into the air [2], which contain dust, nitrogen oxides, carbon monoxide, heavy metals, dioxins – a substance that is dangerous for humans. The burning of leaves, dry grass and other plant residues, in addition to harming human health, leads to the destruction of the soil cover, smoke from fires in foggy days forms smog and hangs in the air for a long time, and much more electricity is consumed for settlements, etc.

The disposal of fallen leaves is possible by including them in the composition of composite pellets or briquettes, with their subsequent combustion or gasification in power plants for the production of heat and electricity [3], but then it is not possible to obtain fertilizers. Simply dispose of leaves and plant residues by composting. The effectiveness of aerobic composting is proven when air is added to the processing and after fermentation receive highly effective organic fertilizers. But composting does not make it possible to use this waste resource for energy purposes. More rational is the utilization of fallen leaves and plant residues by biogas technology, which allows to achieve a synergistic effect by combining the utilization and production of biogas in the same equipment – a biogas fermenter (bioreactor), in which the process of anaerobic (without oxygen access). The operation of such bioreactors makes it difficult to provide basic raw materials only for short periods in the fall and spring, but energy supply systems, such as buildings and structures that use biogas, must operate for periods determined by the daily and monthly load schedules. This necessitates the consideration of the bioreactor as a structure that will function effectively in the technological chain «accumulator of fallen leaves and other plant residues – biogas fermenter – in the substrate accumulator», etc.

Review of the research sources and publications

Currently, in many countries, including Ukraine, biomass is the most commonly used waste of agro-industrial and forestry facilities in biogas plants. Up to 60 varieties of biogas technologies have been developed [4]. Original technologies of anaerobic fermentation of wastes of agricultural and livestock complexes and corresponding circuit-design solutions of bioreactors for their implementation are known [5]. To maintain the optimum temperature in the bioreactor, some of the biogas produced is used, but heat pumps and renewable energy sources may also be used [6].

When developing bioreactors for the fermentation of the abovementioned wastes, it is usually considered possible to dispense with the system of accumulation and storage of raw materials in large volumes to ensure the continuous operation of the biogas fermenter. As noted above, fallen leaves, dry grass and other plant residues are accumulated over short periods in the fall and spring, which necessitates the need to address the effective work of the biorector in the technological chain «fallen leaves and other plant residues – biogas fermenter – substrate accumulator».

The decomposition of the organic constituent of fallen leaves is caused by the activity of certain bacteria. The main physicochemical and structural factors affecting the process of anaerobic fermentation are: the fractional composition of the substrate, its moisture and viscosity; alkalinity, temperature and pressure in the bioreactor; intensity of mixing; fermentation time and stability of the fermentation process; structure of the fermenter tank [4, 5]. According to the technology of preparation and fermentation of raw materials distinguish liquid-phase technology (humidity of fermented organic mass – more than 85%) and solid-phase (humidity of organic mass – less than 85%).

It is advisable to use solid-phase fermentation for biogas fermentation of fallen leaves and other plant residues in order to further use biogas in energy supply systems. A simplified scheme of a solid-phase biogas plant is shown in Fig. 1 [5].

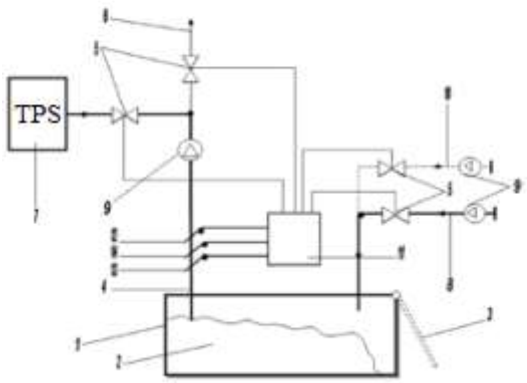


Figure 1 – Simplified scheme of solid-phase biogas plant:
1 – fermenter (bioreactor); 2 – biomass;
3 – loading-unloading hole; 4 – biogas output; 5 – valve;
6 – gas pipeline; 7 – power unit (TPS , CP);
8 – line of the exhaust gas of the power unit; 9 – fan;
10 – fresh air line; 11 – remote control; 12 – methane sensor
13 – carbon dioxide concentration sensor;
14 – sensor for determining the volume flow of biogas

The plant works like this. Raw material (such as biological waste, manure, sludge, fats or green mass) is placed in a hermetically sealed fermenter 1 and is usually heated and stirred. However, as a result of anaerobic processes, biogas and a digested substrate appear. Biogas is sent to power unit 7 for combined production of electricity and heat in mini-TPS units and cogeneration plant (CP). When using a gas piston internal combustion engine as a power engine, the biogas is cleaned of hydrogen sulfide (H_2S) before being fed to the engine. The work obtained in the engine is used to drive the electric generator, and the generated electricity is supplied directly to consumers or to the general electric network. The heat dissipated during engine cooling as well as exhaust gas is used for heat supply to consumers.

Currently, the city of Kyiv is considering building a pilot biogas complex for the processing of fallen leaves, dry grass and other plant waste using a similar technology for biogas and fertilizer production (for information, up to 120.000 tonnes of waste mentioned above are generated in Kyiv) [7]. Among the main ways of intensification of the fermentation process by increasing the level of decomposition of organic mass of fallen leaves, we can create and maintain stable preset temperature conditions of fermentation and ensure the qualitative mixing of biomass, which will ensure a uniform distribution of temperature in biomass, and prevent its delamination in the reactor [5]. The analysis has shown that the main elements of the technological chain of biogas fermentation of fallen leaves and other plant residues: storage for fallen leaves; biogas fermenter (bioreactor); fertilizer accumulator (spent substrate), – it is advisable to develop with the use of schematic and structural solutions based on soil cement technology [8 – 15]. Soil cement is a mixture of soil, cement, and water from which soil cement elements are made by blending technology without soil extraction.

Definition of unsolved aspects of the problem

Thus, the advantage of biogas reactorsmainly that are mainly used nowadays, coated with steel over concrete is the durability, no need for formwork, reduced erection and installation time, but the main disadvantage of such bioreactors is their high cost. Therefore, the use of soil-cement technology for the construction of biogas reactors designed for the fermentation of fallen leaves and other plant residues will largely preserve the advantages of steel-coated reactor structures, but will reduce their cost. This generalization is made on the basis of research and development, including the author's, with the application of soil-cement technology by the blending method.

Problem statement

For the purpose of the work, it was accepted – substantiation of schematically-constructive decisions with application of soil-cement technology with a miscible method for ensuring efficient functioning of technological chain of biogas fermentation of fallen leaves and other plant residues at the power supply systems.

Basic material and results

Storage for fallen leaves and other plant residues

For temporary storage of fallen leaves and branches in Kremenchug, specially dedicated places are practiced, and fallen leaves from private sector homes can be taken away along with the total mass of solid household waste (SHW) if these leaves are packed in plastic bags [16]. In 2018, a leaf improvement facility was created in Brovary to store fallen leaves, mowing products and vegetation residues, which will be collected in city-wide territories for further composting [17]. This object is a trench dug by the excavator in the soil (Fig. 2).

Figure 2 – The object for fallen leaves storing "Lystivnyk"

Storage of leaves in objects of this type does not satisfy the conditions of preservation of its organic mass, since storage will be accompanied by the process of aerobic fermentation of organic matter, which will significantly reduce the potential of fallen leaves for biogas production in the subsequent process of anaerobic fermentation.

Therefore, the authors propose to use satisfactory conditions for storage of fallen leaves and other plant wastes constructed using soil cement technology [8 – 15]. Such construction will be different from the construction of slurry barns made of soil cement, similar to the scheme design [15], with additional equipment: roof and equipment for loading and unloading of fallen leaves.

Biogas fermenter (bioreactor)

From the analysis of design solutions of known biogas plants, the basic requirements to the typical design of the bioreactor (methane tank) are formulated: to ensure uniform mixing of the mass, in the state of fermentation, with a speed not exceeding 0.5 m/s; create a uniform temperature field in the bioreactor using a combination of different types of heat exchangers; provide a special device in the design to prevent crust formation and clogging of the discharge pipe. Cylindrical reactors are preferably used in operating biogas plants. In bioreactors of such construction with some efficiency it is possible to mix the substrate, unload the spent substrate, remove biogas and destroy the surface crust [5].

In the construction of the reactors concrete, reinforced concrete, steel sheet, fiberglass are used. The reactor vessel shall be heat-insulated and corrosion-resistant and the enclosure of the fermentation chamber shall be sealed to prevent air access. General view of the reactors of the cylindrical structure of the biogas plant Zorg Biogas [18, 19] are shown in fig. 3. Such a biogas plant reactor consists of high quality coated steel panels using elamel high temperature sintering technology. This coating is durable, resistant to chemical effects, both internal and external corrosion and dynamic action.

A patented variant of a circuit design for an anaerobic biogas reactor, the housing of which can be made from soil cement in a blending method [20], is presented in Fig. 4.

a

b

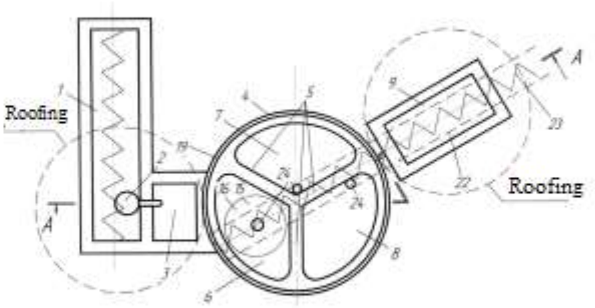
Figure 3 – Reactors of cylindrical structure of biogas plant Zorg Biogas:
a – in the process of installation;
b – during operation

An anaerobic bioreactor for the production of biogas and organic substrate contains a cylindrical tank body with a vertical baffle, a mixer, mechanisms and hydraulic locks for biomass supply and output, a biomass heater, a gas cap gas holder with control and automatic control, valves, and a gas outlet pipe. The housing is constructed of modified soil cement according to the drilling technology of the manufacture of soil cement elements. The housing is a bicameral chamber, three-chamber and has three internal vertical partitions located relative to each other at an angle of 120°, and is made integral and monolithic, with reinforcing reinforcement cages if necessary.

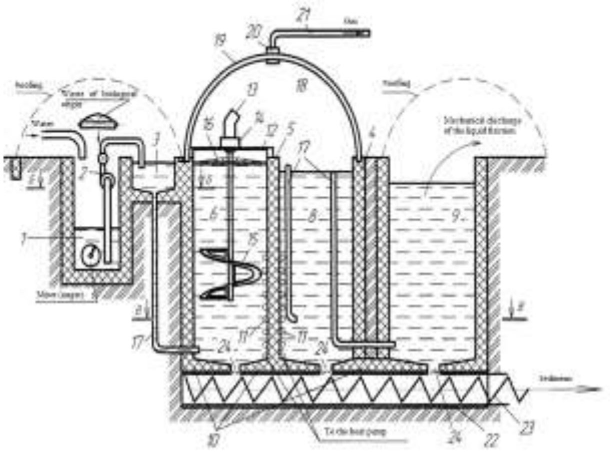
The basic elements of the plant: 1 – receiving tank for waste of biological origin; 2 – pump; 3 – capacity dispenser; 4 – a bioreactor housed in housing 4. The walls and bottoms of tanks 1 and 3 are formed from soil cement.

The anaerobic bioreactor for the production of biogas and organic substrate consists of a cylindrical housing 4 in shape (Fig. 4) with three internal vertical partitions 5, located relative to each other at an angle of 120° and forming three chambers for anaerobic fermentation, respectively 6, 7 and 8. Outside the bioreactor is a collector of fermented mass 9 created by the walls and bottom of soil cement. The shell of cylindrical shape 4, the three internal partitions 5 and the common bottom 10 (Fig. 6) are made in the «monolithic» version – as 10 integral with the soil cement. On the surface of the internal partitions 5, heat exchangers 11 are provided from a polymer pipe with a connection to a heat pump (not shown in Fig. 4). In the fermentation chamber 6 through the beam 12 is installed biomass mixer, including a hydraulic motor 13, a gearbox 14 with a vertically arranged shaft, in the lower part of which is fixed short-base single-screw auger 15, and in the upper part is fixed trapezoidal framework 15 with grid 16 for the destruction of the surface crust. Partitions 5 provide openings for by-pass pipes 17.

The accumulation of gas and its initial temporary storage takes place in the cavity of the gas holder 18, formed by the gas cap 19, which is tightly and hermetically installed in the grooves of the soil cement walls of the housing 4. The top of the cap 19, which is made of dark-colored material 20 to further heat the bioreactor from the sun's rays, has controls and automatic controls 20 and a gas outlet 21.



a



b

Figure 4 – Bioreactor plant installation with loading pump unit and bioreactor:
a – top view; b – cross-section;
1 – receiving capacity; 2 – pump; 3 – capacity dispenser; 4 – the housing of the bioreactor;
5 – partitions; 6, 7, 8 – fermentation chambers;
9 – the collection of the mixed mass; 10 – bottom;
11 – heat exchangers; 12 – mixer traverse;
13 – electric motor; 14 – gearbox; 15 – single screw;
16 – a framework for the destruction of the surface crust; 17 – openings for by-pass pipes;
18 – gas storage; 19 – gas cap holder;
20 – controls and automatic controls;
21 – outlet gas pipe; 22 is a sludge discharge pipe;
23 – long-screw auger

In the lower part of the bioreactor, a reinforced concrete pipe 22 is provided with a long-base screw 23 installed in it, by means of bearing support, to withdraw sediment (substrate).

In this case, the bottom surface 10 of the body 4 of the bioreactor in the fermentation chambers 6, 7, 8 and 25 of the fermented mass collector 9 has internal slopes to the openings with installed water locks 24, through which, periodically and in a certain sequence, the precipitate (substrate) enters the internal the cavity of the pipe 22 with the screw 23.

To create a holistic and sealed housing structure with partitions and a bottom with a diameter of a cement element of 0.5 m, they are arranged along the axis so that the distance between 30 centers of neighboring elements is 0.4 m.

Storage for substrate and composting

In the bioreactor, in the process of anaerobic fermentation, not all the mass of fallen leaves will be fermented. Therefore, to obtain high-quality organic fertilizers, it is necessary to provide not only the storage of the solid mass of the substrate separated after the bioreactor, but also its composting by the use of aerobic fermentation technology.

The simplest solution for composting [5] is to store the spent fallen leaf substrate in an open area when fencing or in a compost pit.

But in these cases, it is almost impossible to provide the conditions necessary to obtain a high quality compost, because: 1) the material must be stirred after 3 – 6 months or 1 – 4 times for the entire cycle to ensure sufficient oxygen access to all layers; 2) the humidity of the material should be kept at the level of 50 – 60% (composted material to the touch should be as «well squeezed towel»). Such conditions can be ensured in the structures of the substrate accumulator and composter, made similarly to the slurry barn for toxic drilling waste and oil and gas wells operation [15].

Such substrate storage and composting structures consist of a ditch and waterproofing, which is made of soil cement (soil cement elements), made by mixing technology without soil extraction, and additional equipment (pump for supplying water, loading and mixing devices).

Study of soil cement parameters as bioreactor structures material

Chemical constituents of substances formed during anaerobic fermentation in a bioreactor, in contact with soil cement, cause II and III types of soil cement corrosion [8]. The second type of corrosion are the processes that develop under the action of water, which contains substances that react with cement. The reaction products formed therefrom are carried out by water or released at the reaction site in the form of amorphous masses which do not have astringent properties. The second type is, for example, the corrosion processes associated with the action on the concrete of various acids and salts. The third type includes the combined processes of corrosion caused by exchange reactions with components of cement stone. The products of such reactions crystallize in pores and capillaries and cause the destruction of cement stone. The processes of corrosion caused by the deposition in the pores of stone salts, which are released from the evaporating solutions and saturate the soil cement, are included to the same kind. Usually, several aggressive factors are affected by soil cement structures at the same time, but one of them is the main process that causes the corrosion of the second kind [8].

The work [8] presents the results of our experimental studies on the determination of chemical corrosion resistance of soil cement for alkaline and acidic model solutions with concentrations of substances of III and IV environmental hazards, which may be present in the technological equipment of the biogas plant.

The corrosion resistance of building materials is their ability to resist the processes of destruction occurring in materials when exposed to external aggressive factors [8].

The resistance of soil cement to the influence of the corrosive environment is estimated by the coefficient of chemical resistance K_r – is the ratio of the compressive strength of specimens that have been exposed to such an environment R_a,, to the compression strength of the control specimens stored in the water R_w

$$K_r = R_a / R_w \,. \tag{1}$$

By the value of K_r distinguishes: materials of high resistance ($K_r > 0.8$), resistant ($K_c = 0.5 – 0.8$), relatively resistant materials ($K_c = 0.3 – 0.5$) and non-resistant materials ($K_c < 0.3$).

To test the soil cement for chemical resistance, laboratory tests were carried out for which cylindrical specimens were made of height h =15 cm and d = 15 cm in diameter, consisting of soil (sandy clay), Portland cement of the M400 brand in the amount of 20% by weight of dry soil and tap water. On the second day after molding, the samples were removed from the molds and stored until tested in water for 28 days (strength gain time). These samples were divided into five groups of 30 samples and placed in a container of chemical solutions of the most aggressive components of drilling mud, and for comparison - in a tank of water for further studies of soil cement for corrosion resistance:

– I group – tap water (H_2O);

– II group – 2.8% caustic soda solution (NaOH);

– III group – 4.0% soda ash solution (Na_2CO_3);

– IV group – 15.0% potassium chloride solution (KCI);

– V group – 50.0% an aqueous solution containing plant residues (model bioreactor medium (BioR), pH less than 7 (acidic medium [5, 6]).

Determination of strength is to measure the minimum effort that destroys specially designed test specimens under static load. The maximum effort made during the test was considered to be a destructive load. According to the tests, the compressive strength R was determined. Before the tests, the density of soil cement ρ and its humidity W were determined, according to which the density of the skeleton was calculated ρ_d. Each determination of material parameters was carried out in 6-fold repetition.

The tests resulted in an average compressive strength of soil cement specimens of a certain time and medium. The coefficient of variation υ was determined for each characteristic.

The results of the test part are summarized in Table 1. For static analysis of the strength values of soil cement spicemens, data were taken from Table. 1.

Table 1 – Averaged results of studies of the physical and mechanical characteristics of soil cement spicemens (cylinders h = 15 cm, d = 15 cm) with different holding time in chemical solutions and water for 365 days

Medium	*W*, % (*υ*)	ρ_d, t/m^3 (*υ*)	*R*, MPa (*υ*)	K_r
H_2O	37 (0.05)	1.33 (0.03)	5.52 (0.06)	1.00
NaOH	35 (0.07)	1.37 (0.05)	5.37 (0.08)	0.97
Na_2CO_3	36 (0.06)	1.38 (0.04)	5.21 (0.07)	0.94
KCl	37 (0.08)	1.56 (0.03)	4.96 (0.08)	0.90
Biomass 50%	38 (0.06)	1.35 (0.04)	5.03 (0.06)	0.91
H_2O	35 (0.04)	1.41 (0.04)	6.43 (0.08)	1.00
NaOH	34 (0.08)	1.44 (0.02)	6.19 (0.07)	0.96
Na_2CO_3	32 (0.09	1.46 (0.04)	5.95 (0.09)	0.93
KCl	30 (0.09)	1.37 (0.05)	5.77 (0.08)	0.90
Biomass 50%	34 (0.07)	1.39 (0.07)	5.60 (0.11)	0.87

υ – coefficient of variation

The graphs show a slow gradual decrease in the coefficient of chemical resistance of the soil cement, depending on the period of holding in an aggressive environment. However, in this case, the coefficient of chemical resistance of the material K_r remains within the limits characterizing soil cement as chemically highly resistant.

The results of laboratory studies allow us to predict the high efficiency and sufficient reliability of the use of soil-cement technology by blending method in biogas plants for the disposal of fallen leaves and other plant residues.

Conclusions

Thus, the presented research results indicate the solution of the scientific problem of developing schematic and structural solutions of the main elements of a biogas plant for the disposal of fallen leaves, a new design of a biogas fermenter (bioreactor), which involves the use of soil cement waterproof screen, on the basis of which can be developed methods of designing and erection of biogas plants by soil cement, boring and mixing technology. In particular, the following is established.

1. The most significant disadvantage of existing designs of bioreactors is: insufficient cost-effectiveness, due to use of concrete, reinforced concrete, steel sheet, fiberglass. Clay and film waterproofing coatings of bioreactors are also unreliable because some of their waste is not recovered for a long period and such screens lose their properties due to destruction. Clay and film waterproofing coatings of bioreactors are also unreliable because some of their waste is not recovered for a long period and such screens lose their properties due to destruction.
2. The space of the bioreactor before excavation can be reliably isolated from the surrounding array by arranging the soil cement screen of the given dimensions. If there is a waterproof layer of soil close to the depth, the screen is immersed in it. When there is no waterproofing, the bottom of the pit is insulated by layer-by-layer laying of cast soil cement or secant soil cement elements, since soil cement has a high water resistance much greater than concrete.
3. The results of experimental studies of corrosion resistance and durability of soil cement waterproof screen for conditions of alkaline and acidic model solutions use with concentration of substances that may be available in the technological equipment of biogas plant for the disposal of fallen leaves are analyzed.
4. The chemical resistance coefficient of the soil cement K_r was experimentally evaluated, which assesses the material's resistance to the influence of the chemical environment. This figure is slowly decreasing depending on the aging time of the soil cement in the aggressive environment. At the same time, it remains within $K_r > 0.8$, characterizing the soil cement as being chemically highly resistant.

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IMPROVING WATERPROOFING PROPERTIES IN THE WALL BASEMENT AREA BY INJECTION METHOD

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The article is focused on the natural experimental research of injection waterproofing impact on basement areas of existing buildings brick walls. Two types of experimental installations for waterproofing material injection have been developed. The second type turned out to be more efficient due to the fact that it could cover larger waterproofing wall area. It can possibly speed up the process of walls waterproofing and major repairs of buildings in general. According to the experimental data, after the injection waterproofing placement, the wall moisture content decreased by half due to the high quality waterproofing material and performed installation work.

Keywords: construction, moisture content, injection waterproofing, major repairs, wall basement area.

ПІДВИЩЕННЯ ГІДРОІЗОЛЯЦІЙНИХ ВЛАСТИВОСТЕЙ ПРИФУНДАМЕНТНИХ ЗОН ІН’ЄКЦІЙНИМ СПОСОБОМ

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У статті розглядаються натурні експериментальні дослідження впливу ін’єкційної гідроізоляції на прифундаментні зони цегляних стін існуючих будівель. Експериментальні дослідження з перевірки гідроізоляційних властивостей ін’єкційного матеріалу проводилися в ПолтНТУ. Для даного експерименту було обрано стіну, яка вся контактує з ґрунтом і не має гідроізоляції. Визначення вологості стіни проводилося декількома способами: 1) контактний поверхневий; 2) локальний; 3) розрахунок масового відношення вологи в матеріалі. До гідроізоляції при контактному поверхневому способі було отримано значення вологості цегляної кладки 8%, що пояснюється тим, що внутрішня поверхня стіни частково просихає та провітрюється у приміщенні, де знаходиться експериментальна стіна. При локальному способі було отримано значення вологості цегляної кладки 45%, що пояснюється тим, що відбулося розшарування цегляної суміші і води, тому поверхневий шар, практично мокрий, показав досить високий результат – це поверхнева волога. При подальшому проведенні експерименту було відібрано серію експериментальних зразків для визначення вологості за розрахунком масового відношення вологи в матеріалі. Загальна їх кількість становила 15 штук. Для вибірки визначено середнє значення вологості W=18,3%, стандарт S=1,39 й коефіцієнт варіації V=0,079. Після влаштування гідроізоляції було відібрано повторно зразки цегляної стіни та визначено значення вологості. Для вибірки визначено середнє значення вологості W=9,1%, стандарт S=0,85 й коефіцієнт варіації V=0,093. В результаті проведення експериментальних досліджень було розроблено два типи експериментальних установок для нагнітання ін’єкційної гідроізоляції. Другий тип більш ефективний, оскільки за його допомогою охоплюється більша горизонтальна гідроізоляційна площа стіни. Згідно з експериментальними даними, після влаштування ін’єкційної гідроізоляції вологість стіни зменшилася у два рази.

Ключові слова: будівництво, вологість, ін’єкційна гідроізоляція, капітальний ремонт, прифундаментна зона

Introduction

The waterproofing issue is one of the most important in building construction and reconstruction as waterproofing materials ensure building protection from the impact of the corrosive moist environment. Despite the existing and new technologies in this area, the waterproofing issue remains urgent as the right choice of techniques and waterproofing materials enables to increase their working life, decrease operational and maintenance cost, increase the possibility of using the underground parts in buildings and either eliminate or minimize renovation work. Stone buildings and structures suffer from hostile environment influence, in particular from moisture and salinization, which cause local corrosive destruction and quicken the load-bearing capacity loss of the structures unless appropriate measures in their protection are taken.

At present there is a large number of techniques and waterproofing materials which can be classified as follows:

– according to the material used there are bituminous, mineral, polymeric, and metal waterproofing types;

– according to the installation method there is coating, sealing, plaster, casting, penetrative, injection, fill, and mounted waterproofing;

– according to the appearance there is mastic, powder solution, blanket, sheet, film, polymembrane and other types of waterproofing.

Therefore, research focused on a particular type of waterproofing and its impact on the functional capacity of brick walls basement areas is relevant from the theoretical and practical point of view. The paper analyses performance and functional properties of injection waterproofing, implementation of repair techniques of such structures with the prediction of their further service life.

The research was carried out at PoltNTU and its main results were implemented as part of the state-financed applied research work "Comprehensive constructive solutions for energy efficiency of public buildings in the context of Europian integration" (state registration number 0118U001097).

Review of research sources and publications

Each type of waterproofing has always been investigated as unexpected results have attracted researchers [8]. In the papers [1, 2] horizontal waterproofing building in the old housing were studied. Various external preventive treatments of external surfaces for increasing waterproofing of the existing building constructions have been researched in the papers [3-5].

Factors like wetness, temperature and the type of building material have a big impact on the injection waterproofing functionality. The main focus of [6] is on testing penetration abilities of specially created different types of injection gels and their final efficiency in dependence on those factors.

Scientists from China [7] have carried out quite powerful research and investigated the effect of cementitious capillary crystalline waterproofing material on the water impermeability and concrete microstructure.

Definition of unsolved aspects of the problem

In light of previous and many other studies, it could be understood that modern developers encourage the use of injection waterproofing materials for major repairs. At the same time, however, this issue is not studied well from the scientific perspective. There is no specific data on full-scale experimental work on wall basement areas before and after the installation of waterproofing by the injection method.

Problem statement

The aim of the study is to conduct real experimental research on the impact of injection waterproofing on basement areas of the existing buildings brick walls.

Basic material and results.

Experimental models description. Experimental research on testing the waterproofing properties of the injection material were conducted in a classroom in the left wing of the central building (on the ground floor) of PoltNTU, which is also a training laboratory, as there is mechanical equipment for conducting power experiments. Since the university was built in the 1930s, at the time when the concept of "waterproofing" did not exist, it affected the state of bearing structures and building envelope at many locations around the central building. The classroom is located in the basement (Fig. 1), which increases the destructive effect of moisture combined with a low-efficiency heating system and insufficient insolation. This place was chosen for the experiment due to the fact that the entire wall is in contact with the soil and, without waterproofing, is completely saturated with moisture.

In parallel with this experiment, injection waterproofing was carried out at two other locations: an espacement between the windows in the PoltNTU central building external wall and the entire perimeter of the outer walls of the public building (Fig. 2).

Experimental elements are external bearing walls of approximately 1.05 meter thickness. The construction of the walls has a three-layer structure: the external and internal bearing layers are made of brick, and between them there is an indent layer, which was afterwards filled with construction debris.

Figure 1 – General view of the experimental room

Figure 2 – General view of the experimental external wall

Waterproofing material description. *AQUAMAT-F* ("АКВАМАТ-Ф") injection waterproofing was chosen for the experimental study. It is a ready-to-use mixture of water-repellent silicon compounds. When the material is injected into the wall or applied to the surface of the wall for soaking, it penetrates into the smallest pores due to its low viscosity and wettability properties. The capillary structure sealing occurs due to the interaction of the hydrophobic solution with the lime, and formation of water-insoluble chemical compounds crystals that seal pores and stop the capillary penetration of salts on the structure surface. AQUAMAT-F is used mainly to block capillary moisture. It creates a barrier at the wall base, thereby protecting it from the penetration of moisture from the soil.

Experiment conducting methodology. The experiment was carried out in accordance with the manufacturer`s technical bulletin. The initial stage was to determine the required location (distance from the angle of the wall was 0.3 m and from the floor 0.25 m). Then, a grid was formed using a measuring tape and a pencil (Fig. 3), which determined the spots for perforation: the distance between the future openings was 0.15 m in horizontal and vertical directions (7 rows with 3 holes in each).

Figure 3 – Perforated wall grid

According to the instructions, perforating was carried out at a small angle not exceeding 30° using a perforating device. The holes were perforated through the wall thickness, with 5 cm left imperforated to its outer surface. The diameter of the holes was 18 mm. The final stage of preparatory work was removing the layer of damaged plaster (about 2-3 cm thick) around the perforated places and to clean the openings from the dust that appeared during the period of work, as the holes must be clean according to the instruction.

The injection system was a construction of plastic hoses 1.5 m long, fixed on a stand 2.2 m high for the possibility of free injection in 2 rows at once, even in the highest holes (Fig. 4).

Figure 4 – The first type of experimental installation for injection waterproofing

The hoses were fixed in the holes with the help of gaskets to prevent leakage. Since the waterproofing material has a low viscosity and good wettability, placing the end of the hose above the level of the holes in the wall is enough (without sagging of the hoses) and it is possible to inject the solution even without using compression, which, in fact, was successfully completed.

The penetration of the liquid into each hole was carried out until the liquid began to appear in the pores of the wall.

In the experimental study of the other object, injection waterproofing system shown in Fig. 5 was used. The principle of its work is similar to the previous one. Hoses were inserted into the holes, which were equipped with a mechanism of dropping bottle. Each hole in the wall was insulated with construction foam. This experimental installation was more effective because it could cover a larger horizontal wall area. This can accelerate the process of waterproofing the walls and major repairs of the building in general.

After injecting all the necessary areas of the walls, every hole was filled with a suitable mortar and finished with plaster.

Figure 5 – The second type of experimental installation for injection waterproofing

Experimental research on the moisture content of the wall structure. Some amount of moisture is always present in the building envelope material due to the processes of sorption and condensation of water vapor, which affects the wall thermal qualities.

Determination of moisture content of the experimental wall construction was carried out in several ways:

1) surface contact (with the help of "VIMS 2.21" ("ВИМС 2.21") device);

2) local measurement (with the help of "VIMS 2.21" device);

3) calculation of the mass ratio of moisture in the material (according to the ДСТУ Б В.2.7-170:2008).

Device description. Universal moisture meter "VIMS 2.21" ("ВИМС 2.21") (Fig. 6) is designed for moisture content measurements of solid and fill building materials (light, porous and heavy concrete, sand-lime and ceramic brick, building sand, stone screening dust), wood (timber, chemically untreated wooden parts and products).

Figure 6 – General view of moisture meter "VIMS **2.21"**

The device can be used to measure moisture content of a wide range of solid and fill building materials with their additional calibration and the development and certification of measurement procedure ("MBB").

Application: construction industry, forestry and woodworking industry.

The device operating conditions:

– ambient temperature from 5° C to 40° C above zero;

– relative air humidity from 30% to 90%;

– single measurement time should not exceed 60 seconds.

The device consists of an electronic unit and connected sensors: volumetric-planar capacitive and probe capacitive transducers. Completeness of sensors is determined by the device use.

The volumetric-planar capacitive transducer consists of planar capacitive transducer and a nozzle, which is arranged on the external concentric electrode. According to this parameter the following types are distinguished:

– with a nozzle – volumetric capacitive transducer (volumetric sensor), designed for fill materials moisture content monitoring;

– without a nozzle – planar volumetric transducer (planar sensor), designed for solid materials moisture content monitoring.

The probe capacitive transducer (probe sensor) is used for moisture content monitoring in deep layers of solid, fill and plastic materials. The probe sensor includes a locking screw.

A nozzle for working with fill material can be used in addition to the probe sensor. The nozzle enables to compact the tested fill material and limit its control area.

The device is intended for in-process non-destructive monitoring of sand moisture content, a wide range of building materials, products, structures and buildings in the process of their creation and operation.

An even and smooth surface was prepared prior to measuring the moisture content of bricks in the experimental wall and a tight fitting of the sensors surface to the monitored wall surface was ensured.

For improving the accuracy of the results, moisture content was measured over the series of measurements at different sections of the wall surface, averaging the result by a series of measurements.

The mechanism is based on the dielcometer method of moisture content measurement, namely, the correlation dependence of the material dielectric permeability on the moisture content in it at positive temperatures.

Research results analysis. Moisture content measurement of the brickwork was carried out in two stages: prior to waterproofing and after it.

The first stage: prior to the wall waterproofing. The contact surface method showed that the moisture content of the brickwork was 8%. It is due to the fact that the inner surface of the building envelope (the wall) is partially dried and ventilated in the room where the experimental wall is located.

The local method showed that the value of the brickwork moisture content was 45%. It is due to the fact that there was a segregation of the masonry mixture and water, therefore, the surface layer, practically wet, showed quite a high result – it was the surface moisture.

In the course of the experiment, a series of experimental samples was selected to determine the moisture content by calculating the mass ratio of moisture in the material (according to the state standard "ДСТУ Б В.2.7-170:2008")

$$W_m = \frac{m_в - m_c}{m_в}\,, \tag{1}$$

where $m_в$ – mass of the material sample prior to drying, kg;

m_c – mass of the material sample after drying, kg;

All experimental samples were weighed and then placed in the drying cabinet in a special vessel. Dry materials (Fig. 7) were re-weighed, after which the moisture content of the experimental material was determined.

Figure 7 – Dried experimental samples of different walls

In this experimental study moisture content is a varying value; therefore, statistical processing of data was conducted. To obtain reliable estimates of statistical characteristics and distribution laws, the moisture content values of all experimental samples were taken. There were 15 samples in total.

The statistical processing of the samples data of the moisture content value was performed in MS Excel using the embedded statistical functions. For sampling the average humidity value was W = 18.3%, the standard S = 1.39 and the variation coefficient V = 0.079, as well as the distribution histogram (Fig. 8) were determined.

After the waterproofing installation, samples of the brick wall were re-selected and, in accordance with the state standard "DSTU Б В.2.7-170:2008", the moisture content was determined. Statistical processing was carried out. For sampling the average moisture value was determined W = 9.1%, the standard S = 0.85 and the coefficient of variation V = 0.093, as well as the distribution histogram (Fig. 9) was created.

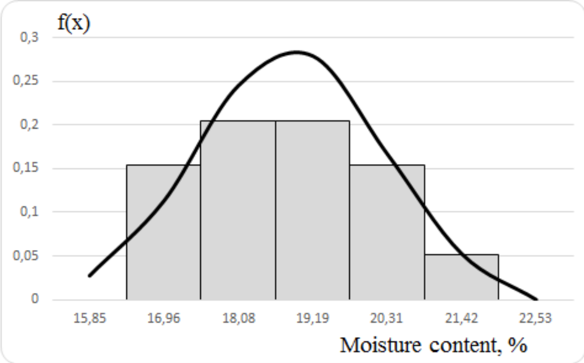

Figure 8 – Distribution histogram of the brick wall moisture distribution prior to waterproofing

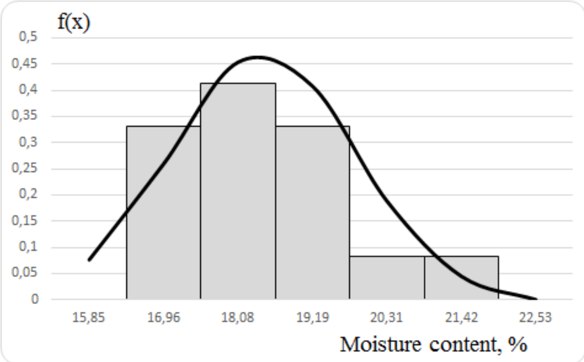

Figure 9 – Distribution histogram of the brick wall moisture distribution after waterproofing

Conclusions

In the process of experimental research, two types of experimental installations for injection waterproofing have been developed. The second type turned out to be more efficient, due to the fact that it covered wall larger horizontal waterproofing area. It could quicken the process of waterproofing the walls and major repairs of the building. According to experimental data the wall moisture content decreased by half after injection waterproofing. It is due to the high-quality waterproofing material and the implementation of installation arrangement. injection waterproofing has proved itself to be a sufficient high-quality method, but the conditions for waterproofing were near-perfect in the experimental studies, which is not always the case in actual practice.

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INNOVATIVE MATERIALS AND TECHNOLOGIES IN THE OIL AND GAS INDUSTRY

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During the exploitation of equipment and piping systems of sulfuric acid, shale, metallurgical, mining, energy and other industries, the metal of a number of structures directly contacts with sulfur-containing agents at high temperatures. This results in intense corrosion, loosening and saturation of the surface layer of metal with sulfur (elementalor in the form of various compounds, including sulfides, iron, oxides) with a concentration of up to 0.6%. Repair welding of such metal at its partial replacement is connected with the big labor costs caused by necessity of mechanical removal of a surface layer, as without this operation in weld metal by standard electrodes hot cracks, pores and notfusion are formed.

Keywords: corrosion, hot cracks, pores, slag, seam.

ІННОВАЦІЙНІ МАТЕРІАЛИ ТА ТЕХНОЛОГІЇ В НАФТОГАЗОВІЙ ПРОМИСЛОВОСТІ

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Під час експлуатації обладнання та трубних систем сірчаної кислоти сланцевої, металургійної, гірничої, енергетичної й інших галузей промисловості метал ряду конструкцій безпосередньо контактує із сірковмісними агентами при високих температурах. Це призводить до інтенсивної корозії, розпушування та насичення поверхневого шару металу сіркою (елемента у вигляді різних сполук, у тому числі сульфідів, заліза, оксидів) з концентрацією до 0,6%. Ремонт не зварювання такого металу при його частковій заміні пов'язане з великими витратами праці, зумовленими необхідністю механічного видалення поверхневого шару, оскільки без цієї операції зварювання стандартними електродами в металі утворюються гарячі тріщини, пори і нерозплавлення. Досліджено та розроблено прогресивні зварювальні електроди на основі нових металургійних та технологічних принципів забезпечення високої стійкості зварювальних та монтажних з'єднань проти утворення гарячих тріщин у металі зварного шва. Розроблені інноваційні електроди марки ANM-1 дозволяють зварювати металеві конструкції з низьковуглецевих сталей з поверхневим насиченням сіркою за наявності накипу, іржі та інших домішок. Вони вирізняються високою стійкістю до утворення гарячих тріщин, свищів і пор у шві. Електроди були успішно випробувані на багатьох підприємствах, пов'язаних з переробкою сірковмісних матеріалів (руди, сланці, нафтопродукти, природний газ тощо). При проведенні наукових досліджень було використано високоточне сучасне обладнання, зокрема, для вивчення зварювальнихі технологічних властивостей, залучена інформаційно-аналітична система моделі АНП-2; автоматичний зварювальний апарат зі штучними електродами; сучасні інверторні випрямлячі виробництва Інституту електричного зварювання НАНУ імені Е.О. Патона, а також для оцінки якості зварних швів використовував аналізатори-контролери власної конструкції. Використання електродів ANM-1 дозволяє значно підвищити ефективність ремонтних робіт за рахунок усунення трудомістких операцій видувки повітряної дуги та подальшого очищення країв монтажного з'єднання шліфувальною машиною.

Ключові слова: корозія, гарячі тріщини, пори, шлам, шов.

Introduction

Ukraine is one of the most technologically advanced countries in the world, which has its own oil and gas, oil refining, metallurgical industry, developed enterprises for energy production and energy supply. A prerequisite for the successful functioning of these major industries is the constant creation and updating of suitable pipe and electrode materials, as well as welding and assembly technologies, which provides these industries with competitiveness in the world market and satisfies internal needs, which in turn provides energy independence of Ukraine.

Review of the research sources and publications

During the operation of technological equipment and metal structures in chemical-aggressive environments, the metal directly contacts with sulfur-containing agents at high temperatures. This leads to intense corrosion, loosening and saturation of the surface layer of metal with sulfur (elemental or in the form of various compounds, including iron sulphides, oxides) with a concentration of up to 0.6% [1 – 8].

Repair by welding such metal structures in their partial replacement is associated with large labor costs, which are caused by the need for mechanical removal of the surface layer, because without such operations in the weld metal, standard welding electrodes produce hot cracks and other unacceptable defects [4 – 8].

Therefore, the existing traditional electrode materials and technologies of their use already significantly limit the advanced design and technical and technological opinion regarding the repair of equipment and pipeline networks of long-term exploitation in corrosive-aggressive environments, so to further improve them with a new need characteristics of the required properties [2 – 8].

Definition of unsolved aspects of the problem

Research and development of progressive welding electrodes on the basis of new metallurgical and technological principles of ensuring high resistance of welding and assembly joints against the formation of hot cracks in the weld metal.

Problem statement

The basic scientific idea is that the reason for the destruction of welding joints of oil and gas pipelines and other metal structures of long-term operation, whose surface layer of metal is saturated with sulfur (up to 0.1 – 0.6%), is the low resistance of weld metal in the formation of hot cracks in the process of cracking cracks are formed in the process of repair welding, which makes it impossible to exploite the repaired metal structures further, as this can lead to further destruction. Therefore, the idea of providing high fracture toughness during repair welding of long–life metal structures, the surface layer of which is saturated with sulfur, is used in the electrode coating instead of ferroalloys, in particular ferromanganese, manganese metal.

Research methodology

In conducting systematic and complex scientific researches, high-precision modern equipment was used, in particular, for the study of welding and technological properties, the information–analytical system of the ANP-2 model was involved [7]; automatic welding machine with artificial electrodes; modern inverter rectifiers manufactured by the Institute of Electric Welding of NASU named after E.O. Paton, as well as to assess the quality of welds, used analyzers-controllers of their own design. For experiments, electrodes with variable manganese content of metal in the coating (from 0 to 30%) were prepared – the rest of the coating composition is as follows (in%): ilmenite – 50; muscovite mica – 8; quartz sand – 10; cellulose – 2. Supplemented the coating composition to 100%, introducing iron powder from 0 to 30%. The binder was 25% sodium liquid glass, density 1.55 g/cm^3, with module 2.76.

Basic material and results

The doping of weld metal with manganese was carried out not through the electrode wire, but through the electrode coating, which provided less burnout in the welding process and stable doping and modification of the weld metal structure. This has provided resistance to hot cracking, since manganese metal is able to more effectively bind sulfur into a chemical-resistant compound that is insoluble in molten metal and removes it into the slag.

It is seen that the increase in the content of manganese in the coating greatly reduces the concentration of sulfur in the weld metal, in particular more than 10 – 20 times, and dramatically decreases the corrosion rate, regardless of the service life of structural steel on average 2 – 4 times.

Given the large volume of experimental material that characterizes the often uncertainty and contradiction of information regarding the effect of sulfur and manganese on the corrosion behavior of steels obtained by traditional methods, we have for the first time used an alternative method of experimental results analysis based on the use of artificial neural networks (SNM). The use of SNM allows to create qualitatively new hardware and software, which significantly extend the classes of tasks to be solved and increase the efficiency of analysis and forecasting [7].

Using a neural network simulation method, we obtained a predictive model of weld behavior performed on grade 3 low carbon steel in corrosive environment.

The results of experimental studies of the effect of metal manganese on the corrosion rate and the sulfur content of the weld metal are presented in Fig. 1 - 2.

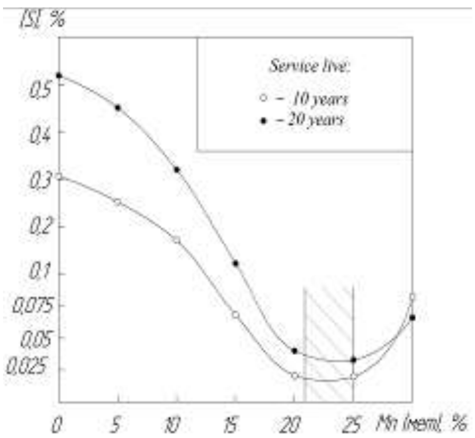
Fig. 2 shows the simultaneous joint effect of the sulfur content in the weld metal and the manganese metal in the electrode coating on the corrosion rate of the samples. At the same time, it can be argued that the content of manganese in the coating in the range of 23 – 25% significantly reduces the tendency of weld metal to corrosion. This made it possible to optimize the content of manganese metal, which is introduced into the coating in the form of powder, limiting it to a range of 22 – 25%.

For example, for steel with a lifetime of 20 years, the initial sulfur content in the surface layer of metal (up to 1 mm) was about 0.52%, then the introduction of 20 – 25% in the coating of manganese metal allowed to reduce the sulfur content in the weld metal to 0.03 – 0.045% that is, 12.6 – 17.3 times.

Using the neural network method of analysis of the obtained experimental results, a spatial model of simultaneous influence of the sulfur content in the surface layers of the metal of equipment (decomposers) and manganese metal in the electrode coating was constructed with different terms of exploitation in an aggressive environment (Fig. 2).

The influence of manganese metal on the resistance of weld metal against the formation of hot cracks. The tendency of electrodes to form hot cracks was quantitatively evaluated according to the method of MVTU named after M.V. Bauman after on the LTP-1-6M installation [8]. The criterion of the resistance of the weld metal to the formation of hot cracks during welding was the indicator A (mm/min), which characterizes the minimum value of the rate of deformation at which the formation of hot cracks can be seen. Welding was performed on alternating current in modes: I_{zv}=180A, U_d =22 – 24V, V_{zv} = 14m/h. The results of the studies are shown in Fig. 3 – 5.

a)



b)

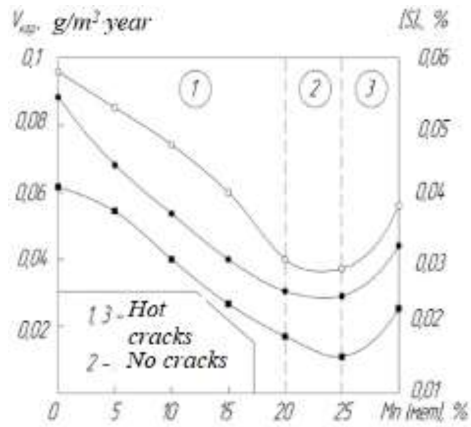
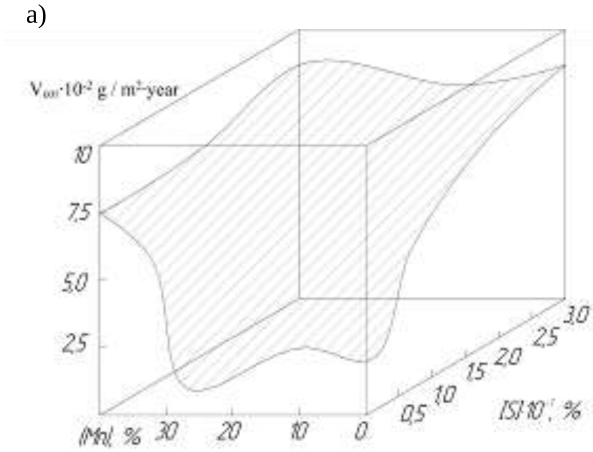


Figure 1 – Effect of manganese in the electrode coating on the sulfur content of the weld metal (a) and on the corrosion rate of the weld joints (b) of the low carbon steel hull grade VSt3sp decomposers lasted for a long time. Designation (b):

o – steel has been used for 18 years (in the surface layer up to 1 mm deep, sulfur content was ≈0.45%);
● – 14 years respectively ([S] ≈ 0.32%); ■ – 8 years respectively ([S] ≈ 0.25%)

a)



b)

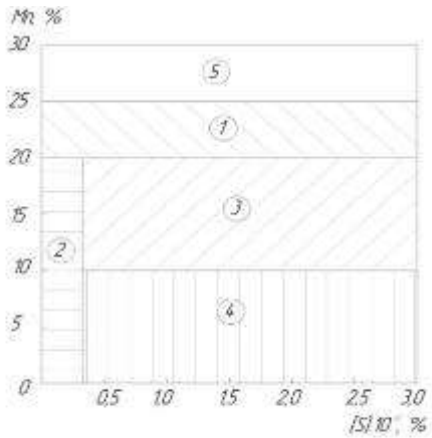


Figure 2 – Neural network analysis of corrosion rate of weld metal samples in corrosive–aggressive model environment NACE with different content of manganese metal in the electrode coating and concentration dissolved in sulfur metal (a) and predicted by SNM of stability in hot welds made on low carbon steel Vst3sp with different content in the surface layers of sulfur by electrodes ANM–1 with different content of manganese metal in the coating:

1 – areas of high resistance to hot cracks; 2 – area of variable stability; 3 – area of low stability;
4 – is an area of very low resistance; 5 – is an area of unstable stability

a)

b)

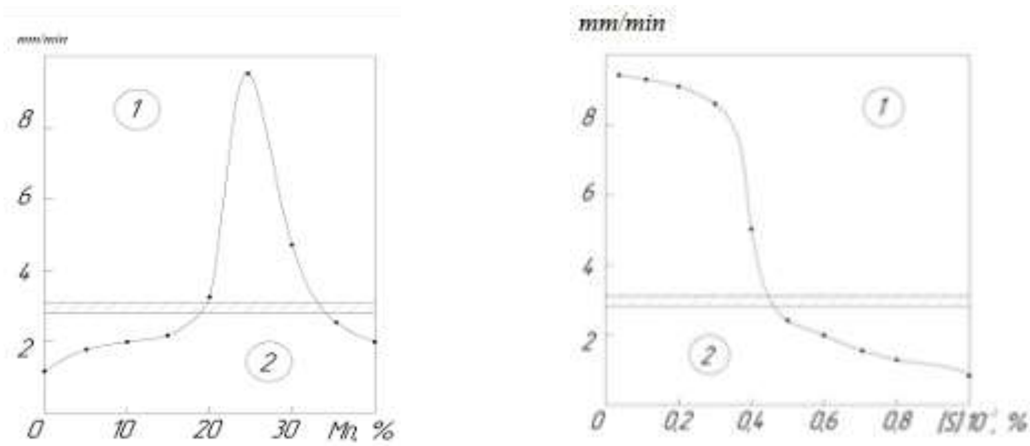


Figure 3 – Effect of the manganese content of metal (a) in the electrode coating and sulfur in the weld metal (b) on the resistance against the formation of hot cracks in the welds

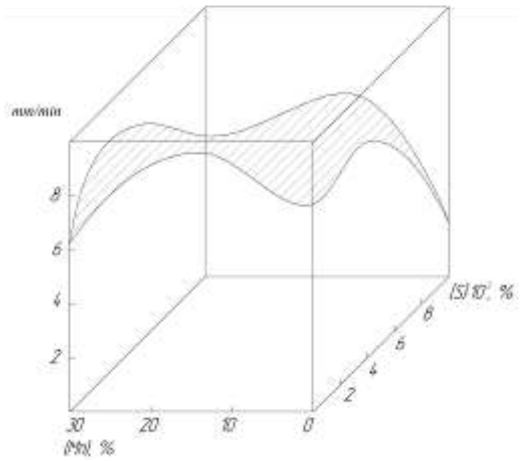


Figure 4 – Three-dimensional model of resistance to hot crack formation in welds, depending on the content of manganese metal in the electrode coating and the concentration of dissolved sulfur in the surface layers of metal long-term operation in corrosive environment

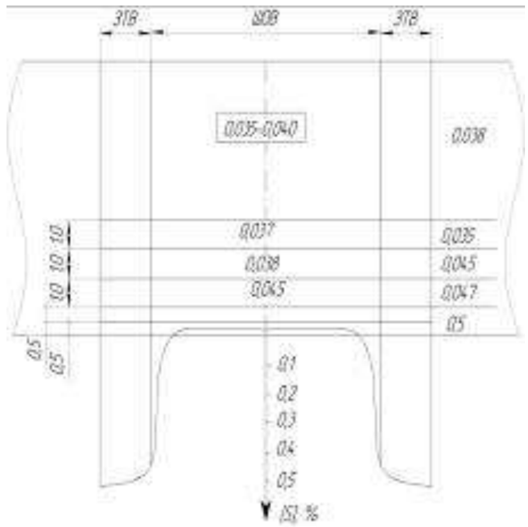


Figure 5 – Profile of welding joint made by ANM-1 electrode on low carbon steel with different sulfur content in the surface layer

Analysis of the data shows that the introduction into the electrode coating of manganese metal in the amount of 20 – 30% significantly increases (4 – 5 times) resistance to the formation of hot cracks. In the opposite way, the content in the deposited metal of sulfur influences the formation of hot cracks. Thus, when the sulfur content of 0.042% or more sharply reduces the resistance of the metal against the formation of hot cracks.

The above data are processed by mathematical modeling methods, the results of which are shown in a three-dimensional model (Fig. 4). Obviously, in spatial form, areas of high and low resistance to the formation of hot cracks during welding are visible.

Figure 5 presents a profile of a welded joint made by ilmenite electrodes of the ANM-1 brand. It is seen that the weld seam is cleared of sulfur and her content does not exceed 0.04 – 0.05%, that is, the reduction of sulfur content occurs more than 10 times, which allows guaranteed to increase the resistance of the weld metal against the formation of hot cracks in the process of welding contaminated with sulfur metal.

The influence of the content of manganese metal in the coating on the separation of the slag crust from the surface of the deposited metal. The results of measurements of the index of separability of the slag crust from the surface of single-layer welds are shown in Fig. 6.

From the data shown in Fig. 6 it can be seen that the lightest separability of the slag from the surface of the weld metal is characterized by electrodes which place in their composition metal manganese in the range of 20 – 30%.

Thus, the obtained results were taken into account in the development of new electrode coatings in order to improve welding and technological properties.

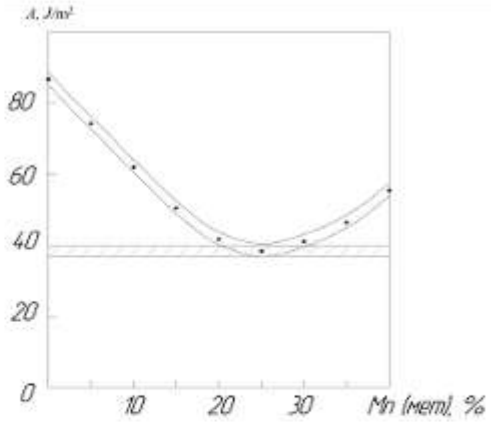


Figure 6 – Effect of metallic manganese content in the electrode coating on the index of separability of slag crust from the surface of welds

The results of quantitative assessment of the tendency of welds against the formation of hot cracks for different brands of electrodes are shown in Table 1.

Table 1 – Results of quantitative assessment of the tendency of welds against the formation of hot cracks for different brands of electrodes are shown

Electrode Brand	The main metal, thickness, mm	A, mm/min
ANM–1	VSt3sp, 16	9.82
UONI–1355		4.30
ANO–11		4.64
critical value of the indicator A – 3 – 3.5 mm/min		

Considering that the critical value of indicator A for structural steels is 3 – 35 mm/min, high resistance to the formation of hot cracks in the joints made by new electrodes (ANM-1) is noteworthy.

Table 2 shows the characteristics of the stability of the combustion of an electric arc AC and the transfer of molten metal across the electrode gap. Indicators B_z and τ_z were determined using the ANP-1 model measurement and information system «Non-stationary Process Analyzer».

Table 2 shows that the ANM-1 electrodes are characterized by high arc burning stability and small-drop transfer of electrode metal. In particular, the stability index of the developed electrodes is almost 3 times higher than the ANO-11 electrodes intended for AC welding.

The results of measurements of the stability of the combustion of the arc (V_z, $Om^{-1}{\bullet}s^{-1}$) and the duration of short circuits (τkz, ms). Electrodes with a diameter of 4 mm.

Experimental and industrial testing. The chemical composition of the weld metal (ANM-1) is typical of electrodes of the E46 type, but differs in the lower content of harmful impurities (in%): C – 0.08; Mn – 0.8 – 1.2; Si – 0.15 – 0.20; S ≤ 0.025; P ≤ 0.025.

The values of the impact toughness of the weld metal made by the ANM–1 electrodes on VSt3sp steel are given in Table 3.

Table 2 – Stability index of the developed electrode

Electrode Brand	180A	110A
	$B_з$,$Om^{-1}{\bullet}s^{-1}$	$\tau_{кз}$, ms
ANM-1	312.4	9.6
Ferex	126.1	16.7
Schwarz 3K	120.4	17.7
OK48.23	143.4	17.5
UONI-13/55	76.4	16.6
ANO-4	321.2	9.9

Table 3 – The values of the impact toughness of the weld metal made by the ANM-1 electrodes on VSt3sp steel

Electrode Brand	Brand and thickness of the steel, mm			
		+20	–20	–40
ANM-1	VSt3sp,16	100 – 113	70–83	43 – 56

Analysis of the data in Table 3 shows that the ANM–1 electrodes provide the toughness of the weld metal of low carbon steels, which meets the requirements for the type of electrodes E46 according to GOST 9457–75.

Tests for sulfide cracking electrodes were performed according to the method of the International Corrosion Association according to NACE standard TM-01-77 (90) [8]. The basic test time is 680 hours. The results are shown in Table 4, which shows that the electrodes ANM-1 are characterized by high resistance to sulfide corrosion cracking on steel VST3sp in comparison with the best foreign electrodes in particular known manufacturers of welding materials – Japan and Sweden.

The impact toughness of the weld KSV (J/cm²) at temperature, °C.

Table 4 – Electrodes ANM-1 are characterized by high resistance to sulfide corrosion cracking on steel VST3sp

Electrode Brand	Factory (firm), country	A, (J/cm²)
		Steel 3sp
AHM-1	EEZ named after E.O. Paton	6.8
OK74.78	ESAB, Sweden	5.2
Nibaz 65	Romania	4.9
461SHV1	Germany	4.4
E-B235	Vamberk, Czech Republic	3.8
E8016-C1	Kobe Steel, Japan	5.7

Figure 7 shows the comparative results of the corrosion rate measurements of welds of ilmenite electrodes grades ANM-1 and ANO-6 made on steel grade BC3sp with different sulfur content in the surface layers (metal fragments were cut from decomposers intended for alumina production, with different service life). It is seen that the weld metal of the new ANM-1 electrodes has a 2 – 4 times lower corrosion rate than the known ilmenite electrodes ANO-6, especially manifested at high concentrations of sulfur in the surface layers of metal of equipment of long-term exploitation (more than 15 years).

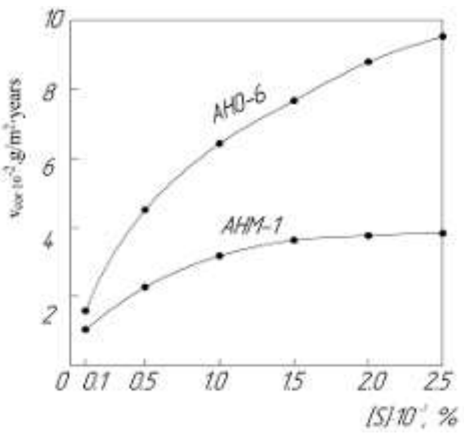


Figure 7 – Corrosion rate of welds on steels with different service life in aggressive media with new electrodes compared to ilmenite electrodes ANO-6

The new ANM-1 electrodes have been successfully passed extensive industrial testing at a number of Ukrainian enterprises and organizations, in particular: SE «Research Plant of Welding Materials Electric Welding Institute of NAS Unamed after E.O. Paton» (Kyiv); PE «PromtruboProvodKomplekt» (Boyarka); PE «METCON» (Kiev); PE «Engineering Center of Welding and Surfacing Materials» (Kiev); PC «Center of Construction and Installation Works and Operations of Buildings and Structures» of PC «Ukrza» (Mariupol); LLC «Azovmashprom» (Mariupol); JSC «Azovzagalmash» (Mariupol); RPE «Naftogazservice» (Gadjach, Poltava region) and others.

The ANM-1 electrodes were tested in a laboratory accredited by the State Standard of Ukraine (accreditation certificate No. 131 of 29.07.1994). The necessary technical documentation (technological instructions for production, specifications, etc.) has been developed for the ANM-1 electrodes.

More than 100 tons of ANM-1 electrodes were manufactured; implemented at 15 enterprises; UAH 250 million was received; economic effect; 245 new jobs were created; 10 License agreements with enterprises were signed; it is planned to create another 50 new jobs and receive 50 million UAH. economic effect.

Conclusions

During the exploitation of the equipment and pipeline systems of sulfuric acid, shale, metallurgical, mining, energy and other industries, the metal of a number of structures directly contacts with sulfur-containing agents at high temperatures. This leads to intense corrosion, loosening and saturation of the surface layer of metal with sulfur (elemental or in the form of various compounds, including iron sulphides, ox sulfides) with a concentration of up to 0.6%. Repair welding of such metal at its partial replacement is connected with the big labor costs caused by necessity of mechanical removal of a surface layer, as without this operation in weld metal by standard electrodes hot cracks, pores and not fusion are formed. Developed innovative electrodes of the ANM-1 brand allow to weld metal structures from low carbon steels with surface saturation with sulfur, in the presence of scale and rust and other impurities. They are highly resistant to the formation of hot cracks, fistulas and pores in the weld. The electrodes have been successfully tested extensively by many enterprises involved in the processing of sulfur-containing materials (ores, shales, petroleum products, natural gas, etc.). The use of ANM-1 electrodes can significantly improve the efficiency of repair work by eliminating the time-consuming operations of aft-arc gouging and subsequent cleaning of the edges of the mounting joint with a grinding machine.

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INCREASE IN FLUID EXTRACTION, WHICH IS AT THE FINAL STAGE OF DEVELOPMENT DUE TO THE IMPROVEMENT OF EQUIPMENT INTENSIFICATION

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It has been established that the fluid reserves, which are at the final stage of development, are difficult to extractable and watered. The methods of intensification are analyzed and it is established that it will be rational in this case to pump carbon dioxide into the reservoir. Its influence on a bundle of oil and water is determined. The results of the statistical processing of the change of the debit before and after injection of carbon dioxide, the influence of the volume of gas injected into the well from the seperate permeability of the reservoir, as well as the expected rate of wellbore after injection, are presented.

Keywords: buffer tanks for carbon dioxide, carbon dioxide, fluid, gas well, injection wells, productive layer, pump station, viscous oil sand, water content.

ЗБІЛЬШЕННЯ ВИДОБУТКУ ФЛЮЇДУ, ЩО ЗНАХОДИТЬСЯ НА ЗАВЕРШАЛЬНІЙ СТАДІЇ РОЗРОБКИ, ЗА РАХУНОК УДОСКОНАЛЕННЯ ОБЛАДНАННЯ ІНТЕНСИФІКАЦІЇ

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Установлено, що запаси флюїду родовища, яке знаходяться на завершальній стадії розробки, є важко видобувними й обводненими. Проаналізовано методи інтенсифікації і з'ясовано, що раціонально в такому випадку буде закачати в пласт вуглекислий газ. Визначено його вплив на в'язку нафту та на воду. Наведено результати статистичної обробки зміни дебіту до і після ін'єкції діоксиду карбону, впливу об'єму газу, що закачується у свердловину, від середньої проникності пласта, а також від очікуваного дебіту свердловини після проведення ін'єкції. Проаналізовано, що розробка газоконденсатних родовищ без підтримки пластового тиску (так званий «режим виснаження») призводить до втрат, часто значних, вуглеводневого конденсату. Для зменшення пластових втрат конденсату використовуються різні способи впливу на пласт, що передбачають нагнітання газів або води зазвичай при початкових пластових тисках. Використання цих методів вимагає великих інвестицій, що не завжди економічно виправдано. Нова технологія заснована на ефектах витіснення пластового жирного газу сухим, випаровування рідких ретроградних вуглеводнів, підтримки пластового і забійного тисків, блокування активної підошовної та законтурної води. Технологія забезпечує збереження фонду діючих свердловин, збільшення їх продуктивності та дебітів, підвищення вуглеводовіддачі пласта, підтримання сировинної бази газопереробного заводу, продовження періоду активного функціонування всієї створеної промислово-заводської інфраструктури. Для реалізації технології та досягнення її проектної ефективності необхідно створити і впровадити нову надійну систему контролю за розробкою. Ця система повинна враховувати геолого-технологічні особливості об'єкта видобутку вуглеводнів і технології його розробки, маючи на увазі геологічну будову колектора, ефективні газонасичені товщини, термобаричні умови та ін. Вона має забезпечувати безперервний контроль інтервалів приймальності при закачуванні газу та інтервалів дренування при відборі вуглеводневої суміші, оперативне визначення компонентного складу продукції на основі застосування високоточної геофізичної та хроматографічної апаратури.

Ключові слова: буферні ємності для вуглекислого газу, вуглекислий газ, рідина, газові свердловини, нагнітальні свердловини, продуктивний пласт, насосна станція, в'язкість, пісок, вміст води.

Introduction

The industrial problem of the oil industry is due to the fact that, due to the more intensive development of easily recovered oil fields, the share of hard – to – recovered reserves is constantly increasing, requiring the application of new methods of increasing oil production.

One of such complex methods is the injection of CO_2 into the formation. The physical essence of the method is the good solubility of CO_2 in formation fluids, which provides a volume expansion of oil in 1.5 – 1.7 times, the ability to mix it with oil, reducing the viscosity of oil (from tens percent to several times) and, as a consequence, an increase in the crowding rate.

However, the application of this gas, as well as any other low – viscosity agent, is accompanied by a decrease in the coverage factor.

Review of the research sources and publications

Gas condensate deposits in their initial state are characterized by high reservoir pressures, usually reaching several tens of MPa. There are deposits with relatively low (8 – 10 MPa) and very high (up to 150 – 180 MPa) initial reservoir pressures. The main reserves of hydrocarbons in deposits of gas – condensate type are confined to objects with initial reservoir pressures of 30 – 60 MPa. In the domestic gas industry, the development of gas condensate deposits was carried out until recently in the mode of using only the natural energy of the reservoir. Such a mode ("exhaustion") requires for its implementation of the minimum capital investment and relatively moderate current material and financial costs. In contrast to the development of a purely gas reservoir, in this case it is necessary to deal with the product, the composition of which is constantly changing. This is due to the phenomena of retrograde condensation of the formation hydrocarbon mixture, which occur when the formation pressure is reduced. The high molecular weight hydrocarbon components of the mixture, after lowering the pressure in the reservoirs below the pressure of the condensation start, are transferred to the liquid phase, which remains immobile practically throughout the development of the deposit due to low phase saturation (no more than 12 – 15%), much lower threshold of hydrodynamic mobility (about 40 – 50%).

GS Stepanova and VN Shustef studied in detail the peculiarities of the process of differential condensation of the reservoir mixture, performing simultaneously for comparison of calculations by contact condensation. According to these researchers, the marginal pressure, below which the calculated composition of the gas phase for differential and contact processes is not the same, is approximately 20 MPa [4, 10, 12].

Problem statement

To create and implement a scientifically – based monitoring system for the development of a gas – condensate deposit in conditions of low reservoir pressure with the effect on the reservoir by gas injection, which includes methods and means of control over the implementation of technology, methods for their forecasting.

Basic material and results

The injection of CO_2 into the reservoir is one of the most effective ways to increase oil recovery. Dioxide of carbon, as well as hydrocarbon solvents, provides a very high percentage of extraction and deprived of their basic disadvantages – the price of the juice.

Carbon dioxide or carbon dioxide forms a liquid phase at temperatures below 310 °C. At a temperature above 31 ºC, carbon dioxide is in a gaseous state, with a pressure of less than 7.2 MPa – from the liquid passes into a vaporous.

The principle of application [4] CO_2 is based on the dependence of the viscosity of fluids in reservoir conditions on the amount of CO_2 dissolved in them. For example, the dissolution of CO_2 in oil reduces its viscosity within 10 – 50%. At the same time, the volume ratio of oil with dissolved gas increases to 50%.An increase in the volume of oil contributes to the growth of the volume of pores occupied by oil, creates favorable conditions for its movement. Reducing the viscosity of the oil leads to an increase in its mobility. In this regard, in order to achieve a given coefficient of oil consumption, a smaller amount of displacing agent is spent.

Due to the solubility of CO_2 in reservoir water, the initial viscosity of water is noticeably increased, as a result of the ratio of the movement of oil and water increases. Dioxide of carbon in the system also leads to a decrease in the surface tension at the boundary of oil – water.

Efficiency of displacement of oil by carbon dioxide is determined both by increasing the coefficient of coverage by the effect and displacement. The increase in the coverage factor by area and volume is due to improved capillary absorption and equalization of the mobility of water and oil.

The ability of carbon dioxide to be readily dissolved in oil and water is a key property that determines the high efficiency of oil displacement with the use of carbon dioxide.

This property also contributes to the separation and washing of the oil film from the top of the rock, increases the wettability of the porous medium with water and thus contributes to capillary water collection in a porous medium saturated with oil, resulting in an increase in the amount of oil displaced. Depending on the composition of oil, pressure, temperature, the solubility of CO_2 in it may be either limited, or close to unlimited. The solubility of carbon dioxide in real oil can reach hundreds of volumes of CO_2 per one volume of oil. So, in other equal conditions, carbon dioxide is better soluble in petroleum with a high content of hydrocarbons in the C3 – C7 series. The high content of resins and asphaltenes in oil, on the contrary, greatly complicates its dissolution. For this reason, unlimited solubility of carbon dioxide in oil is practically impossible. Dioxide of carbon, depending on the thermodynamic conditions (pressure, temperature) can be in solid, liquid and gaseous state (Fig. 1).

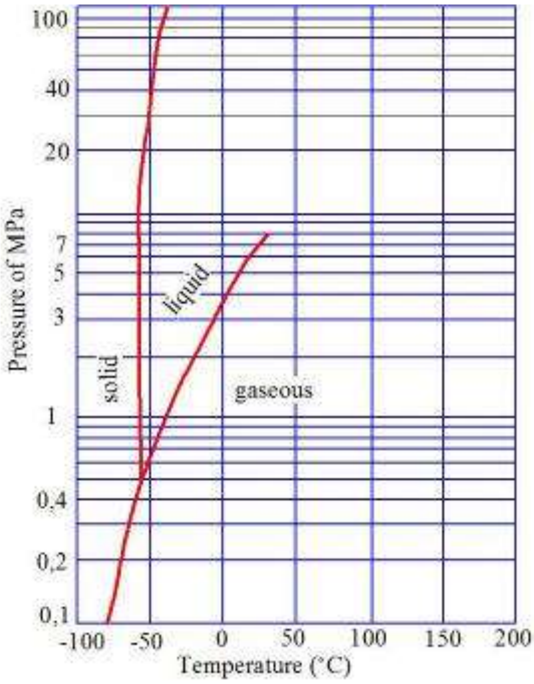

Figure 1 – Carbon dioxide phase diagram

In the chart below, we can judge what kind of oil displacement using carbon dioxide will occur at specific reservoir temperatures and pressures. So, if [5] the formation temperature is < 319ºC. and pressure > 7.5 MPa, then the oil will be replaced by liquid carbon dioxide. If the formation temperature is > 31ºC, the most likely is the displacement of oil by carbon dioxide.

The gaseous carbon dioxide is colorless, has a slightly sour smell and taste. The molecular weight of the compound is 44.010. The density of CO_2 at normal pressure and temperature 0 0C is 1.98 kg/m^3.

With increasing temperature under constant pressure and oil composition, the efficiency of CO_2 in it decreases. With constant oil composition and temperature increase of pressure causes increase of solubility of carbon dioxide. Dioxide of carbon dissolves sufficiently well in water. However, this process is limited in nature. It is influenced by pressure, temperature and degree of mineralization. So, with increasing pressure at constant mineralization and temperature, the solubility of carbon dioxide in water rises. With constant mineralization of water and pressure with increasing temperature the process is ambiguous. When [4] constant pressure and temperature with an increase in mineralization, the solubility of CO_2 in water decreases. Depending on the specific conditions, the solubility of carbon dioxide in water can reach 20%.

The aqueous solution of carbon dioxide reacts with carbonates in kind, dissolves them, while increasing the permeability of the collector and absorbing the ability of the injection wells. When dissolved in carbon dioxide in oil and water, the viscosity of the latter varies.

So, with the increase in the content of dissolved CO_2 depending on the composition of oil, pressure, the degree of pressure and temperature, there is a decrease in the viscosity in 2 – 15 times compared with the initial at zero content of carbon dioxide, while for more viscous oil in much higher degree than for less viscous.

With increasing pressure at constant values of the initial composition of oil and temperature, its viscosity with CO_2 dissolved in it takes ever lesser significance. This is due to the increase in the content of dissolved carbon dioxide in oil.

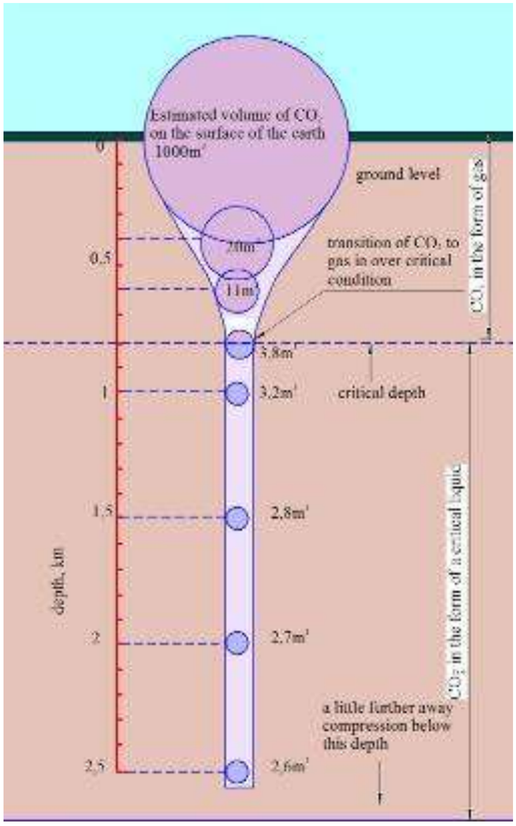

Figure 2 – The scheme of transition of carbon dioxide into a liquid state

With the constant composition of oil and pressure with increasing temperature, the viscose oil when dissolved in it, CO_2 is reduced to a lesser extent, as the solubility of carbon dioxide also decreases. Knowledge of the mechanism of reducing the viscosity of oil when dissolved in it, CO_2 is necessary in the prediction of technological indicators of processes of displacement of oil using carbon dioxide.

At constant saturation pressure with increasing concentration of carbon dioxide the density of oil increases. Increasing the pressure above saturation pressure also contributes to increasing its density. As the temperature rises, it decreases. The pressure, the composition of oil, the ratio of volumes of gas and oil and temperature affect the change in the density of oil with dissolved CO_2 to the same extent as these factors affect the very solubility of carbon dioxide in oil. When carbon dioxide is dissolved, carbon dioxide is formed in water, which, interacting with carbonate rocks – dolomite, sandstone with carbon – tinny cement, leads to an increase in porosity, permeability, degree of heterogeneity of these rocks.

The mechanisms discussed above for reducing the viscosity and increasing the volume of oil when dissolved in it, carbon dioxide leads to an increase in the mobile – state of the oil phase, which facilitates the displacement of oil. In addition, there is an increase in phase permeabilities for oil and water when they are in contact with CO_2.

Thus, carbon dioxide [9] when interacting with oil, water and the rhizome phase causes a change in the physical and chemical properties of the latter.

Carbon dioxide or carbon dioxide forms a liquid phase at temperatures below 31 ºC. At a temperature above 31 ºC, carbon dioxide is in a gaseous state, with a pressure of less than 7.2 MPa – from the liquid passes into a vaporous.

The technological scheme of injection of CO_2 is based on the existing general scheme of industrial arrangement of the deposit and, in particular, on the use of objects of the existing flood system. At the same time it is possible to supply CO_2 to well, using the injection pipelines of the existing production system, to build new pipelines or to deliver tanks.

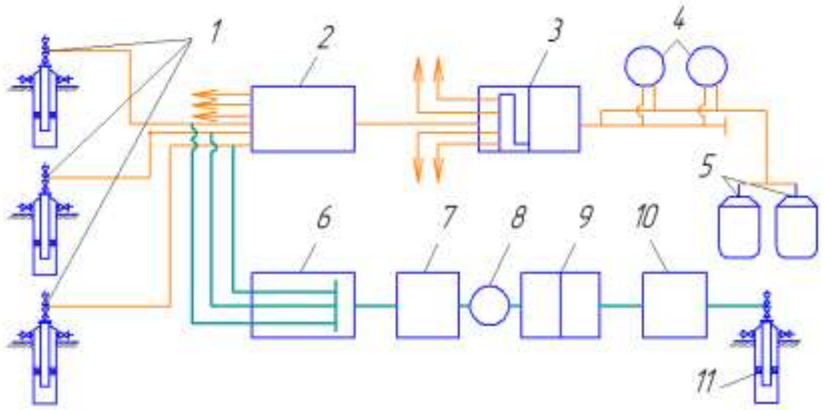

Figure 3 – Technological scheme of injection of CO_2:
1 – injection wells; 2 – point of distribution CO_2; 3 – pumping station; 4 – buffer tanks for CO_2; 5 – CO_2 tanks; 6 – pressure head of the flood system; 7 – block bush pumping station; 8 – buffer tanks; 9 – the main pumping station and water treatment station; 10 – station of the first lifting; 11 – water well

The CO_2 is pumped into the reservoir on the column of the pump – compressor pipes. For maintenance of the operating column from corrosion and high injection pressures, created when the CO_2 is injected into the formation, into the injection wells, injecting the packer devices. Wells are equipped with a standard fitting with the necessary devices to control the process of pumping CO_2 into the formation.

To pump CO_2, special acid – proof pumps with remote control and automatic protection are used. For a continuous supply of CO_2 in the pumping station, reserve pumps are provided. All technological pipelines of a pumping station should be calculated on a pressure with a 1: 4 stock ratio.

The hydrocarbonic acid of H_2CO_3 formed during the dissolution of CO_2 in water dissolves cement in the formation of the formation and thus increases the permeability. Dioxide of carbon in water contributes to the breakdown and "laundering" of film oil that covers the grain of the breed and reduces the possibility of breaking the water film.

At reservoir pressure above [8], the pressure of complete mixing of carbon monoxide from carbon monoxide will suppress oil as an ordinary solvent (mixing displacement).

In a layer three zones are formed:

1) the zone of primitive reservoir oil;
2) transition zone;
3) a zone of clean CO_2.

If CO_2 is injected into a flooded deposit, then a CO_2 shaft is formed in front of the CO_2 zone, which displaces the formation water.

Dioxide of carbon has oil – retaining properties, due to its abilities:

1) readily dissolve in oil and in sewage water, and vice versa, dissolve in itself oil and water;

2) reduce the viscosity of oil, and increase the viscosity of water when dissolved in them, reducing the mobility of water relative to oil;

3) to increase the volume of oil when dissolved in it CO_2 and to increase the efficiency of displacement of oil [10];

4) to reduce the interfacial tension on the edge of oil and water, to improve the moisture of the rock with water when dissolved in oil and water and to ensure the transfer of oil from the film state to the drip;

5) increase the permeability of individual types of collectors as a result of chemical interaction of coal and rock skeletons;

With the displacement of CO_2 oil, depending on the specific conditions, different schemes may apply (Table 1).

Displacement with mixing. The displacement scheme is carried out by feeding both gaseous and liquid carbon dioxide into the formation. Prerequisite is $P_{pl} > R_m$ (mixing), the pressure at which the total mutual dissolution of the displacing agent and the extruded medium occurs. The mixing pressure depends on the temperature and composition of the formation oil, which is generally characterized by the molecular weight.The displacing agent zone in which carbon dioxide is located, as a rule, in a gaseous ($T_{p1} \geq T_{cr}$), or in a liquid state ($T_{p1} \leq T_{cr}$; $P_{p1} \geq P_s$). When $P_{p1} \geq P_s$, the zone of complete mutual solubility is absent and it turns out that displacement occurs without mixing [7].

Oil displacement with gaseous carbon dioxide. At subcritical temperatures in shallow oil horizons and at limited pumping rates provided – P_{p1} (reservoir pressure) < P_s (condensation pressure CO_2). At supercritical temperature ($T_{p1} \leq T_{cr}$, where T_{p1} is the formation temperature, T = 31.04 °C is the critical temperature of CO_2), the displacement process is not thermodynamically limited and proceeds at any pressure values in the formation. Elimination of liquefied CO_2. Realized with $T_{p1} < T_{cr}$, $P_{p1} > P_s$. Component and phase characteristics of this scheme: repressing agent – liquid CO_2, displaced environment – liquid hydrocarbons and formation water.

The displacement is carbonated with water. Less [8] depends on pressure and temperature, with two – phase (liquid – liquid) filtration taking place, and CO_2 is present in both phases, more in water and less in displaced oil – in the zone adjoining the boundary of the phase separation. Pressure with more pressure of the power of CO_2 in water – P_{solute}.

Table 1 – Mechanism and schemes of influence

Scheme of influence	Mechanism of displacement, acting under this scheme	Thermodynamic constraints		Characteristics of the fluid: components, phases
		by temperature	by pressure	
Exhaust gaseous CO_2	Displacement with mixing. Change in viscosity	$T \leq T_{cr}$ $T \geq T_{cr}$	$P \leq P_s$	Gas phase: CO2. Liquid phase or gas – liquid mixture: hydrocarbons (petroleum)
Exhaust gaseous CO_2	Displacement with mixing	$T \geq T_{cr}$	$P \leq P_s$	Carbon dioxide, oil
Displacement with mixing	Displacement with mixing. Bulk effect	$T \leq T_{cr}$ $T \geq T_{cr}$	$P \geq P_{mix}$	1. Liquid hydrocarbon phase. 2. A gas – liquid zone: a mixture of explosives and carbon dioxide. 3. Zone of complete mutual solubility: gaseous mixture of hydrocarbons and CO_2 (without boundaries of phases). 4. Propagation zone: gaseous (predominantly) or liquid (sometimes) CO_2. 5. The zone of complete mutual solubility is absent.
Displacement with carbonated water	Change of viscosity of interphase tension	$T \leq T_{cr}$ $T \geq T_{cr}$	$P \leq P_{mix}$ $P \geq P_s$	Oil phase: hydrocarbons and CO_2 (insignificant quantity). Water phase: water and CO_2 (high content). Gas phase: hydrocarbons and CO_2.

An aqueous solution of carbon dioxide reacts with carbonate in – kind, dissolves them, while increasing permeability. We calculate how the well flow changes after the injection into the carbon dioxide layer and how the permeability changes with the following output data (Table 2).

The radius of the carbon dioxide penetration zone:

$$r_w = 0.5D = 0.73 \text{ m}; \tag{1}$$

$$R_{CO_2} = \sqrt{\left(\frac{V_{CO_2}}{\pi \cdot h \cdot m}\right) + r_w^{\ 2}} . \tag{2}$$

The permeability of the bottomhole zone of the reservoir after injection of CO_2 is equal (according to industrial data):

$c_к = 0.12$ – coefficient of carbonaceous rock;

$$k_p = 1.8 \cdot e^{(0.25 c_k)} ; \tag{3}$$

$k = 2.15$ – growth rate of permeability;

$$k_{bfz} = k_p \cdot k . \tag{4}$$

Average (reduced) reservoir permeability after injection of carbon dioxide

$$k_m = \frac{k \cdot k_{bfz} \cdot \log\left(\frac{R_k}{r_w}\right)}{k_{bfz} \cdot \log\left(\frac{R_{CO_2}}{r_w}\right) + k \cdot \log\left(\frac{R_k}{r_{CO_2}}\right)} . \quad (5$$

Expected effect after injection of carbon dioxide (CO_2)

$$E = k_m / k. \quad (6)$$

Determine the expected well flow after loading CO_2

$$q_2 = q_1 F, \quad (7)$$

where q_1 – the discharge of the well to the injection of carbon dioxide.

Determine the expected oil well after the injection of carbon dioxide, with known water content of the product.

$$q_{2n} = q_2 (1 - n). \quad (8)$$

This calculation will be carried out 15 times. The obtained results will be analyzed using statistical analysis, and for this we will use the STATISTIKA 10 program.

Having statically processed the results got:

– the dependence of the volume of CO_2 injection from the average reservoir permeability after injection of CO_2, where the correlation coefficient was $r = 0.995$, and Fisher's criterion is equal $F = 107.02$ at a critical point $F_{critical} = 161.45$

$$V_{CO_2} = 14.2 + 0.8594 k_m; \quad (9)$$

– the dependence of the volume of pumping of CO_2 from the expected debit of the well after the injection of CO_2, where the correlation coefficient $r = 0.98$, and Fisher's criterion $F = 42.56$ at a $F_{critical} = 161.45$

$$q_2 = 7.8928 + 1.0629 V_{CO_2}; \quad (10)$$

– dependence of the well flow to the injection of carbon dioxide from the expected value of the well bore at the known watering of the products where the correlation coefficient $r = 0.998$, and Fisher's criterion $F = 37.824$ at a $F_{critical} = 161.45$

$$q_2 = 4.4664 + 0.9578 q_1; \quad (11)$$

– dependence of the well flow rate on the injection of dioxide from the expected well flow at the known watering of the products carbon where the correlation coefficient $r = 0.999$, and Fisher's criterion $F = 33.110$ at a $F_{critical} = 161.45$

$$q_1 = 0{,}5751 + 0.9934 q_2. \quad (12)$$

Since $F_p \geq F_{critical}$, the obtained regression equation is assumed to be statistically significant. (The hypothesis of model adequacy was confirmed in all cases).

The results indicate that an important role in the injection of carbon dioxide is played by indicators such as reservoir permeability and water content of products. We have received that the rate will increase almost twice, if the production of wells will not be watered, and if the well products are significantly watered, then we will get equal oil extraction after the injection CO_2.

Table 2 – Output data for the design of the process of injection of carbon dioxide into the formation

№	Well diameter, m (D)	Volume of carbon dioxide, m^3 (Vco_2)	The thickness of the productive layer, m (h)	Sponginess m	Coefficient of carbonaceous rock c_k	Growth rate of permeabilityk	Power supply radius, m (R_k)	The well flow, tons / day (q_1)	Wateriness of products (n%)
1	0.245	12	11	0.12	5.11	5E^{-14}	850	4.56	55
2	0.089	14	15	0.16	5.96	5E^{-14}	800	3.74	63
3	0.095	17	17	0.16	7.24	5E^{-14}	750	2.23	78
4	0.219	11	10	0.12	4.68	5E^{-14}	700	3.74	63
5	0.273	12	13	0.12	5.11	5E^{-14}	650	3.32	65
6	0.219	15	14	0.16	6.39	5E^{-14}	600	2.88	75
7	0.245	18	18	0.22	7.66	5E^{-14}	550	2.23	78
8	0.273	18	20	0.22	7.66	5E^{-14}	500	1.38	87
9	0.299	12	12	0.14	5.11	5E^{-14}	450	4.56	55
10	0.325	15	16	0.16	6.39	5E^{-14}	400	1.38	63
11	0.351	15	15	0.16	6.39	5E^{-14}	350	1.38	87
12	0.377	11	10	0.12	4.68	5E^{-14}	300	2.23	78
13	0.426	17	17	0.22	7.24	5E^{-14}	250	2.88	75
14	0.457	15	15	0.16	6.39	5E^{-14}	200	1.98	80
15	0.508	11	11	0.12	4.68	5E^{-14}	250	2.51	76

Table 3 – Results of designing the process of carbon dioxide injection

№	Well radius, m r_w	Radiation of carbon dioxide penetration, m. Rco_2	Permeability of the breedk$_p$	Permeability of the bottomhole zone of the formation k_{bfz}	Average reservoir permeability after injection CO_2 k_m	Expected effect after injection CO_2 E	Expected discharge of the well after injection $CO_2 q_2$	Expected discharge of wells by oil after injection of CO_2, with known water content of products q_{2n}
1	0.1225	1.705931	6.456173	3.22809E^{-13}	1.22989E^{-13}	2.459781	11.2166	5.047471
2	0.0445	1.363719	7.98777	3.99389E^{-13}	1.16063E^{-13}	2.321259	8.68151	3.212159
3	0.0475	1.411631	10.99262	5.49631E^{-13}	1.21978E^{-13}	2.439561	5.440221	1.196849
4	0.1095	1.712107	5.804301	2.90215E^{-13}	1.15741E^{-13}	2.314818	8.657418	3.203245
5	0.1365	1.571118	6.456173	3.22809E^{-13}	1.25403E^{-13}	2.50807	8.326792	2.914377
6	0.1095	1.464449	8.884864	4.44243E^{-13}	1.31618E^{-13}	2.632363	7.581206	1.895301
7	0.1225	1.209381	12.22718	6.11359E^{-13}	1.50695E^{-13}	3.013897	6.720991	1.478618
8	0.1365	1.149552	12.22718	6.11359E^{-13}	1.56147E^{-13}	3.122938	4.309655	0.560255
9	0.1495	1.515634	6.456173	3.22809E^{-13}	1.25224E^{-13}	2.504482	11.42044	5.139198
10	0.1625	1.375663	8.884864	4.44243E^{-13}	1.40722E^{-13}	2.814435	3.88392	1.43705
11	0.1755	1.421705	8.884864	4.44243E^{-13}	1.40098E^{-13}	2.801964	3.86671	0.502672
12	0.1885	1.718969	5.804301	2.90215E^{-13}	1.18921E^{-13}	2.378412	5.303858	1.166849
13	0.213	1.22187	10.99262	5.49631E^{-13}	1.58409E^{-13}	3.168187	9.12438	2.281095
14	0.2285	1.429216	8.884864	4.44243E^{-13}	1.41757E^{-13}	2.83515	5.613597	1.122719
15	0.254	1.64877	5.804301	2.90215E^{-13}	1.25968E^{-13}	2.519356	6.323583	1.51766

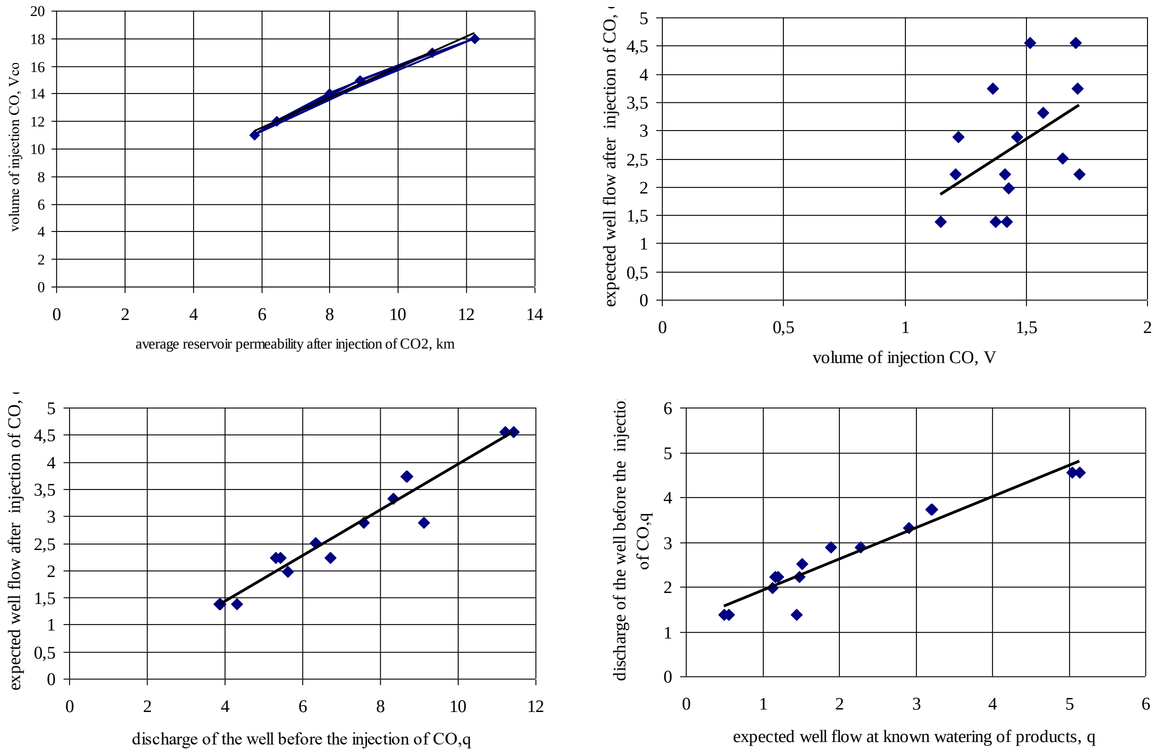

Figure 4 – Graphic dependences of the statistical processing results

Conclusions

It is known that it is very difficult to extract residual oil reserves of especially viscous and saturated water, and when the carbon dioxide is pumped out, which dissolves well into the oil, increases its volume and reduces the viscosity, on the other hand, dissolves in water, increases its viscosity.

Thus, the distillation of carbon dioxide in oil and water leads to equalization of the mobility of oil and water, which creates opportunities for obtaining higher oil yields, both by increasing the displacement ratio and the coefficient of coverage of the oil deposit.

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THE STUDY OF PROFITABILITY OF DWELLING RECONSTRUCTION OF FIRST MASS SERIES

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The study of organizational and technological factors on dwelling reconstruction profitability of first mass series with the purpose to define the effective organizational solutions of reconstruction is described. The study is carried out on the basis of example of dwelling reconstruction of typical series 1-438$_{2.5}$-7. The informative and graphical models of reconstruction are made. The factors having impact on reconstruction indexes are defined. The plan of numerical experiment considering simultaneous impact of the given factors on the profitability index of reconstruction is made. With the help of experimental and statistical modeling mathematical models of dependency of profitability on the factors in the form of charts are made. The charts analysis defined the areas of effective solutions by simultaneous impact of factors. By the results of studies the conclusions about the conditions of such dwellings reconstruction operations are made.

Key words: charts, the areas of effective solutions, models of reconstruction, experimental and statistical modeling, organizational and technological factors.

ДОСЛІДЖЕННЯ ЗМІНИ РЕНТАБЕЛЬНОСТІ РЕКОНСТРУКЦІЇ ЖИТЛОВИХ БУДИНКІВ ПЕРШИХ МАСОВИХ СЕРІЙ

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Описане дослідження впливу організаційно-технологічних факторів на рентабельність реконструкції житлових будинків перших масових серій, з метою визначення ефективних організаційних рішень реконструкції. В інформаційних джерелах відсутня інформація щодо методики дослідження реконструкції подібних об’єктів. А фактична реконструкція будинків носить епізодичний характер та не дає можливості визначити вплив факторів на показники реконструкції, тому що проходить при фіксованих значеннях факторів. Як наслідок – небажання інвесторів вкладати кошти в реконструкцію житлових будинків типових серій. Для вирішення поставленої проблеми були вибрані впливові фактори, визначений перелік робіт, побудовані та досліджені абстрактні моделі реконструкції. Дослідження проведені на прикладі реконструкції житлового будинку типової серії 1-438$_{2.5}$-7. Побудовані інформаційні та графічні моделі реконструкції. Визначені фактори що впливають на показник реконструкції серед яких: кількість робочих годин на тиждень, ступінь суміщення робіт, необхідна частка заміни прорізів та комунікацій. Визначені рівні варіювання кожного з факторів: мінімальне, максимальне та середнє значення. Складено план чисельного експерименту що враховує одночасний вплив визначених факторів на показники реконструкції. Отримані основні показники реконструкції: тривалість, вартість та рентабельність робіт. За допомогою експериментально-статистичного моделювання побудовані математичні моделі залежності показника рентабельності від факторів у вигляді математичних формул та діаграм. Аналіз діаграм визначив зони ефективних рішень при одночасному впливі всіх факторів. За результатами досліджень зроблені висновки про умови проведення реконструкції подібних будинків.

Ключові слова: діаграми, зони ефективних рішень, моделі реконструкції, експериментально-статистичне моделювання, організаційно-технологічні фактори.

Introduction

During 60-80 years of the XX century in the countries of the former USSR and Europe the dwellings mass building of first mass series has started. The USSR has been built in postwar period, thus the works were carried out under the conditions of strong economy of material costs. Dwellings were considered as lodging of 25-30 years of operation. [1]. The amount of such dwellings forms 25% of all housing facilities of Ukraine [2]. The level of comfortable living is very low, flats size length is small, a bathroom is combined with a toilet, inner walls have weak sound insulation. This type of buildings is characterized by great amount of heat loss, lack of elevator, balconies, rubbish chute and attic. The analysis of demolition of such buildings forms 30-40%.[3]. Every third house needs urgent repairing or modernization in Ukraine. Considering the described difficulties reconstruction of such houses is possible and enables to prolong their service life till 50 years. Lack of such houses reconstruction capabilities and demolition analysis in modern literature; calculations of reconstruction additional financing lead to investors' unwillingness to put money into such projects. Study of abstract models of reconstruction with the help of computer programs enabled to obtain calculations and to trace the impact of organizational and technological factors on reconstruction indexes including profitability.

Review of the research sources and publications

As a rule, volumetric and planning solutions of such houses are described the native literary sources. According to them, the set of reconstruction measures to raise the living comfort level is proposed [4 – 6]. The description of implemented objects enabled to define the set of works that can be used for majority of series of such houses. In foreign literary sources there are examples of successful reconstruction [7 – 10], and the necessity of houses demolition of such series. On the basis of analyzed publications it can be concluded that the dwellings of first mass series can be divided into 2 groups: first group where reconstruction is proper, the second group where there are houses to be demolished.

Definition of unsolved aspects of the problem

The analysis of informative sources showed that lack of reconstruction methods impact description on technical and economical indexes does not enable to have an idea about the conditions of dwellings profitable reconstruction of first mass series. The amount of implemented objects in Ukraine is less than 20, thus it does not enable to make conclusions about factors influencing reconstruction indexes.

Problem statement

The study of simultaneous impact of factors by their different combination on reconstruction indexes enables to obtain maximum amount of the material for the study with minimum costs. The solution of the given task is implemented via making abstract models imitating reconstruction of the dwelling under different conditions and their study with usage of software. By the results of the study the areas of effective solutions of profitable reconstruction have been defined.

Basic material and results

The examples of carried out reconstruction showed that reconstruction is made by different conditions of works execution: under eviction or habitants' living in the house while doing the reconstruction. The conditions of works execution have impact on amount of working hours per a week. Two basic models have been made for the study considering these conditions.

The studies were conducted using mathematical theory of experiment planning, which is the basis of experimental and statistical modeling [11, 12]. According to the classical theory of reduced experiment varying factors planning must be over the range according to classical –1; 0; +1. Every factor and the ranges of varying are presented below in details.

The first factor-amount of working hours per week (X_1) depends on the amount of working hours per week and the amount of working hours per day [13].

Accepted amount of working hours per week is:

– by habitants' eviction while conducting reconstruction: 40, 80 and 112 hours.

– by habitants' living during reconstruction: 40, 48 and 60 hours.

The second factor – degree of works' overlapping (X_2) presents ratio of reconstruction period duration to the total of works duration in each division (Formula 1) is characterized by coefficient of overlapping and is variable from 0.15, 0.2 and 0.25

$$X_2 = k_{com} = \frac{T_c}{\Sigma_1^N \cdot \Sigma_1^n t_i} \;, \qquad (1)$$

where:

T_c – duration of the reconstruction period , days;

N – amount of processes;

n – amount of divisions by flow organization;

t_i – duration of i-th flow, days.

The third factor – necessary part of slots displacement (X_3) was defined on the basis of visual inspection of such houses and forms 20, 50 and 80% from total amount of slots.

The fourth factor – necessary part of communications replacement (X_4) depends on the quantity of porches and risers in analyzed house and it is variable from: 8.33, 50.0 and 91.66% from total amount. The range of variation of two last factors is connected with independent replacement of separate elements by habitants during house operation.

Reconstruction profitability presents relation of profit to reconstruction cost. For its determination it is needed to obtain works duration, to calculate the cost and profit from the dwelling reconstruction.

The studies were carried out by the example of dwelling reconstruction in one of the spread series 1-438$_{2.5}$-7. For the reconstruction laboriousness there is formed informative model in the form of estimate in the program АВК-5 (3.2.2) at prices of 2016 year.

With the help of program Microsoft Project on the basis of works laboriousness it is made linear charts by simultaneous impact of all factors according to two plans of numerical experiment. The first plan considered the condition of habitants' eviction while making operations (Table 1-2). The second plan was elaborated under the condition of habitants' living during the reconstruction.

The cost of reconstruction considered the costs of habitants' eviction during operations by formula

$$C = C_{r.m.w.} + C_{fl.b.} + C_{t.l.}\,, \tag{2}$$

where:
C – cost of object's reconstruction, hryvnas.;
$C_{r.m.w.}$ – cost of repair and construction works by estimation, hryvnas;
$C_{fl.b.}$ – costs on flat's buying on the ground floor connected with flats' alterations for additions to elevator shaft, hryvnas;
$C_{t.l.}$ – costs on payment of temporary lodging to habitants during reconstruction

$$C_{t.l.} = C_{m.c.} \times T_r\,, \tag{3}$$

where:
$C_{m.c.}$ – monthly costs on payment of temporary lodging to habitants during reconstruction, hryvnas;
T_r – reconstruction duration, months.

Reconstruction profitability is in inverse proportion to works cost. Calculations were made by formula

$$P = \frac{Pr}{C} \times 100\,\%\,, \tag{4}$$

where:
P – profitability, %;
Pr – profit of reconstruction object, thousand hryvnas;
C – cost of object reconstruction, thousand hryvnas;

Reconstruction profit was calculated by formula

$$Pr = C_p \times C, \tag{5}$$

where:
Pr – profit of object's reconstructionї, hryvnas.;
C_p – profit from new flats' sale on the ground, fifth and sixth floors after reconstruction.

On the ground floor the flats are repurchased and replanned because of additions to elevator. After reconstruction termination built-on flats on the fifth and sixth floors and altered flats of the ground floor are put up for sale.

Factors impact on indexes is ambiguous: in case of eviction the first two factors have an impact on works duration (Table 1), and all the fourth only on cost and profitability (Table 2). By habitants' living in the house the factors impact is pairwise: the first two factors have an impact on reconstruction duration (Table 3), and the third and the fourth on cost and profitability (Table 4). On the basis of calculations, it is obtained numerical indexes of duration, cost and profitability of dwelling reconstruction (Table 1-4).

To define the factors impact on dwellings reconstruction profitability of first mass series it is made mathematical models with the help of program COMPEX. The description of dependencies of factors impact on profitability is presented in the form of analytical formula and charts.

Factors impact on profitability by habitants' eviction during reconstruction is described by formula

$$\begin{aligned} Y_1 = P &= 7.531 + 10.402\, X_1 - 3.98\, X_1^2 + \\ &+ 1.58\, X_1X_2 - 0.44\, X_1X_3 - 0.064\, X_1X_4 - \\ &- 5.108\, X_2 - 0.904\, X_2^2 + 0{,}241\, X_2X_3 + \\ &+ 0.031\, X_2X_4 - 2.28\, X_3 + 0.144\, X_3^2 + \\ &+ 0.015\, X_3X_4 - 0.33\, X_4 + 0.074\, X_4^2\,. \end{aligned} \tag{6}$$

The analysis of the given mathematical dependency is shown that factor X_1 has the most sufficient impact on dwelling reconstruction profitability (amount of working hours per week). Positive mark of the coefficient X1 indicates that the change of this factor is in direct ratio to profitability change (Y_3).

Table 1 – Plan of experiment and impact of varying factors X_1 and X_2 on the index of duration of the dwelling reconstruction duration (Y_1), condition of reconstruction – habitants' eviction during operations

№ of point	Coded factors		Full-scale factors		Indexes
	X_1 – amount of working hours per week	X_2 – coefficient of works' overlapping	X_1 – amount of working hours per week (hour)	X_2 – coefficient of works' overlapping	Y_1 – reconstruction duration, (days)
1	*2*	*3*	*4*	*5*	*6*
1	-1	-1	40	0,15	484
2	-1	0	40	0,2	670
3	-1	1	40	0,25	807
4	0.11	-1	80	0,15	243
5	0.11	0	80	0,2	335
6	0.11	1	80	0,25	404
7	1	-1	112	0,15	173
8	1	0	112	0,2	239
9	1	1	112	0,25	288

Table 2 – Plan of experiment and impact of varying factors (X_1, X_2, X_3 and X_4) on the index of study reconstruction cost (Y_2) and profitability (Y_3), condition of reconstruction – habitants' eviction during operations

№ of point	Coded factors				Full-scale factors				Indexes	
	X_1 - amount of working hours per week	X_2- coefficient of works overlapping	X_3- necessary part of slots displacement	X_4 – necessary part of replacement of inner sanitary-engineering communications (%)	X_1 amount of working hours per week (hours)	X_2- coefficient of works overlapping	X_3-necessary part of slots replacement (%)	X_4 – necessary part of replacement of inner sanitary-engineering communications (%)	Y_2-reconstruction cost (mil.hryvnas)	Y_3-reconstruction profitability (%)
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*	*10*	*11*
1	1	1	1	1	112	0.25	80	91.66	25.17	8.56
2	1	1	1	-1	112	0.25	80	8.33	25.0	9.26
3	1	1	-1	1	112	0.25	20	91.66	24.05	13.6
4	1	1	-1	-1	112	0.25	20	8.33	23.89	14.37
5	1	-1	1	1	112	0.15	80	91.66	23.69	15.34
6	1	-1	1	-1	112	0.15	80	8.33	23.52	16.13
7	1	-1	-1	1	112	0.15	20	91.66	22.57	21.05
8	1	-1	-1	-1	112	0.15	20	8.33	22.41	21.93
9	-1	1	1	1	40	0.25	80	91.66	31.84	-14.21
10	-1	1	1	-1	40	0.25	80	8.33	31.68	-13.77
11	-1	1	-1	1	40	0.25	20	91.66	30.72	-11.08
12	-1	1	-1	-1	40	0.25	20	8.33	30.56	-10.61
13	-1	-1	1	1	40	0.15	80	91.66	27.69	-1.33
14	-1	-1	1	-1	40	0.15	80	8.33	27.52	-0.75
15	-1	-1	-1	1	40	0.15	20	91.66	26.57	2.82
16	-1	-1	-1	-1	40	0.15	20	8.33	26.41	3.45
17	-1	0	0	0	40	0.20	50	50	29.44	-7.22
18	0,11	1	0	0	80	0.25	50	50	26.02	4.98
19	0,11	-1	0	0	80	0.15	50	50	23.95	14.06
20	0,11	0	1	0	80	0.20	80	50	25.69	6.34
21	0,11	0	0	1	80	0.20	50	91.66	25.22	8.34
22	0,11	0	-1	0	80	0.20	20	50	24.57	11.18
23	0,11	0	0	-1	80	0.20	50	8.33	25.05	9.04
24	0,11	0	0	0	80	0.20	50	50	25.14	8.69
25	1	1	1	1	112	0.25	80	91.66	25.17	8.56

Factor X_2 has impact half as large (coefficient of works overlapping), and the factor X_3 has impact five times less (necessary part of slots replacement). The impact of factor X_4 (necessary part of communications replacement) is insignificant. Negative marks of coefficients of factors X_2, X_3, X_4 show that their impact is in inverse ratio to profitability raise (Y_3).

Impact of all 4 factors X_1 (amount of working hours per week), X_2 (coefficient of works' overlapping), X_3 (necessary part of slots' replacement) and X_4 (necessary part of communications replacement) on reconstruction profitability (Y_3) by their different combination is presented in the chart «square by square » (fig. 1). To make the chart, it is made 9 two-factor charts (impact of factors X_1 and X_2), which are situated on the basic square.The basic square considers the impact of factors X_3 and X_4. On the chart the interval of reconstruction profitability change is depicted by contour lines with pace 5%.

Table 3 – Plan of experiment and impact of varying factors X_1 and X_2 on the index of the dwelling reconstruction (Y_1), condition of reconstruction – habitants' eviction during operations

№ of point	Coded factors		Full-scale factors		Indexes
	X_1 – amount of working hours per week	X_2 – coefficient of works' overlapping	X_1 – amount of working hours per week (hour)	X_2 – coefficient of works' overlapping	Y_1 – reconstruction duration, (days)
1	*2*	*3*	*4*	*5*	*6*
1	-1	-1	40	0.15	483
2	-1	0	40	0.2	651
3	-1	1	40	0.25	821
4	-0,2	-1	48	0.15	402
5	-0,2	0	48	0.2	542
6	-0,2	1	48	0.25	673
7	1	-1	60	0.15	322
8	1	0	60	0.2	518
9	1	1	60	0.25	547

Table 4 – Plan of experiment and impact of varying factors X_3 and X_4 on the index of reconstruction cost (Y_2) and profitability (Y_3), condition of reconstruction – habitants' eviction during operations

№ of point	Coded factors		Full-scale factors		Indexes	
	X_3 – necessary part of slots replacement	X_4 – necessary part of replacement of inner sanitary-engineering communications	X_3 – necessary part of slots replacement (%)	X_4 – necessary part of replacement of inner sanitary-engineering communications (%)	Y_2 – reconstruction cost (mil.hryvnas.)	Y_3 – reconstruction profitability (%)
1	*2*	*3*	*4*	*5*	*6*	*7*
1	-1	-1	20	8.33	20.18	35.37
2	-1	0	20	50	20.26	34.83
3	-1	1	20	91.66	20.34	34.29
4	0	-1	50	8.33	20.74	31.69
5	0	0	50	50	20.83	31.18
6	0	1	50	91.66	20.91	30.67
7	1	-1	80	8.33	21.30	28.27
8	1	0	80	50	21.38	27.78
9	1	1	80	91.66	21.46	27.30

The study of factors' impact: necessary part of slots replacement (X_3) and necessary part of communications replacement (X_4) on reconstruction profitability (Y_3) without habitants' eviction is described by mathematical dependency by formula

$$Y_3 = P = 31.179 - 3.523\, X_3 + 0.127\, X_3^2 + 0.028\, X_3X_4 - 0.512\, X_4 + 0.02\, X_4^2\,. \quad (7)$$

The analysis of the obtained analytical dependency illustrates that factor X_3 has the most substantial impact on reconstruction profitability of dwelling (Y_3) gives factor X_3 (necessary part of slots replacement). Its impact is 7 times more than factor X_4 (necessary part of communications replacement). Negative marks of the coefficients X_3 i X_4 indicate that factors change is in inverse ratio to profitability change (Y_3).

The impact of two factors X_3 and X_4 on indexes of reconstruction profitability (Y_3) is depicted on 2-factor chart on Figure 2.

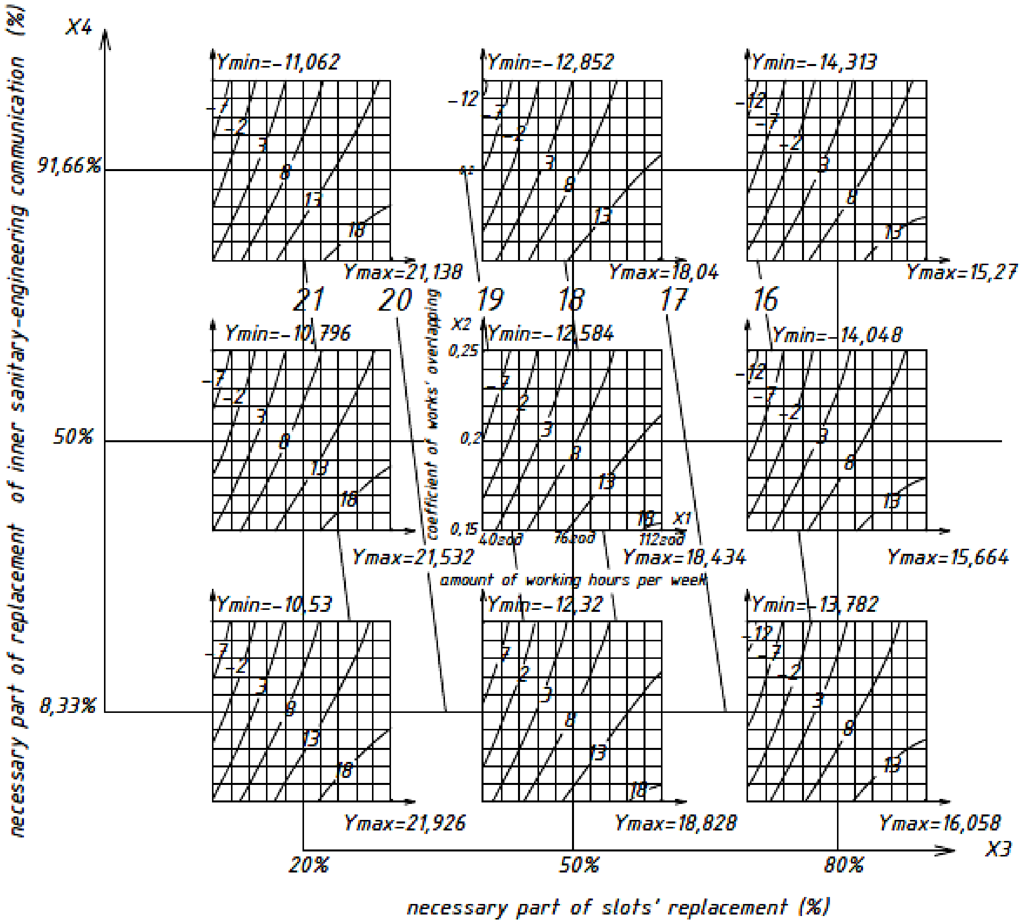

Figure 1 – Chart "square by square" presents impact of factors (X_1, X_2, X_3, X_4) on reconstruction profitability (Y_3) under the condition of habitants' eviction

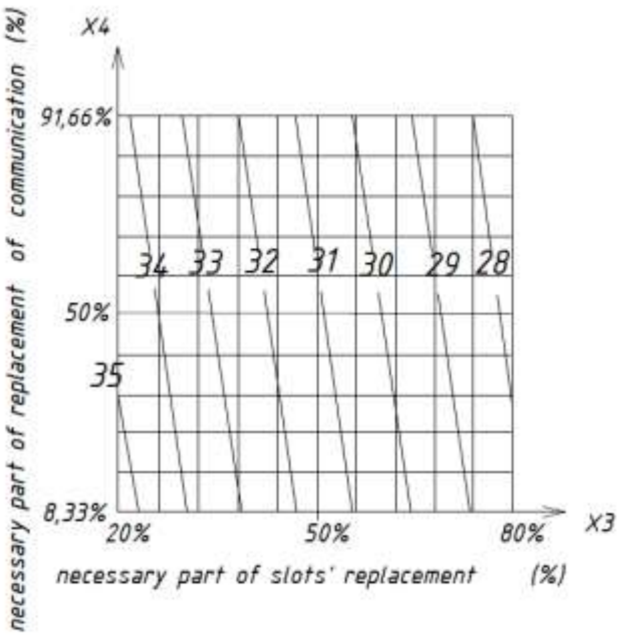

Figure 2 – 2-factor chart of factors impact (X_3, X_4) on reconstruction profitability (Y_3) without habitants' eviction

Change of index of reconstruction profitability (Y_3) is presented in the form of contour lines with interval 1%. Maximum value of reconstruction profitability index is + 35.371%, and minimum + 27.301%.

Edge of reconstruction profitability index efficiency has been adopted on the basis. By reconstruction on investments by the state this index is 8% and more.

In the given study the boundary permissible index of dwelling reconstruction is $Y_3 \geq 8\%$. Such proportion of profitability enables to execute works with getting profit.

As one can see from Figure 2 reconstruction without habitants' eviction is profitable by any combinations of factors' values and is more than 8%.

Superposition of restriction of profitability index $Y_3 \geq 8\%$ by reconstruction with habitants' eviction on charts of impact of factors X_1 and X_2 (Figure 3) illustrates that by any combination of factors reconstruction is unprofitable because there is an area of acceptable solutions in every chart. It can be reached the maximum indexes(Y_3) of reconstruction profitability by combination of factors X_1 from -1 to 0 (from 40 to 76 working hours per week), by X_2 from -1 to 0 (coefficient of works' overlapping from 0,15 to 0,25), by X_3 from -1 to 0 (necessary part of slots' replacement from 20% to 50%) and by X_4 from -1 to +1 (necessary part of communications' replacement 91.66%).

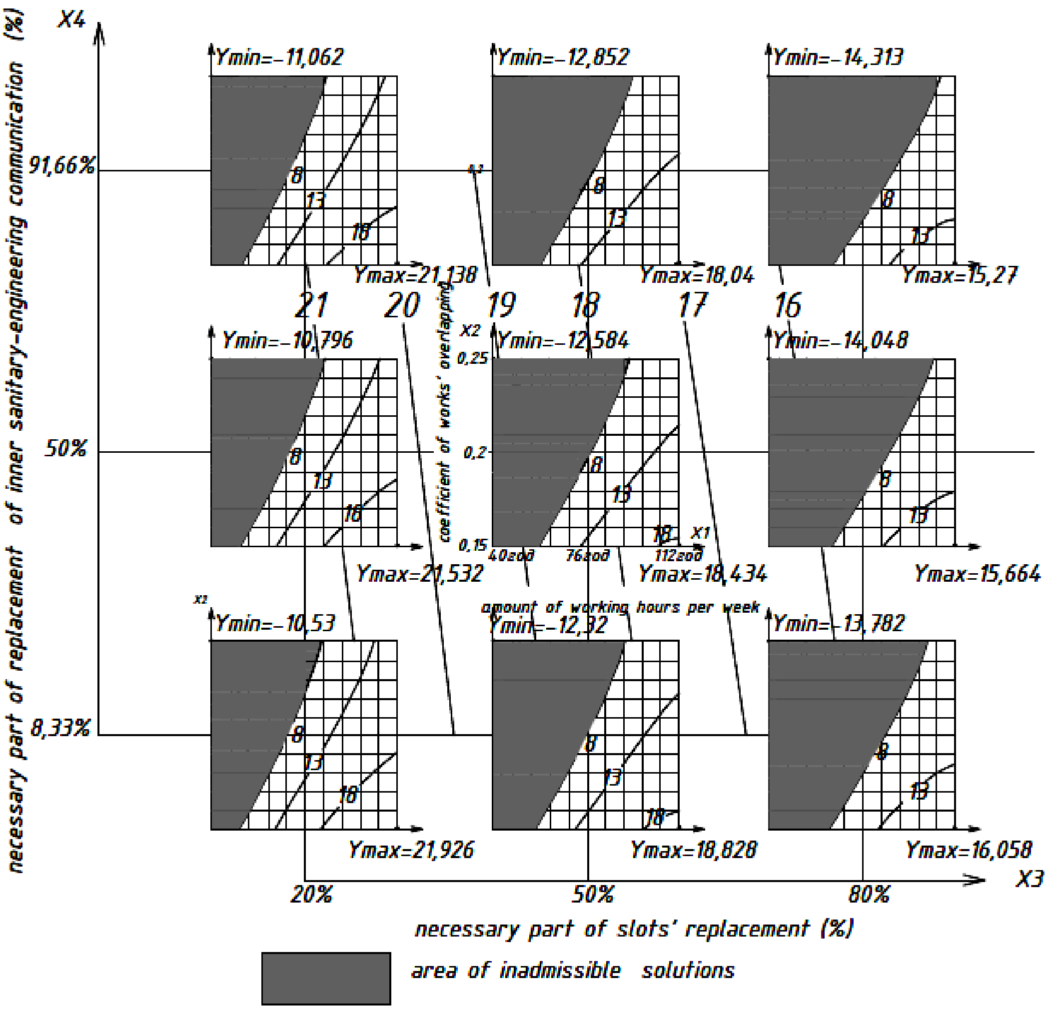

Figure 3 – The chart "square by square" illustrates the restriction of reconstruction profitability $Y_3 \geq 8\%$ by the condition of habitants' eviction

Conclusions

The dependencies between organizational and technological factors and indexes of reconstruction profitability have been obtained for the first time on the basis of experimental and statistical modeling.

The analysis of graphical image of profitability dependency on factors has illustrated that reconstruction by habitants' living in the house during the works execution exceeds the index of lucrative profitability in 3 times. By reconstruction with habitants' eviction during works' execution the areas of effective solutions have been defined. The results of the study illustrated that dwellings reconstruction is profitable. The methodology of the study can be used for the study of other indexes of dwellings reconstruction of first mass series.

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ORGANIZATIONAL INNOVATIONS IN THE ACTIVITIES OF CONSTRUCTION COMPANIES IN UKRAINE IN THE TRANSITION TO WORLD STANDARDS OF MANAGEMENT

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The scientific bases and practical recommendations for the construction enterprises of Ukraine on the progress of their structure and activity through the implementation of organizational and process innovations proposed in this work are considered. Such innovations are developed on the basis of studying and attracting the best world experience, the latest standards of management of construction investment projects, modern forms and methods of corporate management. Implementation of the proposed innovations will improve the quality, timeliness and efficiency of the construction of individual objects, should improve the structure, functions and performance of enterprises of the domestic construction complex, and ultimately - should increase the competitiveness of the economy and social life in new economic conditions.

Keywords: construction and investment projects, improvement of the organizational structure of the enterprise, project management standards.

ОРГАНІЗАЦІЙНІ ІННОВАЦІЇ В ДІЯЛЬНОСТІ БУДІВЕЛЬНИХ КОМПАНІЙ УКРАЇНИ ПРИ ПЕРЕХОДІ НА СВІТОВІ СТАНДАРТИ ГОСПОДАРЮВАННЯ

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Розглянуто наукові основи і практичні рекомендації щодо порядку формування та використання організаційних і процесних нововведень в діяльності будівельних компаній України при переходу на світові стандарти господарювання. Ці нововведення стосуються організаційної структури й основних процесів управління як окремих будівельно-інвестиційних проектів, так і діяльності всієї компанії в цілому. Розглядаючи повний життєвий цикл зведення окремих об’єктів визначено системну модель і перелік основних стандартів професійної діяльності та управління будівельними проектами за умови їх повної та ефективної реалізації «під ключ». В основу цього рішення покладено досвід найкращої практики будівництва передових компаній світу й американські стандарти проектного менеджменту PTB.K, які при цьому застосовуються. Крім того також визначено, що оскільки потужні будівельні компанії одночасно реалізують «портфель» практично не пов’язаних між собою проектів, то для їх успішного втілення в системі корпоративного менеджменту кожної компанії необхідно створити та якісно використовувати механізми й інструменти мультипроектного управління всім комплексом проектів. Для практичної матеріалізації такої ідеї запропоновано нову, більш досконалу організаційну структуру й основні бізнес-процеси і діяльності будівельного підприємства, які перетворюють його в сучасну будівельно-інвестиційну проектно-орієнтовану та інжинірингово-девелоперську компанію. Втілення запропонованих інновацій сприятиме підвищенню якості, вчасності та ефективності зведення окремих об’єктів, має забезпечити покращення структури, функцій і результатів діяльності підприємств вітчизняного будівельного комплексу, а в кінцевому підсумку – повинне генерувати зростання конкурентоспроможності економіки та суспільного життя в нових умовах господарювання.

Ключові слова: будівельні та інвестиційні проекти, вдосконалення організаційної структури підприємства, стандарти управління проектами.

Introduction

Today, in the conditions of transition to new standards of management and development of competitive environment in Ukraine, before business, heads of the enterprises and other participants of construction, the main question is search and effective implementation of orders and construction investment projects. The successful experience of leading countries and companies of the world shows that such a task can be effectively implemented on the basis of project and multi-project management standards, and first of all-American standards PMBoK:2017 [1]. They have been successfully used for more than 40 years in the leading construction and investment companies of the world, and since 2017 their sixth version has been operating. Therefore, the attraction of advanced international experience to Ukraine and the formation of its own organizational innovations-standards of project and corporate management for doing business in the construction sector-is an urgent task that has scientific novelty and practical value for both the construction industry and the development of other spheres of the domestic economy.

Recent studies and publications analysis

In addition to the above-mentioned American project management standards PMBoK:2017, Japanese project management systems P2M, European requirements for the competence of project management specialists IPMA Competence Baseline (ICB) and other standards are also widely used in the world [1, 2]. Such well-known Ukrainian and foreign scientists as: S. Bushuev, V. Voropaev, Yu. Zabrodin, V. Zarenko, N. Ilyin, O. Mikhaylichenko, D. Pinto, M. Razu, V. Shapiro, H. Zachenko made a personal contribution to the development of the theory and practice of project management. [3-8]. But for successful standards application of project management in difficult conditions of Ukraine economy transformation and its transition to the world standards of managing there is a need for further theoretical researches development and development of more perfect practical recommendations on processes management of development and realization "turnkey" construction investment projects taking into account domestic realities of business.

The main purpose of this work is to highlight the theoretical research results and practical tools development for the application of organizational innovations and international experience of project management in the construction enterprises activities of Ukraine while ensuring their competitiveness by switching to international management standards.

Main material and results

The conducted research enabled to build a system model of construction investment project management (CIP). It is shown in Fig. 1. Considering this model, it is advisable to emphasize that in the project planning (investment) formation documentation, as well as standards for managing the processes of "turnkey" project implementation throughout its life cycle, it is necessary to clearly represent such key parameters (systems, subsystems, processes and elements) of the CIP:

1. The composition of the main participants and stakeholders, their powers and status.
2. Structure, amount, functions and professional skills of the project team.
3. The structure and duration of the main phases, periods, stages and complexes of the project.
4. The essence of the basic processes of project management and their maximum integration into a single project management process (MP) throughout its life cycle (in terms of implementation "turnkey").
5. The essence of the main functions (standards) of project management, which should be integrated with each other and cover the entire life cycle of the project.
6. Time and other parameters of project planning that determine the structure and quality of plans, and ultimately-the effectiveness and competitiveness of the project "turnkey".

In the activities of modern construction organizations (enterprises and companies) need to form two types of project management:

1. At the organization of separate objects construction (that is realization "turnkey" of separate construction investment projects), generally, standards of project management are applied, are resulted on Fig. 1 and in table. 1. With a variety of projects, in general, these standards are repeated considering the specific content and conditions of implementation of CIP.
2. On the other hand, a construction organization can simultaneously implement a number of different and unrelated) CIP, which form a portfolio of projects of this organization. For the organization of successful activity of such enterprise (company) the basis of its corporate management should be strategic and multiproject management of portfolio of CIP and the enterprise production program (which, as it is known, is developed for one – two years of the enterprise activity). At the same time, multi-project management should become a new (innovative) basis not only for production, but also for other types of company management.

To effectively address the above two project-oriented transformation areas and progress of Ukrainian construction enterprises in a competitive environment, it must be developed and implemented a number of organizational and process innovations in the system of their future activities. These innovations have been considered.

1. To solve the first of the above-mentioned tasks on the construction organization of individual facilities (implementation of "turnkey" individual projects of CIP) it is recommended in the process of preparation for construction to develop their own (individual) professional activity standards in project management, the recommended list of which is given in table. 1. At the same time, each individual standard (management procedure) should define:

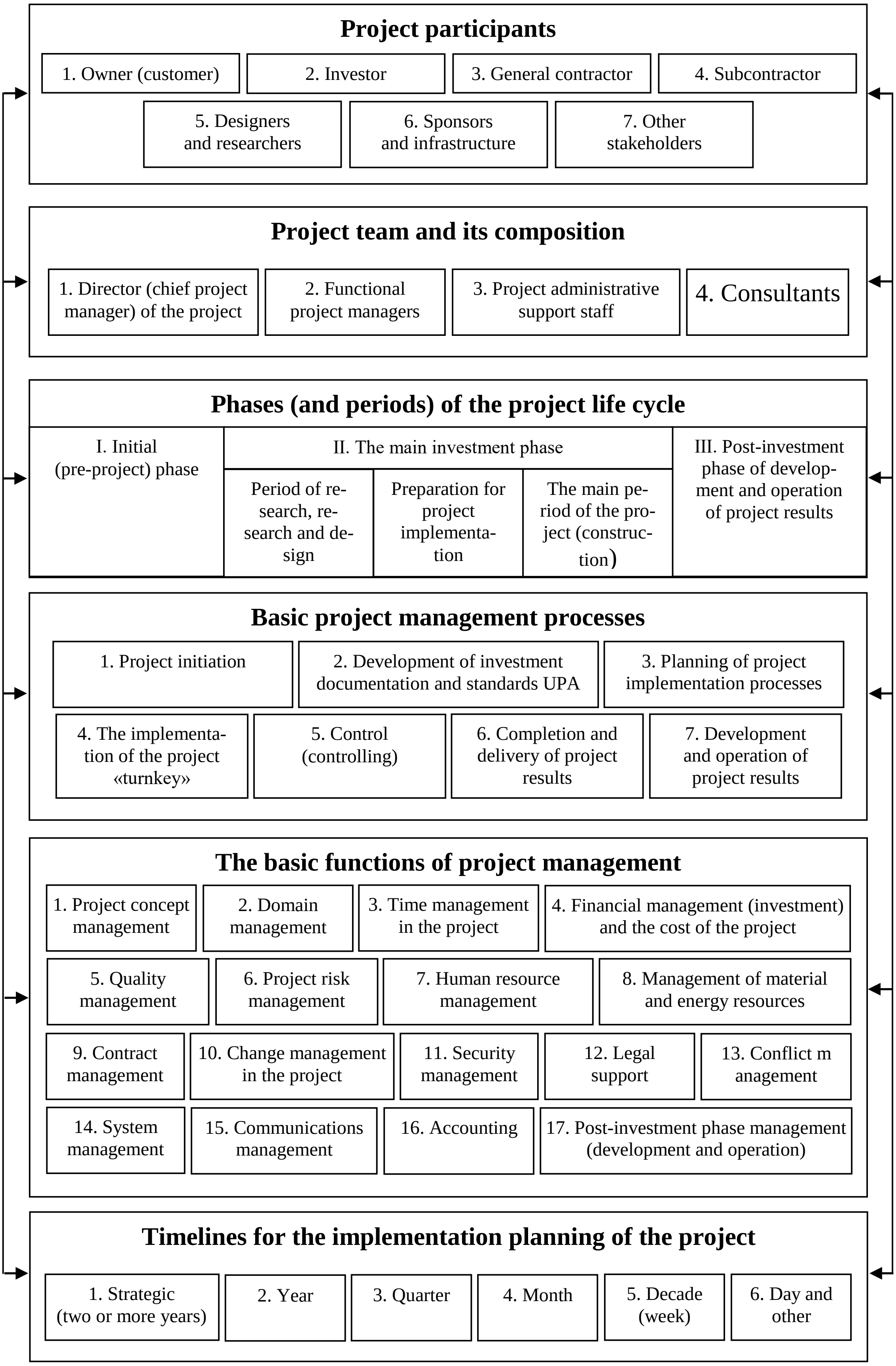

Figure 1 – System model of construction investment project management (investigated on the basis of [8])

Table 1 – List of modern professional management standards of construction projects and programs for the enterprises development

Organizational process	Name of design standards
1	*2*
1. Provision process integrations professional activities and project management	1.1. Procedure for project charter formation
	1.2. Development of a preliminary scope of work
	1.3. Formation of project management plans
	1.4. Project management guidelines and common standards
	1.5. Order of works monitoring and coordination
	1.6. General requirements for project change management
	1.7. Closing contracts and the project as a whole
2. Organization of project content management processes	2.1. Content planning
	2.2. The definition of a content entity
	2.3. Creating a hierarchical work structure
	2.4. Organization of control over the project content implementation
3. Management calendar terms of the project implementation	3.1. Definition of the list of works
	3.2. The definition of interdependence and the structure of the work
	3.3. Identifying and evaluating the resources needed
	3.4. Assessment of labor intensity and duration of work
	3.5. Development and optimization of calendar and resource plans
	3.6. Implementation, operational management and updating of plans
4. Provision cost effectiveness in the project	4.1. General requirements for determining the cost of work and costs
	4.2. Development of estimates, budget and financial plans
	4.3. Cost management and efficiency
5. Provision project quality	5.1. General requirements regarding the quality of the project
	5.2. Quality planning of the project and its results
	5.3. Organization of the quality assurance process
	5.4. Organization of quality control
6. Personnel organization and human resource management systems in the project	6.1. Human resource planning
	6.2. Organizational planning of staff performance standards
	6.3. The order of staff recruitment (project team)
	6.4. The development of the project team
	6.5. Team and human resource management
7. Management communications project's	7.1. Defining the characteristics and structure of communications
	7.2. Planning the structure and functions of communications
	7.3. Information support
	7.4. The content and procedure of reporting on the project progress and the obtained results
	7.5. Managing project participants
8. Project risk assessment and management	8.1. Planning of the risk management system
	8.2. The procedure for risk identification
	8.3. Qualitative risk analysis
	8.4. Quantitative risk analysis
	8.5. Risk response planning
	8.6. Project risk monitoring and management
9. Management deliveries in the project	9.1. Procurement and supply planning
	9.2. Holding of tenders and conclusion of contracts
	9.3. Collection of information about suppliers and terms of delivery
	9.4. Selection of suppliers and terms of delivery
	9.5. Contract administration
	9.6. Closing of contracts
10. Development and use of professional activity and management standards of specific functions of construction investment projects and industrial enterprises programs development	10.1. Pre-design work
	10.2.Feasibility study (business planning)
	10.3. Organization and management of special complexes of substructure, construction and other works, as well as their participants
	10.4. Organizational and technical preparation of construction
	10.5. Safety, health and environmental protection
	10.6. Organization of delivery of completed construction projects
	10.7. The organization and operation of the facilities
	10.8. Development of facilities
	10.9. Else

– the content and sequence of works performed according to this standard;- timing and frequency of the management procedure;
– input guidance information and incoming resources;
– mechanisms and tools that are needed to implement this procedure;
– the procedure results;
– system of responsibility distribution among executors of procedure (standard) and on its results as a whole;
– system of procedure efficiency indicators (application of the standard) of management.

The key requirement for the table. 1 the list of standards is that they should be integrated with each other and with the corporate management standards of the construction organization, and cover the entire life cycle of the project in which they are applied.

2. Considering innovative and organizational renewal of structure and corporate management functions of the construction organization which simultaneously realizes a number of various CIP (that is the second group of updating of activity of the domestic enterprises), it is necessary to consider the following remarks and offers.

During the years of Ukraine's independence there was a rapid decentralization of large and hierarchically constructed organizational structures in all sectors of the economy, including construction. At the same time, another economic integration process of small and weak construction enterprises into new organizational structures for Ukraine, such as firms, companies, corporations and holdings, began. In the conditions of competition development such integration is a natural process of mutual adaptation, expansion of economic, production, organizational and investment cooperation, and also own interests protection of each managing subjects. An example of successful cooperation is the creation of a powerful and modern holding company "Kyivgorstroy", which brings together dozens of construction, production, commercial and other enterprises and organizations, thousands of employees along with all their property. Due to this, "Kyivgorstroy" successfully works not only in the city of Kyiv and the nearby region, but also in other regions of Ukraine, qualitatively and in a timely manner performing various types of construction "turnkey" and CMP complexes.

That is, in recent years in the construction industry of Ukraine there are not only decentralization and transformation of large state associations (ministries, factories, trusts, construction enterprises) in new organizational forms of primary (small) construction organizations with different forms of ownership (mainly joint-stock and private type), but also began to create modern corporate structures (holdings, companies, financial and industrial groups, corporations, alliances, etc.), which seek to implement in full "turnkey" various construction investment projects with mandatory involvement, effective, quality and timely development of investments. The subjects of investment activity, which include all participants in the construction, in developed countries (and increasingly in Ukraine) can cover several functions in the project implementation process: the developer and the customer; the customer-designer-Builder-user, etc. It should be noted that in the advanced countries and companies the world investment direction of the construction project, which is a set of interrelated stages of one whole investment process in construction projects today requires the implementation of: scoping (or pre-investment) stage of a concept project proof, investment intentions effectiveness, analyzing their alternatives and selecting the most appropriate identify sources of funding, etc.; surveying, design and the necessary research (project engineering); construction and installation works; commissioning, provision of a full range of engineering and development services during operation to ensure the required size and payback period of investments.

For successful use of the recommendations and innovations considered in this paper, in order to improve the performance and results of the modern project-oriented construction and investment enterprise, it is necessary to carry out its organizational modernization and create a new structure in the form of a future powerful holding company with engineering development and project-oriented subsystems and functions. This organizational structure should take into account the results of analysis and programming necessary for business activities of strategic economic zones (SEZ), the implementation of "turnkey" projects and programs, as well as other decisions regarding the development of the enterprise.

Two levels of management should be created in the future company: corporate management center of the whole company (corporate office); management centers of individual strategic economic zones. At the same time, each center will be determined by its functional and managerial characteristics and status: cost center and profit center. In addition, each center will be separated into a legally personified, but economically dependent, division in the form of a subsidiary legal entity, which is part of the entire holding. Should be implemented the rationale and decided that would consider the modern view that holding a legal person (i.e. a person who manages other accountable persons) it is better to create on the basis of a corporate control center, but only on the basis of organizational unit, which brings together executives at the highest level (top management). In accordance with this, the entire management center of the new company should be merged into one legal entity. To manage strategic economic zones, it is advisable to create six legal entities that must manage subsidiaries of their strategic economic zones (SEZ). The Central element of the management system at the SEZ level will be a project-oriented modern construction and development-engineering company, which will organize the activities of other business units within their key functions and specific projects (Fig. 2). In order to build and offer a modern (more perfect) organizational and production structure of a powerful construction company or enterprise (with the number of employees over 400-600 people), it is first necessary to determine its new essence, main features, tasks, business processes, which are characteristic of the leading companies of the world today.

The conducted research enabled to develop a more perfect organizational structure of a new type of construction enterprise for Ukraine a modern project and program-oriented construction and investment company. Its activity is based on the world standards of strategic, multi-project, project, corporate and production management, engineering and development. The structure of this company is shown in Fig.2. Its key characteristics and organizational innovations are:

1) integration of the best domestic and foreign experience in the organization and management of construction of various facilities and similar enterprises activities;

2) combination and inclusion in the new organizational structure of this company of modern divisions (for example, such as departments of perspective directions of development of business processes and professional project management and engineering (PMO office) and the like, which work for a single result - ensuring the competitiveness of the construction company and its products - completed construction projects. These divisions should become innovative centers of attraction and use of the best world experience and standards of management of unique projects and programs, and also provide use of engineering and development;

3) that all projects (CIP), main construction sites, auxiliary and other divisions, commercial structures should be transformed into strategic business units (SEZ), which have greater administrative, economic and economic independence.

4) continuous development (in parallel with the Department of professional project management and engineering) of other planning and ensuring the progress departments of promising business activities areas of the company, its development, marketing, corporate service, etc;

5) other innovations.

As evidenced by the experience of leading construction and investment companies in the world, the implementation of the domestic enterprises activities proposed by this work innovations and practical recommendations can contribute not only to the involvement in business processes of advanced organization and management standards of construction in Ukraine, but also enables to fully use the latest achievements of science and technology, ensures the growth of economic entities competitiveness. In this case it becomes possible to obtain:

– reducing the complexity and duration of construction (in the implementation of "turnkey" projects) by 12-20%;
– reducing the cost of the entire project by 10-15% or more;
– increase in times of quality and the project results implementation;
– improvement of technologies, organization forms and working conditions;
– other useful effects.

Conclutions

The authors believe that the introduction of the proposed innovations and practical recommendations not only improve the quality and modernization of the structure, functions and performance of domestic construction enterprises, but also ensure their competitiveness through the use of international management standards, the embodiment of the best experience, the latest achievements of science, technology and high technologies.

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MONITORING ACTIVITY OF THE CO_2 EMISSION OBJECTS SYSTEM COMPONENTS IN FORMATION OF THE AIR MASS IN INDIVIDUAL CLOSED PREMISES

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According to the developed methodology of the experiment and with the use of electronic devices for measuring the content of carbon dioxide (CO_2) in the air and using a set of special laboratory tools, the following were studied on the example of a school audience: a) the variety of interactions of the external environment and the subsystem - classroom air; b) the activity of constituent internal objects (present students, plants, absorbent surfaces of structural elements and the interior) in release and absorption of CO_2, depending on conditions and variation in the action of dominant factors; c) their manifestation and influence on forming the composition and content of the air mass, and therefore the internal microclimate of the classroom, which quality directly affects the health and performance of the present people.

Keywords: room air, CO_2 emission sources, CO_2 concentration, adaptive microclimate support systems

МОНІТОРИНГ АКТИВНОСТІ КОМПОНЕНТІВ СИСТЕМИ ОБ’ЄКТІВ ЕМІСІЇ CO_2 ПРИ ФОРМУВАННІ ПОВІТРЯНОЇ МАСИ ОКРЕМИХ ПРИМІЩЕНЬ

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Дослідження спрямоване на моніторинг і оцінювання внеску, з’ясування співвідношення обємів кожного окремого джерела емісії CO_2 у загальному масо-газо прояві та пошук інноваційних рішень для створення адаптивних систем стабілізації і підтримування мікроклімату в аналогічних об’єктах. Підібрані оптимальні режими функціонування вентиляційних засобів. Встановлено вклад CO_2 від життєдіяльності рослин у повітря замкнутої аудиторії у стадіях «фотосинтезу» і «затемнення»; їх активність під впливом дії низки факторів. Виявлена доцільність використання поризованих алюмосилікатних адсорбуючих матеріалів в каналах припливного і рециркуляційного повітря адаптивних систем забезпечення мікроклімату зі змінною витратою повітря і періодичною роботою, з циклічним тепловим реверсом в наслідок прояву ними повторюваності і відтворюваності адсорбуючої здатності у релаксаційних перетвореннях, вузькості гістерезисних залежностей, можливості легкого в енергетичному відношенні реверсуванні напряму перетворень (навіть з використанням малопотужних засобів відновлювальних джерел енергії, НВЧ обробкою), що забезпечить надійність, стабільність роботи і відтворюваність технічних характеристик впроваджених систем

Ключові слова: повітря приміщення, джерела емісії CO_2, концентрація CO_2, адаптивні системи підтримування мікроклімату

Introduction

This work is devoted to the academic problem of continuously searching ways to maintain balance and a compromise between the air quality in the rooms and energy efficiency of the technical systems that provide it.

Indoor air, mainly due to ventilation, can be ideally provided with CO_2 content like that in the supply air. However, this is only true if there are no CO_2 sources or sinks in the room. The main CO_2 source in a room is usually people. It`s content depends on the people amount, their job characteristics and the ventilation work intensity in the room. In the closed premises environment, some amount of CO_2 also reacts to humidity in atmospheric air with formation of carbonic acid. The room air composition is influenced by the present plants life processes, which, as a result of photosynthesis from carbon dioxide and water form oxygen under the solar radiation action in the chlorophyll presence as a catalyst, as well as organic compounds (mainly carbohydrates),. Carbon dioxide absorbers are adsorbing porous surfaces of brickwork, partition and another parts of interior.

Review of the research sources and publications

Allocations formed mainly as a result of active people`s activity inside the building, are quantified in compliance with the international standard requirements [1], and, being a variable value, necessitate the introduction of automatic air flow rate change in the premises, regulation of its composition, rational use of electricity and, ultimately, the ability to reduce operating costs. And since indoors carbon dioxide level is one of the main criteria for the people presence in room, it is based on the design of adaptive ventilation systems with CO_2 control (DCV) [2].

The systems (DCVs) cover all of the above problems and permit them to be solved totally. Their implementation requires a thorough and proper consideration of the behavior, activity and each component's role in the system involved in the CO_2 emission processes or participating in its redistribution. The existing generalized information in these fields mainly relates to time-limited study [3], often limited in scope, for example, novelties could not spread and pass efficiency testing (one of them is implementation of forming composite photocatalytically active materials methods based on TiO_2 in integrated air purification systems created by aggregate principle [4-7], etc.) or obtained on the basis of some leading manufacturers' closed research protocols.

The authors' research performed on the example of air study in a classroom of an educational institution was aimed at obtaining a comprehensive picture, trend detection, identifying objective features and regularities of heat and mass transfer transformations in such thermodynamically complex interrelated objects.

Definition of unsolved aspects of the problem

The analysis of the experience gained [8-11] shows that the academic problem solution is only possible with the use of integrated inflow air supply self-adjusting systems [12, 13] with the additional inflow and exhaust ducts arrangement with cut-off valves, simultaneous air flows purification and implementation of continuous monitoring systems and object management. [3].

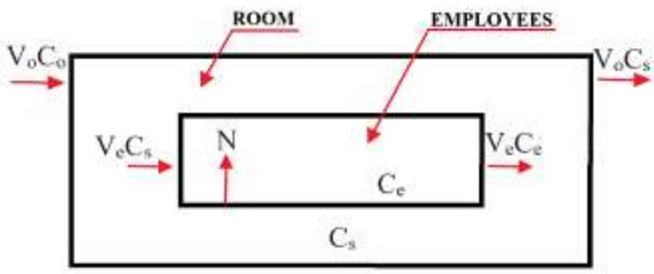
In such systems, the incoming inlet air volume and its quality are determined by the gas composition indices, the indoor circulation mode [1], which primarily determine the functioning efficiency of continuously operating ventilation system. It is clear that the real load on the heating, ventilation and air conditioning systems inside the building differ in different parts of the building, changing over time.

And developing specific projects of the above systems is accompanied by the development of models corresponding to the composition and functioning features of the respective objects (as an example, see Fig. 1 [14]), to the technical means used to reproduce acceptable operation modes. The parameters and modes choice of technical equipment operation is carried out in accordance with the regulatory documents requirements [8-11], which have now become national standards in all countries, for typical service areas types – 4 categories of indoor air quality: from IDA 1 – high quality to IDA 4 – low quality.

There are two ways to determine the required air exchange indoors [8-11] – based on specific air exchange rates and based on the permissible pollutants concentrations calculating. In the first case, the required air quality is obtained due to supplying a certain outside air amount to the room, depending on the room purpose and its operation mode; in the second case, depending on the pollutants size and their characteristics in the room.

From the analysis of the normative requirements for the indoor air quality according to the above mentioned standards, it is also evident that in the first category rooms, both with high and low pollution, the ventilation with untreated outside air is problematic. The way out is to use carbon dioxide adsorbers in ambient or recirculating air flows.

In most cases, the intake ducts of modern systems do not provide for gaseous impurities removal, but only filters for mechanical impurities removal are used; in special cases air purification is implemented, mainly with activated charcoal [15, 16]. In the existing developments variants, due to the complexity and versatility of the studied processes, the lack of properly formed approaches and generalizations in their understanding, there is insufficient thoroughness in justifying the choice of sorbent materials (regarding their functionality degree, the characteristics set choice, technical capabilities and application fields); the concentration effect of gaseous impurities of organic and inorganic nature in the fresh-air intake places is not taken into account [17], etc.



Calculation of the inflow air flow

$$V_o = k / (C_s - C_o)$$

where:
V_o – the volume of inflow air per unit per person,
V_e – breathing rate,
k – the CO_2 amount emitted by a person per time unit,
C_e – the CO_2 concentration in the exhaled air,
C_s – the indoor CO_2 concentration,
C_o – the CO_2 concentration in the incoming air.

Figure 1 – Two-chamber system model for calculating inlet air flow [14]

An analytical signal for regulating air exchange in such adaptive systems is the carbon dioxide concentration value measured by the special electronic sensors included into the raw information collection system of the computer control center (управляючого комплексу). Their physical and technical specifications, reliability, efficiency of the implemented measurement methodology determine the system's success as a whole.

Creating efficient adaptive systems for maintaining air quality in rooms with a large number of people requires a clear understanding the such objects specifics, considering the number and behavior peculiarities of their present "active" participating constituents, as well as the objective processes taking place in them. It is also necessary to realize the fact that there was no possibility to use universal methods and means of solving the problem due to a large number and individuality of projects and their existing options specificity; due to the limited access to necessary information on their design and technical specifications and opportunities due to commercialization; different level of engineering support and possibilities of a project developers' and real consumers, etc. The available information and certain own developments in this field, trends in the advanced electronic means implementation (with short exposure time, increased accuracy, reliability) and control systems have permitted the authors to continue research on this difficult but extremely relevant subject.

Problem statement

The research problem was to find out the behavior of constituent components' individual segments in the system of the gas exchange participants: CO_2 emission-absorption processes of different nature and mechanisms in real conditions and in the conditions that imitate different states and actual stay stages of the latter in the academic classrooms during the academic period at fixed values of indicators, uniquely the environment state and the building itself (on the example of a particular an educational institution classroom). The obtained results form the basis of setting tasks and making innovative decisions in designing adaptive complexes for maintaining the building's microclimate quality according to modern technologies.

According to the developed step-by-step methodology it is necessary to:

– make a real, objective view of the CO_2 mass change in the classroom in the course of educational process;
– identify the internal CO_2 sources and objects of CO_2 emission and absorption, and to evaluate their nature;
– build a model research system adequate to real processes. To develop a methodology for studying its constituent components;
– take the empirical findings obtained as a basis for the requirements in formulating the tasks for designing and making innovative decisions at appropriate stages during the modern systems designing for maintaining the microclimate quality in the classrooms.

Main material

Research methods and tools

In order to solve the problem under consideration and to develop an adequate model, a comprehensive sequential methodology was suggested to study the mutual behavior of constituent components present in the indoor air system; the study of their mutual influence degree and activity in varying the staying conditions; the active objects actions on the behavior and changes of the closed system as a whole under the conditions that imitate the real transformations. Each of these fields has both its own academic and applied value.

The study object is the school classroom`s air which should be regarded as a complicated interconnected set of interacting environment and the studied room air subsystems, between which (to the extent of its openness and isolation, in this case, heat-mass transfer is performed mainly due to natural or supply-and-exhaust ventilation) the mean value of the air composition indoor CO_2 content is determined.

The objects of the present study are the CO_2 inflow-absorption processes in the closed classroom air and their influence on the students' physical condition; search for methods to reduce the increased CO_2 concentration.

Field indoor air studies on the CO_2 content during the education process were carried out directly in the classroom. For the study an ordinary school was selected in a historic building in the center of Poltava, surrounded by commercial and residential buildings, located in the street with lively traffic and a parking place opposite. The classroom was located in the 1st floor ($S = 86{,}4\ m^2$, $V = 259{,}2\ m^3$), and had 3 plastic double-chamber windows ($6{,}33\ m^2$). doors ($2{,}84\ m^2$), exterior brick walls (0,8 m thick, $S_{outside} = 50{,}78\ m^2$, waterproof outside;

grounded and painted inside), 3 heating appliances (thermal energy supplied from other sources (available computers, lightning systems) can be neglected), 3 convection ventilation ducts.

Air exchange modes, which ensure the required classroom air quality and are created by the ventilation system, were worked out in a specially designed flow chamber V = 9 m^3, equipped with a supply and exhaust system (gate valve; ducts, *Vents Ø125 mm*; *Domovent 125 CT* fan), air flow rate controller *Vents RS-2 N*, *AM-ANEMOMETER 4204* anemometer, a system for dosing and measuring CO_2 concentration.

Step by step constituent subprocesses empirical study was carried out by static method in the developed sealed chamber-container (Fig. 2), the equipped with a dismantable hinged holder for sample of studied objects, an internal evaporator dispenser and an external reactor for small and burst CO_2 dose-injections by means of a compressor, fan-mixer for internal gas environment, additional internal heater, electronic CO_2 meter, hinged studied "passive" absorber plates and photo catalytically active plates-samples, a lighting system.

The study of the processes under study included a sample tests series under appropriate simulation conditions, each beginning with the laboratory chamber ventilation and the equilibrium conditions restoration.

a) b) c)

Figure 2 – Chamber for model study of system constituent components participation in the studied processes:

a) external view; b) to understand the adsorbent materials behavior under static conditions; c) for the photos catalytically active components study

The applied static experiment methodology has significant advantage: the sampling and measurements significantly reduces the human impact on the measurement process, which increases the results reproducibility and reduces the measurement errors. It enables more objective assessment of the certain processes effects and factors, influencing the gases generation and emission in the activity zones, as well as assessment of flows from small objects.

The materials selection and development of laboratory units design was performed considering requirements to minimizing their impact on the studied transformations course.

To measure CO_2 concentration in the studied environment, *AZ 7755* (AZ Instrument Corp., Taiwan) portable multifunctional electronic gas analyzer was used, which enables simultaneous measurement of temperature, relative humidity and can be attached to an external interface.

The one-probe system for fixing CO_2 concentration enables to measure the averaged total, contribution simultaneously by all the present active emission agents of this gas in the room. In order to clarify the contribution action of individual components, it is necessary to maximally limit the environment influence on the internal heat and mass transformations (by sealing and thermal insulation of the laboratory chamber), more careful consideration of internal sub-processes, existing phenomena (to better understand their mechanisms); introducing simplifications into the proposed thermodynamic model (based on analogies to real study objects); planning the experiment and considering the dominant influence factors with fixing invariability and adequacy of their relevant course conditions; sequential staged study of the causal dependencies.

In the biological experiment with involvement of plants, the following indoor plants were used belonging to *Chlorophytum comosum* species of the same age and close development stage; with the similar growth conditions (humus quality, humidity, temperature); with a known vegetative leaf cover area; planted in pots with a sealed bottom.

The following illuminators were used: a fluorescent low pressure ionizing radiation lamp (simulating daylight) with 8 W of power and a bactericidal lamp with same power with the wavelength ~ 254 nm. The illumination nature was changed by the use of the appropriate illuminator type (spectral composition), the intensity was altered by the number of sources used, the distance to the samples, the shape and area of the latter.

Plates, made of aluminosilicate clays by a technology similar to the ordinary building bricks manufacturing were used as adsorbents.

The required calculated CO_2 volume used to simulate the external conditions, was created by a chemical equivalent interaction in aqueous sodium bicarbonate and hydrochloric acid solutions, taken in the appropriate amounts.

Results of the study and their discussion

The work was aimed at characterizing the system under study, development of methodology for study and subprocesses consideration stages with determining the degree of dominance factors influence on the object under study.

Stage I. Air study in school classroom.

The air microclimate study in the classroom was carried out in October 2019. The averaged data obtained during the school day, in a classroom with 32 pupils with an mean air volume of 8,1 m^3/person, in the fixed conditions in two stay modes (see Fig. 3, 4) clearly demonstrate the ventilated room's advantages (opening windows during breaks between lessons) in carbon dioxide content.

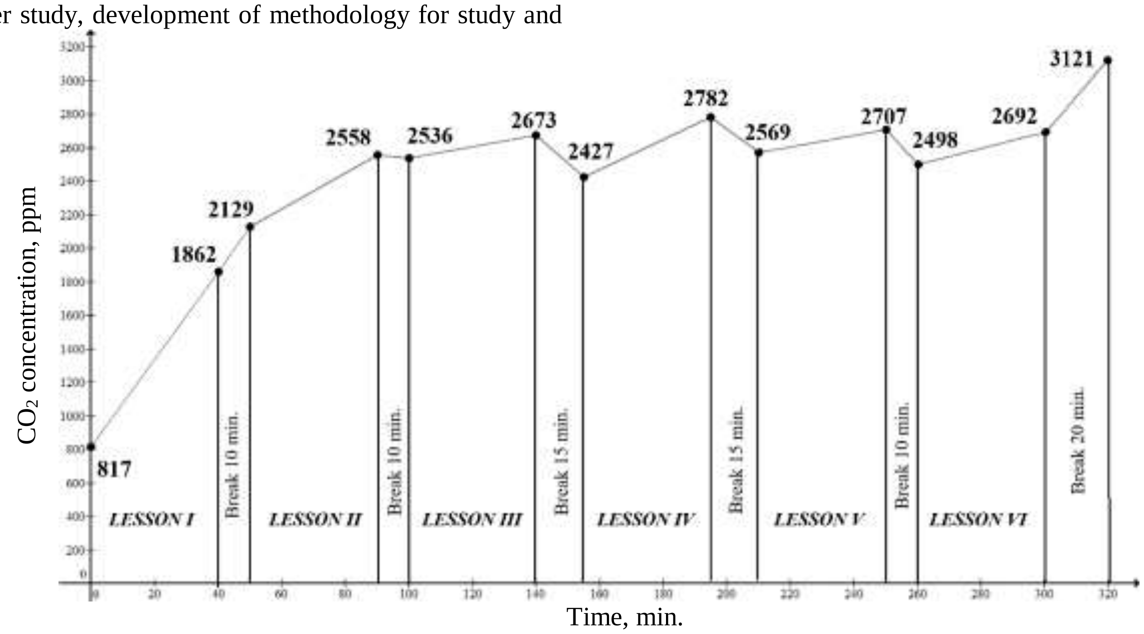
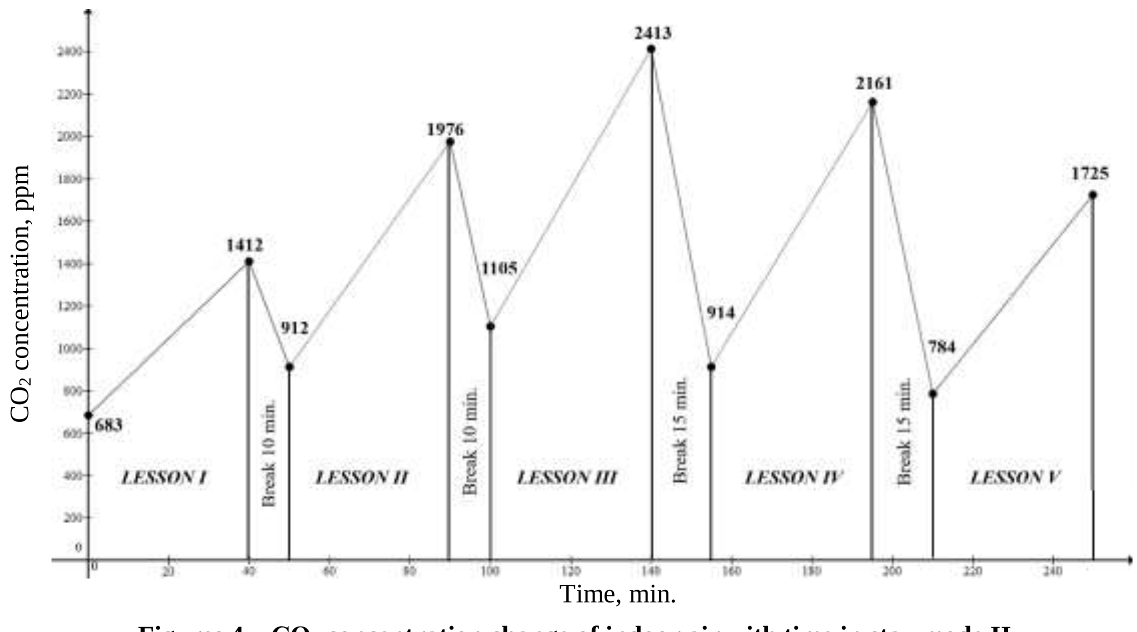


Figure 3 – CO_2 concentration change of indoor air with time in stay mode I (without airing a closed classroom V = 259,2 m^3 during breaks between lessons at fixed values of t = 19,3°C; CO_2 concentration = 502 ppm; r = 49%)



Figures 4 – CO_2 concentration change of indoor air with time in stay mode II (with airing a classroom V = 259,2 m^3 during breaks between lessons at fixed values of t = 19,3°C; CO_2 concentration = 502 ppm; r = 49%)

In *mode I* in an unventilated room during the study time, the background ventilation rate is k = 0,1 h^{-1}. In such circumstances, the CO_2 concentration will be 1900 ppm already at the end of the first lesson (contributes to the increase in humidity and temperature) far exceeding the recommended 1000 ppm. By the end of all lessons, the concentration will increase by 3 times (3365 ppm) – this value is significantly unfavorable for the efficient pupils' work in the classroom.

When ventilating in *mode II* (during each break, windows and doors open, all pupils leave the classroom) the air environment condition in the classroom improves compared to the previous version. However, the CO_2 concentration remains high at the end of the classes, it significantly exceeds the recommended values. This method of stabilizing CO_2 concentration becomes unfavorable in terms of heat losses, especially in winter (when $t_{outside}$ is ~ - 18 ° C), it can cause diseases of the modern people.

The performed field study really clarifies the dynamics and changes range in the CO_2 concentration during the studied processes, enables modeling the latter, predicting the ways to optimize solutions when constructing engineering support systems for providing quality microclimate in classrooms.

Stage II.. Indoor air quality and ventilation

This stage was aimed to determine optimal ventilation system's efficient operation mode with different ways of organizing air exchange indoors, the air quality indicator being the CO_2 content.

When designing ventilation systems and calculating heat loads, the required air exchange is specified in the design documentation in compliance with the requirements and regulatory documents recommendations [10, 11]. When designing and operating, isolate or eliminate or at least reduce emissions of the major harmful sources. Additional air purification systems are also used to improve the indoor air quality.

At minimum, the ventilation air exchange must be sufficient to disperse the biological human`s discharge. In addition, the air exchange increases by the discharge amount from the building itself and its engineering systems. According to table 3 [11], the recommended total specific values (in terms of one person) of ventilation air exchange for classrooms (category I premises with very small discharge) is 19,8 m^3/h. The air exchange rate was found to be 2,5 h^{-1}, the air flow rate was ~ 0,45 m/s.

During the experiments to ensure the estimated inlet air flow in the various implemented modes (see graphs in Fig. 5), carried out by regulating and controlling the air flow rate and with the parallel addition of appropriate CO_2 doses, simulating gas discharges during the educational process, and measuring its concentration changes within the set time to stabilize the CO_2 concentration (thus, at k = 2,5 h^{-1} – after ~ 23 min.). Dependencies are asymptotic. Thus, the proper sanitary and hygienic conditions in the studied classrooms are ensured.

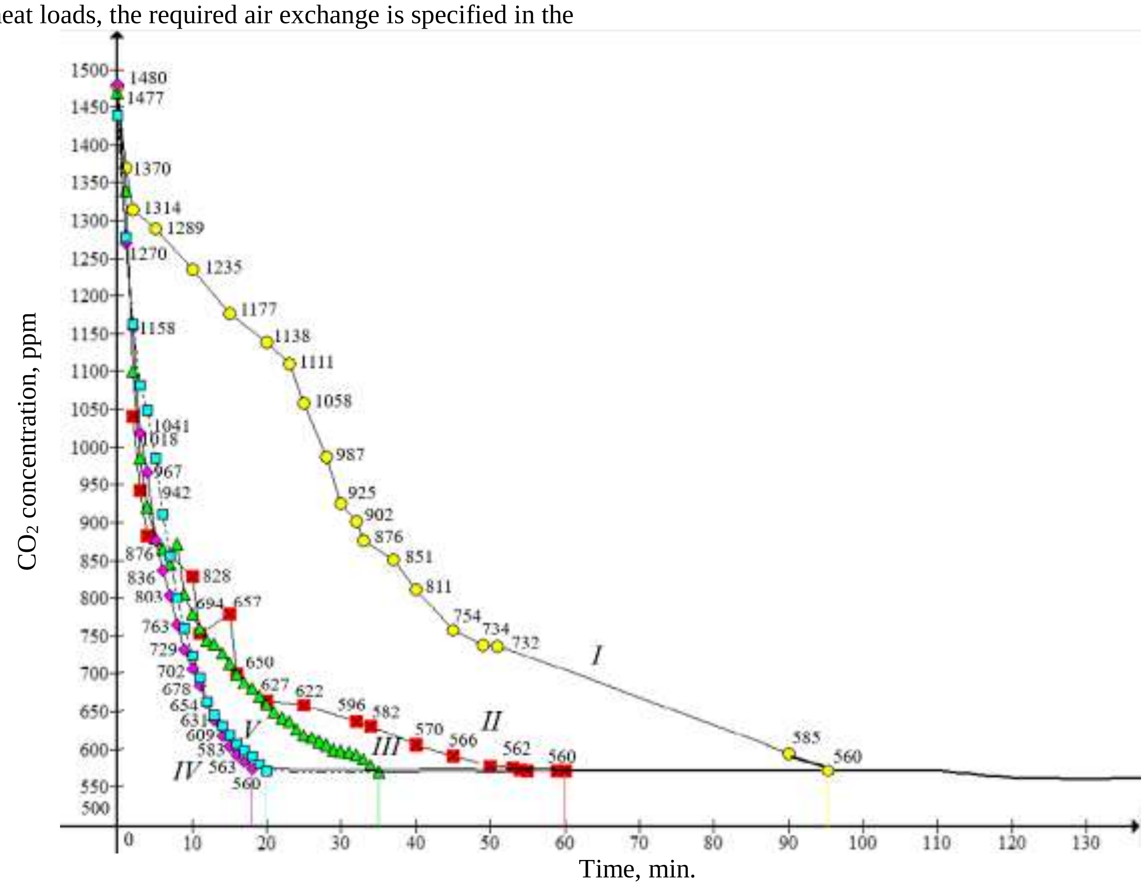


Figure 5 – The nature of CO_2 concentration changes in the laboratory chamber over time during the tests in different modes of air exchange and burst injection of same initial carbon dioxide doses I – k= 1,00 h^{-1}; II - k= 1,50 h^{-1}; III - k= 2,00 h^{-1}; IV - k= 2,75 h^{-1}; V - k= 3,65 h^{-1}

Stage III. CO_2 contribution by the modern plants in the closed classroom air

The study was carried out in a specially designed sealed chamber (Fig. 6) in order to clarify the intensity effect and spectral radiation composition on the plants photosynthetic activity (through the example of *Chlorophytum comosum*), in the conditions close to real ones. The plants behavior when illuminated (dimmed) by a fluorescent "daylight" lamp is clearly characterized by the empirical data in Figs. 7-8, Tables 1, 2, by the experiment conditions, by the calculated mean conditional specific trends (indices).

To obtain correct study results in the specified regimes at different imitating initial CO_2 doses (see Tables 1-2) the study was performed with the same plant; at constant temperature; while minimizing losses due to leakage and the absence of other potential object-participants. In the both experiments, the lower plant part was gas-insulated; the total active leaves surface area was ~ 3200 cm^2.

In Fig. 7 the highlighted color zone reflects the evolutionary changes nature in the CO_2 content in room air over time in the fixed range concentrations from its highest to the lowest values; the upper limit determines the time zone for the CO_2 content index values recovery in the classroom to its initial level in the first scenario, i. e. without ventilation (Fig. 1) with the flow up to 4000 ppm; the lower limit – in the second scenario (with ventilation) and the flow up to 1500 ppm.

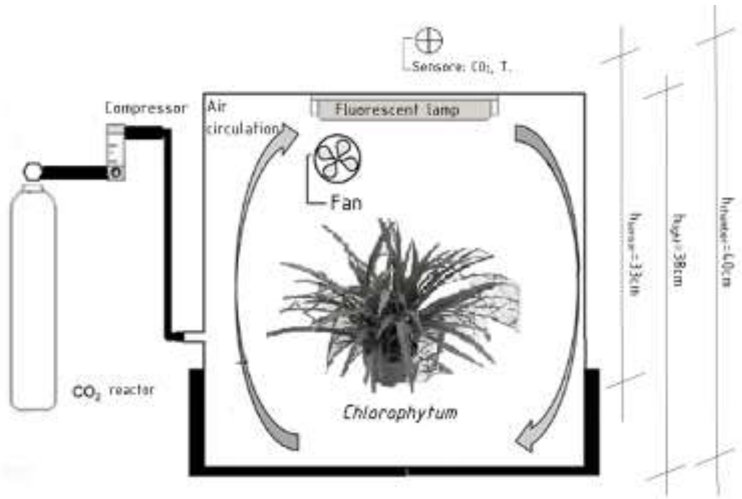


Figure 6 – Laboratory chamber for studying the plants activity in simulated conditions

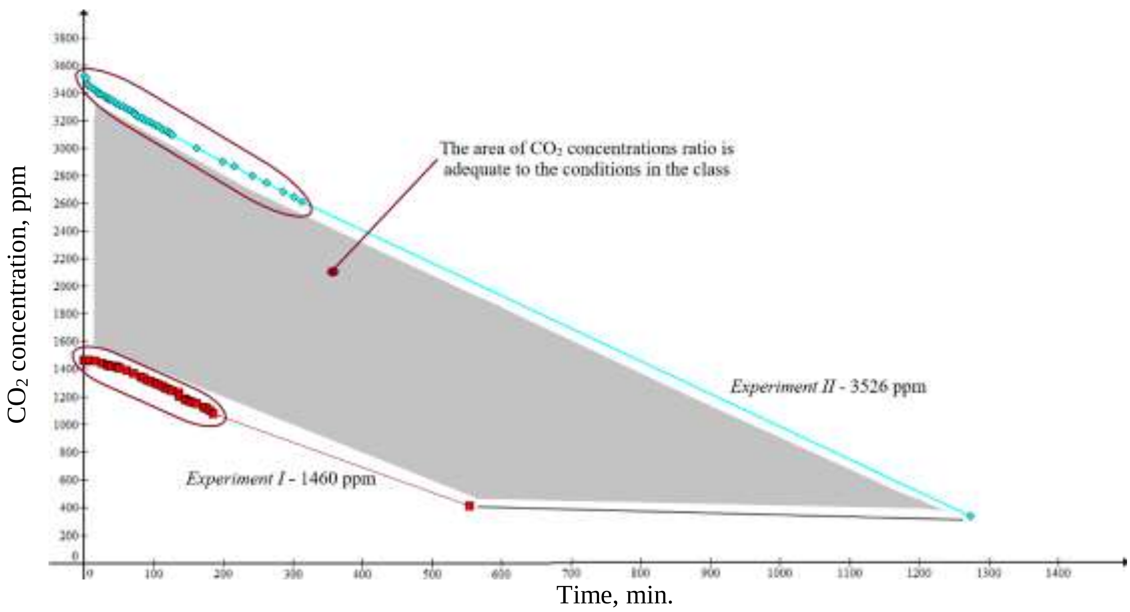


Figure 7 – Injected CO_2 burst dose influence on the restorative processes duration in the studied air volume of the plant in photosynthesis mode (under static conditions)

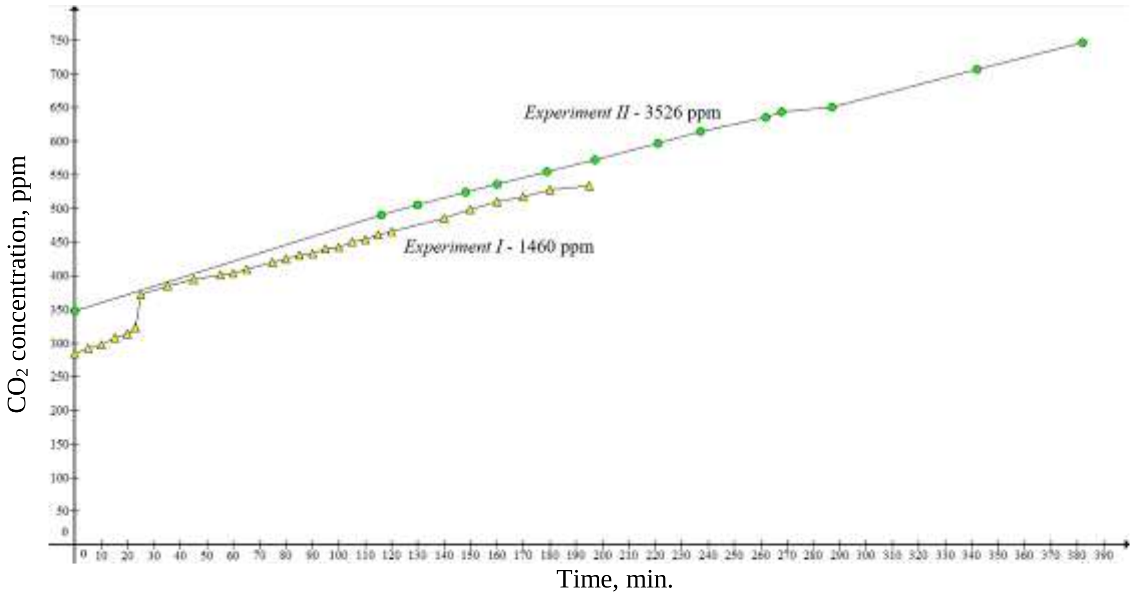


Figure 8 – CO_2 emission by the plant in the "darkening" stage for a long time after the photosynthesis period in experiments I, II (without changing other conditions)

Table 1 – The plant activity during daytime – artificial light

Experiment I	*Experiment II*
t = 19 ºC	
$S_{\Sigma\ act.}$ = 3200 cm^2	
Initial concentration CO_2 = 1460 ppm	Initial concentration CO_2 = 3526 ppm
τ_Σ = 185 min.	τ_Σ = 313 min.
ΔCO_2 = 395 ppm	ΔCO_2 = 911 ppm
$v_{aver.}$ = 3,29 ppm / min.	$v_{aver.}$ = 2,91 ppm / min.
$v_{aver.\ active.}$ = $1{,}02\cdot10^{-3}$ ppm / cm^2·min.	$v_{aver.\ active.}$ = $0{,}91\cdot10^{-3}$ ppm / cm^2·min.

Table 2 – The plant activity in the "eclipse" stage

Experiment I	*Experiment II*
t = 19 ºC	
$S_{\Sigma\ act.}$ = 3200 cm^2	
Initial concentration CO_2 = 1460 ppm	Initial concentration CO_2 = 3526 ppm
τ_Σ = 194 min.	τ_Σ = 382 min.
ΔCO_2 = 249 ppm	ΔCO_2 = 399 ppm
$v_{aver.}$ = 1,28 ppm / min.	$v_{aver.}$ = 1,04 ppm / min.
$v_{aver.\ active.}$ = $0{,}40\cdot10^{-3}$ ppm / cm^2·min.	$v_{aver.\ active.}$ = $0{,}33\cdot10^{-3}$ ppm / cm^2·min.

Oval highlighted segments characterize the CO_2 amounts used by a plant during its photosynthetic activity (reducing the CO_2 content in room air) over a working day (~ 5-6 hours) under the conditions imitating the largest and the smallest CO_2 discharge from the modern people.

The nature of such CO_2 use by the plant approaches to that proportional with time (this is indicated by the tangent slope angle of arrival to empirical concentration dependences on time). In close premises, CO_2 concentration (under other conditions unchanged) may be less than its content in the supply air. It enables implementation of conditional specific trends, characterizing the biological plant species behavior, (indices) (see Tables 1, 2), by which, to some extent, their activity in the studied processes can be roughly assessed under the appropriate conditions. It enables to approximately determine the required time interval to restore the level of CO_2 content to its initial values and the required amount of the above species plants. In experiments I, II with carbon dioxide concentration up to 4000 ppm, the conditional specific index and photosynthetic plant activity of the *Chlorophytum comosum* species plants are hardly changed.

When switching off the lighting (during dimming, under the same adequate conditions, with presence of the same plant), reverse CO_2 emission process occurs (see Fig. 8), but much slower (in this case, by 2,5 ÷ 3 times) compared to the activity at the photosynthetic stage. In experiments I and II (with different additionally introduced carbon dioxide doses into the working volume) it is found that from the moment of reaching the initial CO_2 concentration level the course of concentration dependences passes through close intermediate states, which testifies to similar mechanism of their transformations, their repeatability, reproducibility, and, therefore, some plant behavior stability over time under certain conditions and with close mean conditional index values of its activity.

The obtained data acquire value when using the similar biological properties forms in applied innovative solutions.

Previous studies [19-21] found that *Chlorophytum comosum* exhibits the highest photosynthetic activity at irradiation with wavelengths in areas with maxima of 440-445 nm (in the blue spectrum part required for vegetative development) and at 640-660 nm (in the red zone, that is necessary for all adult plants for reproductive development and for strengthening the root system).

Significant absorption spectral activity for the studied plant in the electromagnetic waves range, which is emitted by fluorescent daylight lamps, was established and characterized in the results of the above studies. Hard radiation (254 nm, of the same power – 8 W) of the bactericidal lamp revealed irreversible destructive effect on plant living cells, that was established due to the sharp increase of CO_2 concentration in the operation zone air after several hours of exposure.

The influencing factors on the CO_2 contribution by the modern plants in the air of the closed classroom were:

– plants species,
– their number,
– their development stage and their condition,
– lighting system (its intensity, spectrum) applied,
– carbon dioxide content in the surrounding atmosphere,
– favorable plants conditions (humidity, soil quality, environmental temperature, pot and root system openness, etc.),
– features of photometric factors influence,
– the active photosynthesizing surface area size and its condition,
– the ratio of contributions due to activity at the "photosynthesis" and "darkening" stages.

The original obtained data need detailed consideration and further study using more sophisticated methods and precision technical experimentation equipment.

Stage IV. Influence of artificial mineral aluminosilicate adsorbing materials present on the composition of room air mass.

This effect manifestation and its activity are due to the physical heterogeneous adsorption on the interphase surfaces: the room air – building elements porous surfaces.

According to modern concepts, physical adsorption is caused by dispersion, orientation and deformation interaction forces. The gas medium molecule energy interaction with the solid adsorbent surface, caused by intermolecular forces, decreases proportionally ~ $1/r^3$ (r is the distance from the molecule center to the phase disengagement boundary). At $r = 3r_o$ (r_o is the radius of the molecule) the adsorption potential decreases up to 3÷4% of its value near the surface. This leads to the conclusion that the gas is adsorbed in the monomolecular layer form on the energetically inhomogeneous adsorbent surface. The molecule transition from one site to another may be related to overcoming some energy barrier (localized adsorption). In such a heterogeneous process, mass transfer processes play an essential role; adsorption and diffusion transfer itself; the phase disengagement surface area, its nature and condition.

There are currently no sufficiently reliable methods to determine or calculate the particles activity, that are located at the phases disengagement boundary Therefore, in determining the adsorbent behavior activity or the products concentration in substrate molecules interaction with potential participants, it is necessary to introduce conditional concepts –conditional transformation rate (trend) index, attributed to the interphase surface unit with introduction of the appropriate assumptions and time intervals.

The particular surface atoms or molecules energy state, their geometric arrangement determines the specific kinetic regularities of these heterogeneous transformations. For the most part, they are multi-staged. And the heterogeneous process speed as a whole depends on the individual stages speed and their ratio.

Adsorption transformations are reversible. Their equilibrium constants depend on pressure, temperature. At a given temperature, the pressure increase promotes desorption, its decrease – adsorption; at a given pressure, increasing of the temperature promotes desorption, decreasing - adsorption. These features of adsorbing materials behavior are the basis of the modern cyclic purification systems work with regeneration. They are CO_2 redistribution objects in a separate room, this study is dedicated to their behavior. Such phenomena do not require significant energy activation and last for a very short time.

Any really existing object is in complex interrelationship with the environment and exhibits a complicated behavior in natural gas exchange processes, in which the absorption-release processes are only constituents. To simulate such active samples behavior, with a characteristic dominant adsorption-desorption property, an experiment was performed using porous aluminosilicate adsorption plates under the conditions simulating a change in the CO_2 concentration in the air room volume, cyclic daily and seasonal changes in temperature; the stay conditions peculiarities and environment composition, the specific individual factors effect, etc. (see Fig. 9-10).

Table 3 -- Assessment of the constituent components conditional activity in the CO_2 content reduction processes in the studied air zones, related to plant photosynthesis and adsorption by artificial aluminosilicate materials

Photosynthesis	Adsorption
S=3200 cm^2	S=1075 cm^2
t = 18,8°C	t = 19,0°C
τ_Σ = 313 min.	τ_Σ = 301 min.
ΔCO_2 = 911 ppm	ΔCO_2 = 1588 ppm
$v_{сер.}$ = 2,91 ppm/min.	$v_{сер.}$ = 5,28 ppm/min.
$v_{сер.\ акт.}$ = $0{,}91 \cdot 10^{-3}$ ppm/cm^2·min.	$v_{сер.\ акт.}$ = $4{,}91 \cdot 10^{-3}$ ppm/см^2·min.

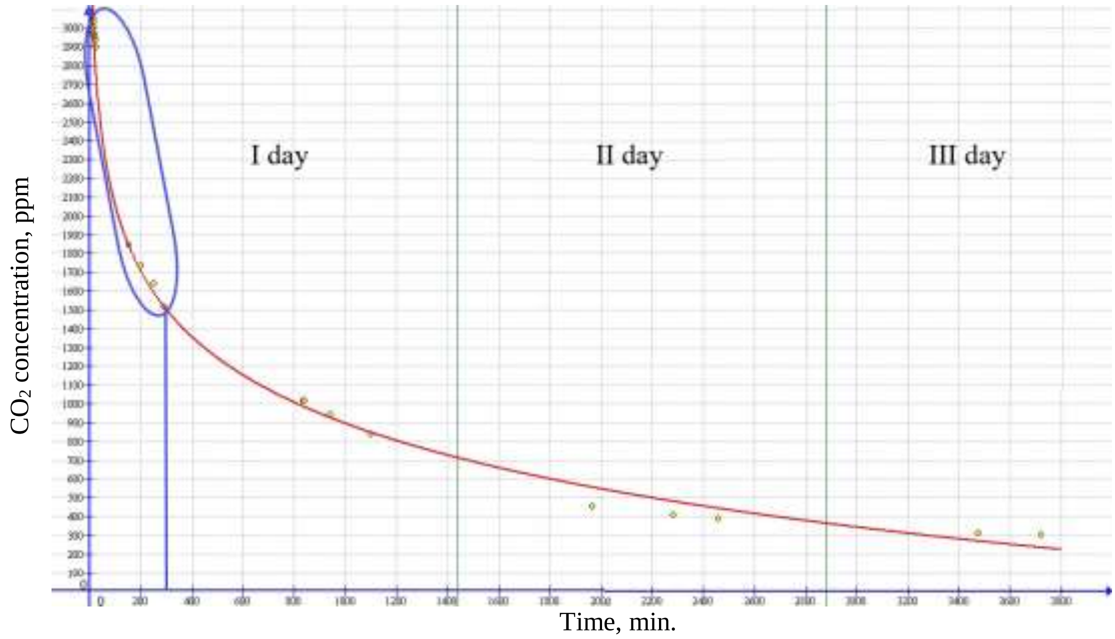


Figure 9 – Change of CO_2 concentration in the closed chamber with time in the artificial aluminosilicate plates (S = 1075 cm^2, T = 292 C, burst CO_2 introduction ~ 3100 ppm; chamber volume – 40 dm^3, static method; under conditions imitating stay length in the classroom during weekdays; during weekends)

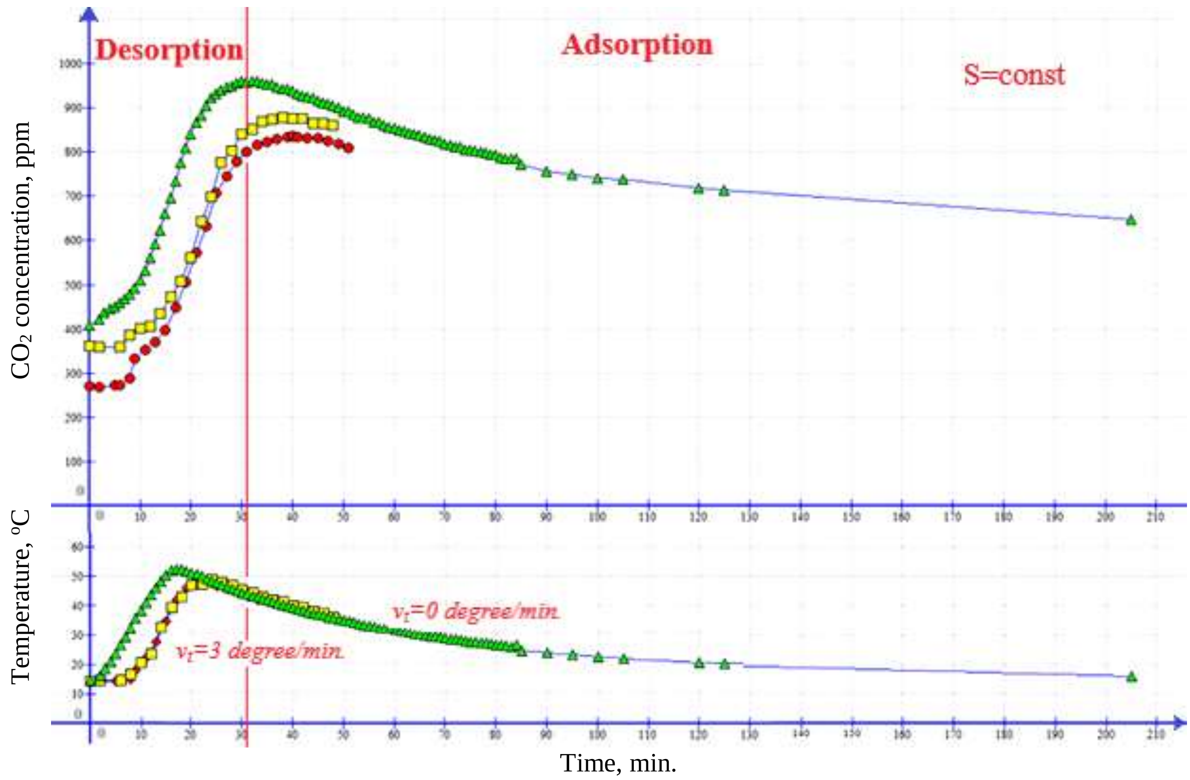


Figure 10 – Modeling behavior and analysis of properties reproducibility and artificial aluminosilicate adsorbing materials physical and technical characteristics in 3 cyclic tests series: heating (at a rate of 3 degree/min.) – cooling in a closed space and constant $S_{adsorption}$

Modeling of processes with adsorption-active objects was carried out:

a) with an internal CO_2 source,

b) with an external source (for example, simulating CO_2 concentration in inlet air):

– low intensity,

– high intensity.

In the simulation, the gas "burst" effect value on the system under study was chosen commensurate with ΔCO_2 value during the active influence period within the working education shift, during the weekend.

A comparative plant life processes analysis and the re-release by the studied adsorbents using averaged indices system in the formation of CO_2 contribution to the study object indicates that the absorbent surfaces action influence in adsorbing construction surfaces products is by ~ 4,9 times higher than the biological detectable activity, (considering particular staying conditions, with all these phenomena differences in their nature and behavior features). To change their action efficiency it is necessary to vary: the forms and types of constituent participants, active areas, thermodynamic systems conditions, time and exposure mode, etc.

This will form the further research basis.

Generalized empirical data on adsorption and desorption by adsorbent material in cyclic heating are graphically interpreted in fig. 10.

Alumosilicate porous clay products were selected as the adsorbent, dried and annealed according to the brick manufacture technology. This enables on the one hand

a) to simulate the CO_2 absorption processes using constructions made of such material,

b) on the other hand, to purposefully create adsorption channels for the design and construction of objects and providing them with air-purification systems (large classrooms in educational establishments, large concert and sports arenas, training halls, waiting rooms, large shopping centers, etc.),

c) the acquired knowledge permit to better know and understand the phenomena nature, mechanisms, kinetics, activation ways; to improve their efficiency and operational capacities.

The study evaluated the CO_2 adsorption-desorption processes contribution to the total heterogeneous set of gas-exchange transformations and determined their significance. The empirical data obtained are valuable for understanding the impact of changing the studied object conditions on the equilibrium processes displacement direction and the related transformations permit generalization.

The empirical study of artificial aluminosilicate adsorbing products adsorption-desorption processes enables:

– to know and understand the phenomena nature, mechanisms, kinetics, activation ways better; improve their efficiency and operational capacities;

– to find out the prevailing tendencies manifestation conditions, which enables to organize appropriate operation modes for purification complexes;

– to understand the facts that

a) in the dynamic heating mode due to the limited heat transfer speed, active CO_2 molecules exchange centers in the contact adsorbing material layer are involved in interaction with some delay. That is registered with the CO_2 concentration measurement system in the operating test facility volume.

b) transformation cycles repeatability and reproducibility permit to predict operation stability and reliability, principles of designed systems functioning;

c) the hysteresis properties dependencies narrowness enables stable operation of technically implemented systems;

d) low temperature values for reversing the transformations domination indicate the possibility to choose an easy, in energy terms, construction variant of purification (accumulation) systems even with the use of renewable energy sources low-power means;

– to identify directions for searching efficient alternatives to the already existing solutions.

Such an experimental study is complex, needs much labor input and a long time to establish equilibrium conditions. It requires stability in the measurement system's operation, performing tests in adequate conditions.

Stage V. Model ideas about the processes in the particular school classroom air.

The separate classroom air should be considered as a multi-component open subsystem with a strong connection with environment, each present component of which is an active participant in complex by nature (often understudied) multi-staged transformations. Most components properties are environment parameters functions. And when the state of the latter changes, the interactions course becomes reversible, the transformations depth in such objects will depend on the set of specific intense actions and extensive factors.

The CO_2 flow into the room air space occurs:

1) from an external source - from environment air space

– through inlet air supply by means of the implemented ventilation system (prevailing) and is determined

a) by the degree of room isolation from the environment,

b) by the degree of air exchange provided by the particular applied ventilation means;

– as a result of mass-gas exchange through enclosing structures, etc.;

2) from internal living and non-living origin sources in the process of

– carbon dioxide emissions generated by people in the occupied space;

– activity manifestation in the modern plants life in the conditions under consideration;

– changes in the behavior of the existing adsorbing surfaces present, etc.;

3) when changing the stay conditions

– under changes in intensive and extensive external and internal factors influence,

– under influence of factors that cause changes in the direction and resulting processes magnitude, nature, relation, dominance;

– maintaining the stability and reproduction of the hysteresis physical properties of the used active objects in the course of relaxation transformations;

– under other factors influence.

The results of systematic versatile multi-staged studies on detecting the CO_2 contribution by constituent components to the separate room airspace, permitted to obtain our own and literature data [22, 23] on the possibility of implementing activation methods for air purification using new photocatalytically active layered perovskite-like oxide materials $M_2Ln_2Ti_3O_{10}$ (M – Li, Na, K; Ln – La, Nd) (see Fig. 2 c) are necessary prerequisites for formulating the technical requirements and development for designing the adaptive systems to maintain a microclimate, with the possibility to regulate inlet-recirculation air flows ratio and adsorption and photocatalytic treatment (both solar and artificial radiation) and using composite TiO_2-containing materials, their modifications (as an example, see Fig. 11).

The obtained results open the search directions for development of modern systems for separation, purification, regeneration, accumulation, CO_2 storage, its partial or complete restoration; improvement of their technical capacities and specifications using innovative solutions and may become a necessary scientific basis for engineering generalizations.

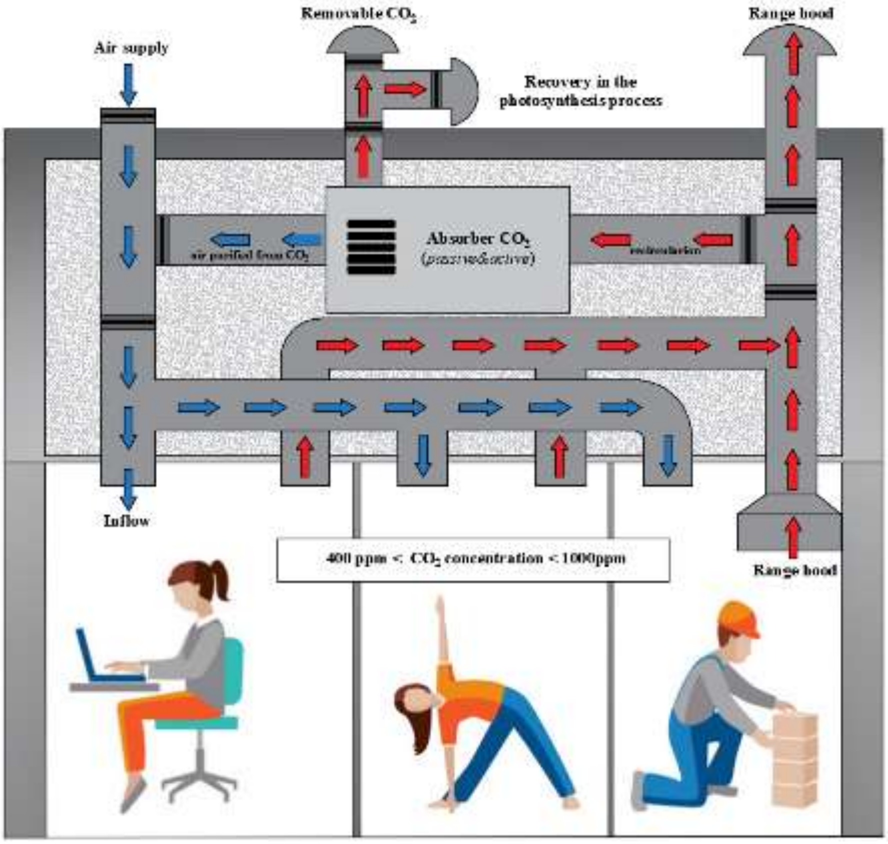


Figure 11 – Possible construction scheme of a comprehensive adaptive air supply system with additional inflow and exhaust ducts arrangement with shut-off valves, simultaneous purification of regenerative air streams by passive (active) adsorbers and implementation of continuous monitoring and object management systems

Conclusions

According to the developed methodology and using electronic devices for measuring the carbon dioxide (CO_2) content in the air and using special laboratory facilities set on the school classroom example, the following was studied:

a) the variety of interactions between the external environment and the subsystem - classroom air;

b) internal objects constituents activity (present pupils, plants, adsorbing surfaces of structures elements and interior) to emit-absorb CO_2 depending on the conditions and variation of dominant factors action;

c) their manifestation and influence on the forming the air mass composition and content, and therefore the classroom internal microclimate, which quality directly affects the health and working efficiency of the modern people.

The study is aimed at monitoring and real assessment of contribution, clarifying the ratio of the CO_2 emission volumes by each source in the overall mass-gas manifestation and search for innovative solutions to develop adaptive systems for stabilization and maintenance of microclimate in similar objects.

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THE EXPERIMENTAL RESEARCHES OF REINFORCED CONCRETE I-BEAM ELEMENTS WITH NORMAL CRACKS WHEN TURNING

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The data of experimental researches of the rigidity of reinforced concrete I-beam elements with normal cracks at the action on them of the twisting moment have resulted in this paper. It is shown that the dependence "torque-twist angle" is almost linear. Significant nonlinear deformations appear in the last stages of loading before failure. Therefore at normative torques, it is recommended to consider the work of reinforced concrete elements of the I-beam cross-section with normal cracks linear. It is shown that the presence of longitudinal reinforcement affects the strength and rigidity of beams with normal cracks. Quite a large part of the external torque is perceived by the pin forces in the longitudinal reinforcement. The difference between the external torque and the moment of the pin forces in the armature is perceived by the upper shelf of the I-beam element. In the absence of longitudinal reinforcement, the upper shelf can collapse at loads much smaller than the destructive load of beams with longitudinal reinforcement.

Keywords: I-beam, torsion, normal cracks, torsional strength, longitudinal reinforcement, pin force

ЕКСПЕРИМЕНТАЛЬНІДОСЛІДЖЕННЯЗАЛІЗОБЕТОННИХ ДВОТАВРОВИХ ЕЛЕМЕНТІВ З НОРМАЛЬНИМИ ТРІЩИНАМИ ПРИ КРУЧЕННІ

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Встановлено і представлено залежність «крутний момент-кут закручування»,вона є практично лінійною. Суттєві нелінійні деформації з'являються на останніх етапах навантаження перед руйнуванням. Тому при нормативних крутних моментах рекомендовано вважати роботу залізобетонних елементів двотаврового поперечного перерізу з нормальними тріщинами лінійною. Показано, що наявність поздовжньої арматури впливає на міцність і жорсткість балок з нормальними тріщинами. Достатньо велику частину зовнішнього крутного моменту сприймають нагельні сили в поздовжній арматурі. Різницю між зовнішнім крутним моментом і моментом нагельних сил в арматурі сприймає верхня полка двотаврового елементу. При відсутності поздовжньої арматури верхня полка може руйнуватись при навантаженнях, набагато менших, ніж руйнуюче навантаження балок з поздовжньою арматурою. Армування експериментальних балок з нормальними тріщинами тільки поздовжньою арматурою суттєво впливає на їх жорсткість. На міцність при крученні поздовжня арматура елементів з нормальними тріщинами впливає не так суттєво, як на жорсткість. Достатньо велику частину зовнішнього крутного моменту сприймають нагельні сили в поздовжній арматурі. Різницю між зовнішнім крутним моментом і моментом нагельних сил в арматурі сприймає верхня полка двотаврового елементу. Збільшення діаметра поздовжньої арматури призводить до зменшення деформацій і відповідно збільшення жорсткості балок при крученні. На основі експериментальних досліджень і враховуючи попередні теоретичні дослідження автора спростовано здавна існуючу думку про те, що поздовжня арматура не впливає на міцність при крученні. Наведені факти, на погляд автора, повинні бути враховані при проведенні практичних розрахунків несучих систем залізобетонних будівель і споруд.

Ключові слова: двотавровий елемент, кручення, нормальні тріщини, жорсткість і міцність при крученні, поздовжня арматура, нагельна сила

Introduction

It is known that considering the spatial work of repeatedly statically indeterminate systems (concrete floors, bridges, frameworks of buildings) can significantly clarify the efforts arising in the individual elements of a complex system. It is also known that the redistribution of forces among the individual elements of statically indeterminate systems depends on the ratio of bending and torsional stiffness of these elements [1]. At the same time, in reinforced concrete statically indeterminate systems, the formation of various cracks (normal, inclined, spatial, separation cracks, etc.) influences the bending and torsional rigidity. Normal cracks are formed at low enough load levels. The formation of cracks entails an abrupt change in the stiffness of the element, and the stiffness can be reduced several times.

Review of research sources and publications

With a fairly broad exploration of reinforced concrete elements stiffness in bending, their stiffness and torsional strength are studied insufficiently. The main attention in scientific works and normative documents devoted to the work of reinforced concrete elements during torsion is paid to elements with spatial (spiral) cracks [4, 5, 9, 11, 12].

However, experimental and theoretical studies have shown [1, 3, 10] that normal cracks also significantly affect the torsional rigidity of the reinforced concrete elements. Numerous and approximate methods for determining the torsional stiffness of elements of rectangular, T-shaped, box-shaped, and hollow triangular sections are considered in [1, 3, 6 - 8]. In [2] the issues of reinforced concrete I-beams stiffness and torsional strength calculation are considered. Publications [6 - 8] are devoted to the experimental study of the reinforced concrete elements work of the rectangular, box, and hollow triangular cross-section.

However, stiffness and strength experimental studies of reinforced concrete I-beams with normal torsional cracks were not performed.

Objective of the work and research methods

Due to the above-mentioned content, the aim of this article is an experimental study of the strength and rigidity of reinforced concrete I-beams with normal cracks under the influence of turning.

Basic material and results

In the course of the experiment, it was supposed to investigate the torsional rigidity and strength of reinforced concrete elements of the I-beam section with normal cracks on the models. There were made samples with data presented in Fig 1.

The aim of the research was to establish the nature of changes in the stiffness characteristics of samples with different diameters of longitudinal reinforcement and different cross-sectional dimensions. Due to the fact that the author theoretically found out that for I-beams with a small wall thickness, the crack height does not play a significant role, it was decided to take the same normal crack height equal to half the cross-sectional height, and vary the cross-sectional size and reinforcement diameter.

Artificial normal cracks were created using Perspex plates, which were inserted at the location of the crack when laying concrete in the formwork. Such cracks divided the samples along the length into separate blocks, interconnected by a part of concrete without cracks and longitudinal reinforcement (Fig. 2). The length of the blocks was 300 mm. In addition, three samples were made solid, without artificial normal cracks.

To analyze the experimental data and establish the relationship between the parameters of deformation (twisting angle of the blocks separated by cracks) and the magnitude of the external load were plotted "torque-twisting angle". The sketches 3-5 present such graphs for some tested beams as samples.

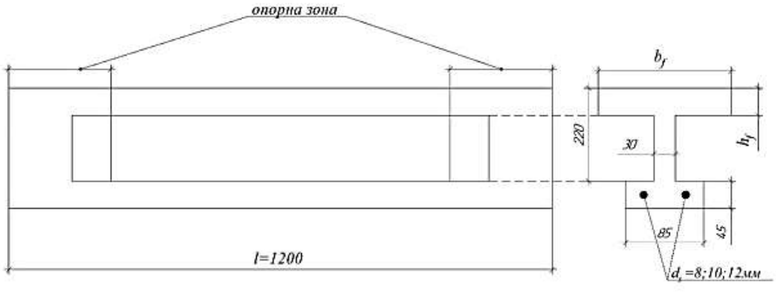

Figure 1 – Dimensions of experimental samples

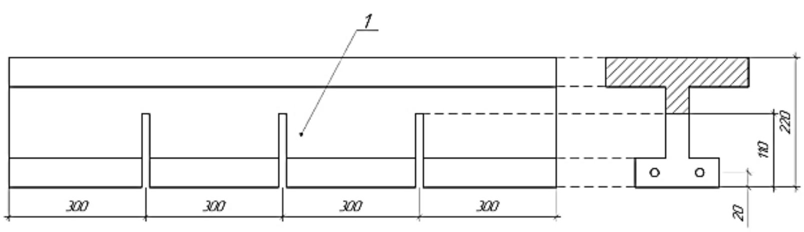

Figure 2 – Scheme of the formation of artificial cracks and reinforcement of samples (1 – Perspex)

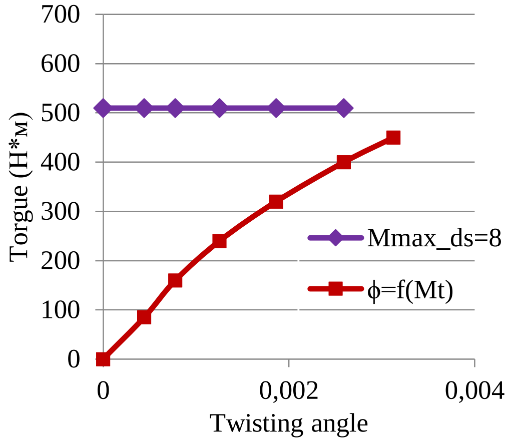

Figure 3 – Twisting angles and destructive moments for beams with dimensions: b_f = 300 mm; h_f = 30 mm; d_s = 8 mm

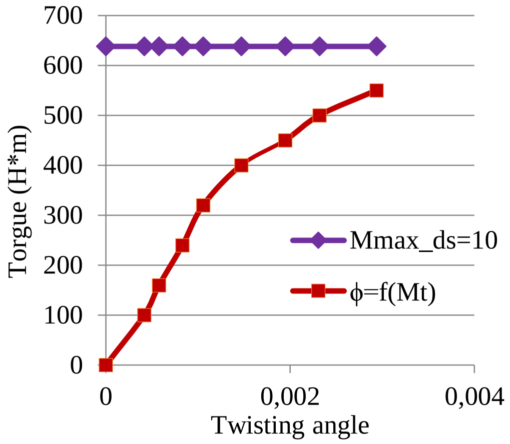

Figure 4 – Twisting angles and destructive moments for beams with dimensions: b_f = 300 mm; h_f = 40 mm; d_s = 10 mm

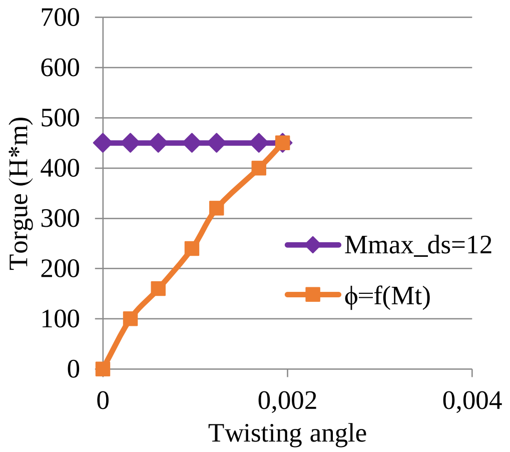

Figure 5 – Twisting angles and destructive moments for beams with dimensions: b_f = 300 mm; h_f = 40 mm; d_s = 12 mm

Also besides the dependencies of the "torque-twisting angle" the maximum torque values Mmax is shown, i.e. the destructive torque, in addition to the torque-to-twist dependencies. In the penultimate stages of the load measuring instruments were removed to prevent their destruction, thus there are no dimensions of the twist angles during failure.

In the figure it can be seen that the "torque-twist angle" dependence is almost linear. Significant non-linear deformations appeared in the last stages of loading before failure. Therefore at normative loadings it is possible to consider work of samples linear.

Beam with dimensions b_f = 300 mm; h_f = 40 mm; d_s = 12 mm (see Fig. 5) was prematurely slightly destroyed by puncturing the longitudinal reinforcement, which is most likely a disadvantage of its concreting. Therefore, its destructive moment is much smaller than the destructive moments of other beams of the same series.

The pattern of cracking of all samples with artificial normal cracks was similar. An inclined crack appeared from the top of the artificial crack and extended to the beam top shelf. In the future, the picture of several stages of the load remained unchanged. In some samples, the concrete peeled off near the longitudinal reinforcement, but it did not affect the strength of the samples, except for the sample (see Fig. 5). Upon further loading, a spatial crack appeared in the beam upper shelf. The deformations increased significantly. Then the moment of beam destruction came.

In fig. 6 a general view of an inclined crack starting at the top of an artificial normal crack is shown.

Analyzing the experimental data, it can be stated that the presence of longitudinal reinforcement affects the strength and rigidity of the beams with normal cracks. [16 – 20]. Quite a large part of the external torque is perceived by the pin forces in the longitudinal reinforcement. The difference between the external torque and the moment of the nail forces in the reinforcement is perceived by the upper shelf of the I-beam element. In the absence of longitudinal reinforcement, the upper shelf can collapse at loads much smaller than the destructive load of beams with longitudinal reinforcement. This fact is confirmed by the premature destruction of the beam, which is described above, from the puncture of the longitudinal reinforcement [13 – 14].

Unlike beams with artificial normal cracks, sloping cracks at the edge initially appeared in the beams without cracks (Fig. 11). These cracks then spread to the lower and upper shelves. A space crack appeared on the top shelf. At the same time deformations sharply increased. After that, the beams collapsed as a result of a loss of the upper shelf load-bearing capacity. Destructive moments of beams without cracks were slightly larger than moments of beams with cracks [15].

In pictures 9-10 there are shown graphs of "load-twist angle" for the beams without artificial normal cracks.

Figure 6 – Inclined crack from the top of the artificial normal crack in the beam

Figure 7 – The destruction of the beam

Figure 8 – Premature destruction of the beam as a result of puncturing the longitudinal reinforcement

For comparison with experimental data in Fig. 9-10 graphs of the elastic calculation of these beams in the program вЂњLiraвЂќ using three-dimensional finite elements are shown. The figures confirm the elastic nature of work to high levels of load.

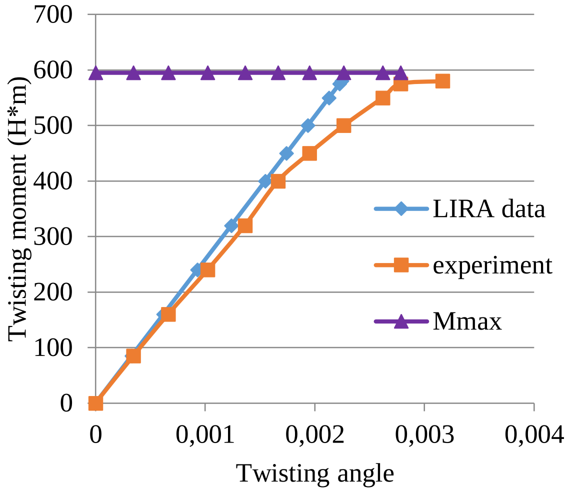

Figure 9 – Twisting angles and destroying moments for the beam without cracks (d$_s$=8 mm)

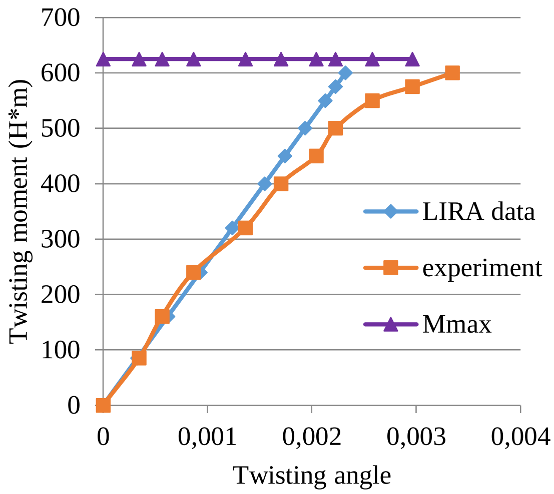

Figure 10 – Twisting angles and destroying moments for the beam without cracks (d$_s$=10 mm)

Figure 11 – General view of of an inclined crack at the edge

Conclusions and the prospects of research

Experimental studies have shown that the "torque – twist angle" diagram of reinforced concrete elements of the I-beam section with normal cracks to high load levels is linear. Plastic flows take place in the last stages of loading, before destruction. The main type of failure is the destruction of the I-beam element upper shelf with the development of a spatial torsional crack.

The experimental beams reinforcement with normal cracks only by longitudinal reinforcement significantly affects their rigidity. The longitudinal reinforcement torsional strength of elements with normal cracks does not affect as significantly as the stiffness. Quite a large part of the external torque is perceived by the nail forces in the longitudinal reinforcement. The difference between the external torque and the moment of the nail forces in the reinforcement is perceived by the upper shelf of the I-beam element.

Increasing the longitudinal reinforcement diameter leads to decrease in deformation and, accordingly, to increase in the beams stiffness during torsion. Increasing the stiffness and strength of the top shelf affects both the beams overall stiffness their strength.

The research shows that the torsional strength of reinforced concrete elements depends on the cross-section of the longitudinal reinforcement in the presence of normal cracks, which refutes the long- held supposal that the longitudinal reinforcement does not affect the torsional strength. These facts, in the opinion of the author, should be considered when conducting practical calculations of reinforced concrete buildings and structures load-bearing systems.

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