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Современные тенденции в строительстве, градостроительстве и планировке территорий

Рецензируемый научно-практический журнал (издается с 2022 года)

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Журнал включен в перечень рецензируемых научных изданий, в котором должны быть опубликованы основные научные результаты диссертаций на соискание ученой степени кандидата наук, на соискание ученой степени доктора наук (Перечень ВАК, категория II) по следующим научным специальностям:

2.1.1 – Строительные конструкции, здания и сооружения (технические науки)
2.1.2 – Основания и фундаменты, подземные сооружения (технические науки)
2.1.5 – Строительные материалы и изделия (технические науки)
2.1.7 – Технология и организация строительства (технические науки)
2.1.9 – Строительная механика (технические науки)
2.1.13 – Градостроительство, планировка сельских населенных пунктов (технические науки)
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CONTENTS

СОДЕРЖАНИЕ

ANNIVERSARY OF THE SCIENTIST
ЮБИЛЕЙ УЧЕНОГО

Dmitriy Rafaelovich Mailyan has turned 70

Dmitry Rafaelovich Mailyan is a Professor, Dr.Sc. (Eng.), Corresponding Member of the Russian Academy of Natural Sciences, Honorary Builder of the Russian Federation, Honorary Worker of Higher Professional Education of the Russian Federation, Honorary Professor of the DSTU.

In 1981, Dmitry Rafaelovich defended his disseration for the degree of Candidate of Technical Sciences (Cand.Sci. (Eng.)) in "Constructions, Buildings and Structures" at the Kiev Institute of Civil Engineering. The dissertation topic was "Strength, Deformability and Crack Resistance of Reinforced Concrete Bars with Prestressed Reinforcement". In 1994, he went on to defend his dissertation for the degree of Doctor of Technical Sciences (D.Sc. (Eng.)) at the Moscow Institute of Transport Engineers. The dissertation topic was "Effective Compressed Prestressed Elements and Methods of Calculating them under Various Loading Conditions Taking a History of Deformation into Account." From 1980 to 1991, Dmitry Rafaelovich worked as a researcher, senior researcher, and leading researcher at the Reinforced Concrete Department of the Rostov Promstroyniiproekt Design Institute. From 1991 to 1996, he was a senior lecturer, associate professor and went on to be a professor at the Department of Mathematics, Mechanics, Computer Science, and Structures at the Institute of Architecture and Arts of the Southern Federal University. Since 1996 up to the present, he has been Head of the Department of Reinforced Concrete and Stone Structures at the Don State Technical University. The Center for Forensic Examinations of DSTU has been successfully in operation under his leadership.

Dmitry Rafaelovich Mailyan is a renowned scientist in the field of construction, innovative reinforced concrete structures, development of methods for designing and calculating various force effects. His teaching and scientific input has involved 400 works, including 30 monographs, 20 textbooks, manuals and reference books, 37 copyright certificates and patents. Under his leadership, 3 doctors (D.Sc. (Eng.)) and 39 candidates of technical sciences (Cand.Sci. (Eng.)) have been trained with excellence. Dmitry Rafaelovich is a member of three dissertation boards (the Don State Technical University and Kazan Federal University); organizer of a few international scientific and technical conferences; a member of the editorial boards of three scientific and technical journals, Deputy Editor-in-Cchief of the journal "Modern Trends in Construction, Urban and Territorial Planning".

Dmitry Rafaelovich Mailyan has been endorsed by Academician of the Russian Academy of Architecture and Civil Engineering Sciences, Doctor of Technical Sciences, Professor, Head of the Department of Constructions, Buildings and Structures at the Russian University of Transport V.S. Fedorov for an Adviser to the Russian Academy of Natural Sciences in the Department of Building Sciences.

The scientist's fruitful academic, research and industrial activities have been highly appreciated at his home university and beyond. As part of a team of Russian scientists and manufacturers, Dmitry Rafaelovich has been awarded the Government of the Russian Federation Prize in Science and Technology and with the honorary title of Laureate of the Government of the Russian Federation for developing the scientific foundations of optimal construction design, advanced information modeling and efficient life cycle management of residential and public buildings in the country's various regions. The scientist has been also awarded the medal of the Order "For the Service to Russia", the Order "For the Service to the Rostov Region".

The editorial staff of the journal "Modern Trends in Construction, Urban and Territorial Planning" would like to extend cordial wishes to Dmitry Rafaelovich on the auspicious occasion of his anniversary and wish with a lifetime of good health, prosperity, inspiration and energy for new scientific discoveries for the benefit of his home university and the Russian construction industry.

BUILDING CONSTRUCTIONS, BUILDINGS AND ENGINEERING STRUCTURES

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Original Empirical Research

EDN: TIJSMW

Technology of Forming Tubular Concrete Columns

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Abstract

Introduction. In manufacturing tubular concrete columns, attention should be paid to a technology of filling a steel tube with a concrete mixture. In practice when a concrete mixture is being fed into a tube from the top, a proper quality of concreting a structure is not guaranteed. In this study, the aim was to test the effectiveness of a technology of forming columns with a self-sealing concrete mixture by means of pressure concreting.

Materials and Methods. Experiments to study the effectiveness of the suggested technology for forming a tubular concrete column with a saturated content of longitudinal and spiral reinforcement were conducted on a transparent model. A self-compacting mix composition was selected that ensured the required workability, flowability, homogeneity, and absence of stratification. Through the course of the experiments, the process parameters were adjusted with the speed of filling the structure with the concrete mixture as the most important one. The uniformity of concrete distribution throughout the model was monitored.

Research Results. The experiment indicated that the concrete mixture easily reached all of the areas of the structure with no loss of quality. No air bubbles were observed on the surface of the structure, indicating high-quality compaction of the mixture. Concrete strength and density variations in the upper, middle, and lower zones of the structure were minimum. The resulting concrete is characterized by a homogeneous structure, with a uniform, evenly distributed composition with no significant voids or inhomogeneities throughout the entire volume.

Discussion and Conclusion. The absence of formwork, scaffolding, and other equipment, high speed of concreting significantly reduce the time and labor intensity of column installation. All of these, combined with the achieved concrete quality, open up avenues for a broad-scale use of the pressure concreting method in construction projects in the Russian Federation. Further work is needed in order to develop a method for ultrasonic quality control of concrete in tubular concrete columns.

Keywords: tubular concrete columns, self-compacting concrete, method of ascending concrete mixture, strength and density of concrete, homogeneity of a structure

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Оригинальное эмпирическое исследование

Технология формования трубобетонных колонн

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Аннотация

Введение. При изготовлении трубобетонных колонн следует обратить внимание на технологию заполнения стальной трубы бетонной смесью. Практика показывает, что при подаче бетонной смеси в трубу сверху должное качество бетонирования конструкции не гарантируется. В данном исследовании ставилась цель опытной проверки эффективности технологии формования колонн самоуплотняющейся бетонной смесью методом напорного бетонирования.

Материалы и методы. Эксперименты по исследованию эффективности предложенной технологии формования трубобетонной колонны с насыщенным содержанием продольной и спиральной арматуры проводились на прозрачном макете. Подобран состав самоуплотняющейся смеси, обеспечивающий требуемую удобоукладываемость, текучесть, однородность и отсутствие расслоения. В ходе экспериментов подобраны технологические параметры формования, среди которых наиболее важным была скорость заполнения конструкции бетонной смесью. Контролировалась равномерность распределения бетона по всему макету.

Результаты исследования. В результате эксперимента выявлено, что бетонная смесь легко достигала всех зон конструкции без потерь качества. Наблюдалось отсутствие выделения воздушных пузырей с поверхности конструкции, что свидетельствует о высоком качестве уплотнения смеси. Разброс прочности и плотности бетона в верхней, средней и нижней зонах конструкции минимальный. Полученный в конструкции бетон характеризуется гомогенной структурой, имеющей однородный, равномерно распределенный состав без значительных пустот или неоднородностей по всему объему.

Обсуждение и заключение. Отсутствие опалубки, подмостей и прочих приспособлений, высокая скорость бетонирования значительно снижают продолжительность и трудоемкость монтажа колонн. Все это, в совокупности с достигнутым качеством бетона, открывает хорошие перспективы для широкого применения метода напорного бетонирования в строящихся объектах в РФ. В дальнейшем необходимо разработать методику ультразвукового контроля качества бетона трубобетонных колонн.

Ключевые слова: трубобетонные колонны, самоуплотняющаяся бетонная смесь, метод восходящей бетонной смеси, прочность и плотность бетона, гомогенность структуры

Благодарности. Авторы выражают благодарность редакции и рецензентам за внимательное отношение к статье и указанные замечания, которые позволили повысить ее качество.

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Introduction. Tubular concrete columns (TBCs) are increasingly used in designing frames of high-rise buildings in the Russian Federation. This is due to a number of significant advantages of the economic, technological, constructive plan, as well as the architectural expressiveness of these structures [1–6]. One of the restraining factors for the broader use of TBCs in this country is the insufficiently developed technology of filling steel tubes with a concrete mixture. Due to the considerable length of steel tubes with a relatively small diameter, which provide "non-removable formwork" for columns on two or three floors or higher, traditional technologies for filling them with a concrete mixture from the top fail to ensure proper concreting quality. It is fairly challenging to ensure and control the quality of laying and compaction of a concrete mixture under such conditions.

According to Japanese builders' experience that are among the leaders in the volume of TBC applications, the most promising technology for their molding is use of the ascending concrete mixture method also known as the pressure concreting method where a concrete mixture is pumped with a concrete pump under pressure through a specially made hole in a steel tube from bottom to top.

[7, 8] have shown that this concreting technology reduces the number of air bubbles in the contact layer of reinforcement with concrete. The resulting concrete layer under the reinforcement was characterized by a better

quality compared to the traditional concreting on top of the mold leading to an increase in the adhesion of reinforcement to concrete.

In order to ensure the necessary high plasticity and mobility, self-sealing concrete mixtures (SSCMs) should be employed. Such mixtures started being used overseas in the last century, but are currently broadly used in this country [9–11]. The use of SSCMs allows one to attain a number of other positive results with the most important ones being:

– no need for vibration during installation;
– no delamination of the mixture;
– reduction in the cement content;
– high strength, density and durability of concrete.

The aim of the study was to test the effectiveness of the technology of forming TBC columns using substructures by means of pressure concreting.

Materials and Methods. At the first stage, the composition of the SSCMs was selected and its quality was checked. To this end, the workability of the mixture was monitored by means of an Abrams cone and the fluidity was identified using the locking ring method. The second method was employed in order to evaluate the capacity of the mixture to penetrate into narrow spaces, including gaps between reinforcing bars, with no delamination or clogging.

Sampling for identifying the normalized parameters of the mixtures was carried out in compliance with GOST R 59715 "Self-Sealing Concrete Mixtures. Testing Methods" taking the instructions of GOST 10181 "Concrete Mixtures. Testing Methods" into consideration.

Concrete strength was monitored based on the results of compression tests of control samples-cubes with a rib size of 100 mm at the age of 3, 7, 28 days in compliance with GOST 18105 "Concretes. Rules for Strength Control and Assessment", GOST 10181 and GOST 31914 "High-Strength, Heavy and Fine-Grained Concretes for Monolithic Structures. Rules for Quality Control and Assessment".

When a mixture of the required quality has been obtained, model experiments on molding the product were performed on a large-scale transparent model of a tube-concrete column (Fig. 1). The model of the tubular concrete column had a height of 1973 mm and a diameter of 300 mm. The shell tube was made of plexiglass. The reinforcement frame was made of 7 longitudinal rods with a diameter of 25 mm and a spiral of rods with a diameter of 8 mm with a pitch of 50 mm. Additionally, two longitudinal rods with a diameter of 25 mm are installed in the lower part of the reinforcement frame with a light distance between them and the remaining rods of 30 mm. That allowed assessment of the penetration ability of SSCMs between the rods of densely reinforced TBCs.

a) *b)*

Fig. 1. TBC sample: *a* — general view of the layout for testing the WBS method; *b* — location of core drilling

In order to supply the concrete mixture, a 125 mm diameter concrete duct system was employed that is connected to the layout with special quick-release fittings. The mixture was supplied by a Cifa PC 709 concrete pump. 72 hours after forming the reinforced concrete core, cores were extracted from the mock-up for them to gain concrete strength under natural conditions. The cores were drilled from four sites located at the bottom, top and in the middle zones of the layout (Fig. 1*b*).

Concrete strength was monitored at the age of 28 days based on the results of compression tests of cylinders with a diameter of 100 mm and a height of about 100 mm in accordance with GOST 28570 "Concretes. Methods for Identifying Strength Based on Samples Selected from Structures" and GOST 31914.

Research Results. The results of identifying the workability of SSCMs by the size of the spreading diameter of a standard cone and the spreading time up to a diameter of 500 mm are shown in Tables 1, 2 and in Fig. 2.

The resulting mixture is highly stable. The cement paste and water did not separate from the aggregate along the circumference of the spread, and there were no signs of delamination of the concrete mixture. The time to stop blurring is 62 seconds. The contours are clear. The mixture was dense and homogeneous throughout the entire test until the installation was completed. Workability retention is180 min long.

The compressive strength of concrete control samples of cubes according to the results of the cube strength control corresponds to class B70.

Table 1

Results of control of workability of the mixture

Composition number	Time t_{500}, sec	Diameter of the spread, cm			Spread grade	Viscosity grade
		d_1	d_2	d_{cp}		
#1	3	77.5	77	77.25	PK3	V2

Table 2

Results of the cone spreading control in the locking ring

Composition number	Diameter of the spread, cm			Measured heights, cm					Fluidity grade
	d_1	d_2	d_{cp}	h_0	h_{x1}	h_{x2}	h_{y1}	h_{y2}	
#1	80	79.5	79.75	12.3	12.8	12.9	12.9	12.7	T4

a) *b)*

Fig. 2. Test results of the concrete mixture: *a* — identifying spread, viscosity grades; *b* — identifying fluidity grades

The results of forming the TBC layout are as follows. The time of filling with the mixture is 42 seconds long, the filling speed is 229.5 liters/min. As the TBC layout was being filled, it was important to ensure an even distribution of concrete throughout the entire volume of the structure. Visual observation of the filling process through transparent walls showed that the mixture started rising inside the reinforcement frame, but when a height difference of more than 50 mm had been reached, it spread along the peripheral zones of the structure. The mixture easily reached all of the areas with no loss of quality. At the end of the concreting process, there was no release of air bubbles from the surface of the structure, which is an indication of a high quality of compaction of the concrete mixture. The condition of the outer surface of the reinforced concrete core is shown in Fig. 3.

a) *b)*

Fig. 3. Concrete compaction quality: *a* — reinforced concrete core in section; *b* — lateral surface of the reinforced concrete core

According to the test results of four cylinders, the compressive strength of concrete ranged from 77.9 MPa to 81.2 MPa. The density of the samples was also in a very narrow range, from 2549 kg/m^3 to 2588 kg/m^3. Such results are indicative of the absence of stratification of the mixture and uniform distribution of aggregates over the entire volume and height.

Discussion and Conclusion. The research has shown that the WBS method is capable of delivering good-quality molding and compaction of TBC concrete. The thixotropic behavior of the SSCMs reduces the viscosity of the mixture and counteracts its segregation due to shear stress during the continuous movement of the mixture in the volume of the structure under the pressure of the updraft. As a result, a high uniformity of concrete is attained in the structure including its height. The variation in strength and density of concrete in the upper, middle and lower zones of the structure is minimal.

For TBCs, which are more broadly used in frames of high-rise buildings, the uniformity of concrete throughout the entire volume of the element is no less significant than its absolute strength. Heterogeneity leads to local stress concentrations and a decrease in the overall load-bearing capacity of the structure, which might negatively affect the mechanical safety of the building as a whole.

It should be noted separately that in calculating the strength of pipe-concrete columns concreted by means of this method, it is not necessary to apply the coefficient of working conditions $\gamma_{b3} = 0.85$ to the calculated compressive strength of concrete (Section 6.1.12 of SP 63.13330.2018 "Concrete and Reinforced Concrete Structures. Basic Provisions"). Combined with a possibility of practical use of low strength variability (in identifying the concrete class), it makes it possible to obtain higher calculated values of compressive forces in concrete structures. As a result, significant improvements of the technical and economic performance of the columns become possible.

The studies allow us to conclude that it is advisable that the TBC molding technology is employed using the WBS method when a self-sealing concrete mixture inside a steel pipe is pressurized from bottom to top.

Let us note the obvious advantages of the method over the traditional ones with a mixture feed from the top:

– a significant reduction in the possibility of capturing additional air inside the substation;

– practical exclusion of a possibility of segregation of the mixture due to impacts on the reinforcement and the walls of the formwork tube during free fall;

– a significant reduction in the filling time of the structure;

– exclusion of the formation of working seams and operations with their organization;

– a reduction of construction installation time;

– a reduction in the number of workers on the construction site;

– eliminating the need to make use of scaffolding, ladders, and ladders.

The concrete obtained in the structure is characterized by a homogeneous structure with a homogeneous, evenly distributed composition with no significant voids or inhomogeneities throughout its volume. It becomes possible to obtain higher values of the calculated resistance of concrete compared to concrete of the same composition, but molded by means of the traditional technologies.

However, due to the lack of regulatory documents with recommended values of technological parameters of molding for the VBS method (upper and lower limits of the filling rate of concrete blocks, diameters of concrete pumps, overpressure values), it is advisable that broader comprehensive studiesare are conducted in the area. For construction practice, it is also essential to develop a methodology for ultrasonic quality control of concrete in columns.

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Modeling the Effectiveness of Damping Systems in Building Structures under Seismic Impact Using the Generalized Ornstein-Uhlenbeck Process

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Abstract

Introduction. At the current stage of science and technology development, the issues of vibration protection of buildings are being intensively investigated, particularly for special structures of a high responsibility class. Various damper designs are also being developed and tested. Modeling the response of damper systems is an urgent task with both deterministic and stochastic approaches to solving it. This study demonstrates the features of the stochastic approach in a variant of the mathematical model of the generalized Ornstein-Uhlenbeck process. It is possible to the effect of using a nonlinear damper in a specific case can be evaluated by means of this model, and the conclusions can be made regarding its benefits.

Materials and Methods. The major research method is the solution of a stochastic differential equation. A numerical experiment is shown with a real example of analyzing the dynamic response of a turbo unit with a viscoelastic nonlinear damper to a random seismic impact. The application of the generalized Ornstein-Uhlenbeck process for mathematical modeling of the dynamic response of damping devices to seismic impacts on critical energy infrastructure is analyzed.

Research Results. A stochastic model is set forth that takes into account both the random nature of seismic excitations and the nonlinear rheological characteristics of dampers. The results of the numerical experiment confirm that while calculating by means of the described model, the use of a nonlinear damper is justified, the mean-square values of the displacement response are reduced by more than two times helping to reduce seismic risks for the turbine unit as well as to increase its dynamic stability and reliability of operation under the action of random impacts.

Discussion and Conclusions. The developed methodology provides a quantitative assessment of the effectiveness of a variety of classes of damper systems and allows for parametric optimization of their settings in order to maximize their protective capacity and to reduce the vibration and dynamic loads on energy equipment. The theoretical significance of the study is the suggested calculation methodology, while the practical significance is in the assessment and recommendations for making use of viscoelastic dampers for a high responsibility class of structures.

Keywords: Ornstein-Uhlenbeck process, damping devices, earthquake resistance, energy facilities, stochastic modeling, vibration protection

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Оригинальное эмпирическое исследование

Моделирование эффективности демпферных систем в строительных конструкциях при сейсмическом воздействии с использованием обобщенного процесса Орнштейна-Уленбека

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Аннотация

Введение. На современном этапе развития науки и техники активно разрабатываются вопросы вибрационной защиты зданий, особенно для специальных сооружений высокого класса ответственности, разрабатываются и испытываются различные конструкции демпферов. Моделирование отклика демпферных систем является актуальной задачей, существуют различные подходы к ее решению, как детерминистские, так и стохастические. В настоящей работе показаны особенности стохастического подхода в варианте математической модели обобщенного процесса Орнштейна-Уленбека. Благодаря использованию данной модели стало возможно оценить эффект применения нелинейного демпфера в конкретном случае и сделать выводы о преимуществах его использования.

Материалы и методы. Основным методом исследования в настоящей работе является решение стохастического дифференциального уравнения, приводится численный эксперимент на реальном примере анализа динамической реакции турбоагрегата с использованием вязкоупругого нелинейного демпфера на случайное сейсмическое воздействие. Анализируется применение обобщённого процесса Орнштейна-Уленбека для математического моделирования динамического отклика демпферных устройств при сейсмическом воздействии на критически важные объекты энергетической инфраструктуры.

Результаты исследования. Предлагается стохастическая модель, которая одновременно учитывает случайную природу сейсмических возбуждений и нелинейные реологические характеристики демпферов. Результаты численного эксперимента подтверждают, что при расчете по описанной модели обосновано применение нелинейного демпфера, среднеквадратичные значения отклика по перемещению снижаются более чем в два раза, что способствует уменьшению сейсмических рисков для турбоагрегата, повышает запас его динамической устойчивости и надежность работы в условиях случайных воздействий.

Обсуждение и заключение. Разработанная методология обеспечивает количественную оценку эффективности различных классов демпферных систем и позволяет выполнять параметрическую оптимизацию их настроек с целью максимизации защитной способности и снижения вибрационно-динамических нагрузок на энергетическое оборудование. Теоретическая значимость работы заключается в предложенной авторами методике расчетов, практическая значимость — в оценке и рекомендациях по применению вязкопластичных демпферов для сооружений высокого класса ответственности.

Ключевые слова: процесс Орнштейна-Уленбека, демпферные устройства, сейсмостойкость, энергетические объекты, стохастическое моделирование, виброзащита

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Introduction. The calculation of buildings for seismic and dynamic loads is one of the evolving and most relevant sections of modern structural mechanics with a lot of research devoted to operation of structures in seismic conditions and use of damping devices of various designs [1–7] in order to protect buildings from vibrations and to prevent vibrations [8–14]. Ensuring the seismic stability of energy infrastructure is a major condition for maintaining reliability and continuity of energy supply at the regional and intersystem levels. Critical elements of an energy system — nuclear and thermal power plants, distribution substations, and extended linear facilities, including overhead and cable transmission lines — are subject to a considerable range of damage while exposed to varying degrees of seismic vibrations and frequency composition. Degradation mechanisms include loss of stability of supporting structures, resonant modes in flexible spans and suspensions, violation of integrity of foundations and anchor nodes, failure of secondarssystem equipment due to exceeding permissible displacements and accelerations. Correct selection and parametric adjustment of damping systems designed to reduce dynamic responses and redistribute inertial loads is thereby of particular importance [15, 16]. Traditional deterministic approaches to calculating damping devices and vibration isolation based on routine accelerograms and fixed loading scenarios typically fail to reflect the variability of actual seismic impacts, their unsteadiness, intereventuality and intraevent variability, and possible nonlinear effects in the design response. As a result, the formed design solutions might be sensitive to the uncertainties of the input parameters leading either to inefficient resource overruns or to underestimation of a probability of failure. Stochastic modeling of the dynamics of "soil–structure–equipment" using Markov processes, particularly the Ornstein-Uhlenbeck (OU) processes, provides a formal apparatus for describing a random component of a seismic impact and a system response of a probabilistic formulation. The OU process, which has the properties of Gaussian, incremental stationarity, and average return, enables approximation of filtered white noise with a specified correlation structure making it convenient for modeling not only the input acceleration of the base, but also generalized disturbances formed by the transition of wave fields through geological layers and supporting parts. Extended formulations include piecewise stationary and non-stationary modifications of OU processes with slowly changing dispersion and frequency parameters of the dominant components, vector models for a consistent description of multicomponent records (two horizontal and vertical components) [12]. Integration of such stochastic representations into the equations of motion of mechanical subsystems with linear or slightly nonlinear damping enables one to obtain analytical or quasi-closed expressions for response statistics: spectral densities of accelerations and displacements, variances of relative displacements, probabilities of exceeding limit states, average times prior to failure at specified thresholds. For damping devices with nonlinear characteristics (e.g., viscoelastic inserts with modulus dependence on frequency and temperature, frictional and viscoplastic dampers, inertial mass dampers), methods of equivalent linearization, stochastic averaging, and Fokker-Planck-Kolmogorov techniques are employed in order to estimate stationary and transient distributions of states. This provides probabilistic damping performance indicators, including risk curves and vulnerability functions for equipment elements and structural systems. The practical design methodology in a stochastic formulation entails calibration of the parameters of an input seismic process based on regional attenuation relations and an accelerogram database; identification of damper parameters based on experimental hysteresis curves and frequency-dependent modules; design of stochastic models of the structure-base interaction taking into account the base impedance and radiation damping; conducting probabilistic dynamic analysis in the time and frequency domain with sensitivity evaluation to key uncertainties. Optimization of the parameters of damping devices is formulated as a task of minimizing risk functionals (e.g., mathematical expectation of damage or a probability of exceeding the maximum responses over the service life) with limitations in weight, dimensions, manufacturability and operational reliability. For objects with a higher class of responsibility, it is advisable that multi-purpose optimization with the criterion of robustness to variations in input processes and degradation of material properties over time [17, 18] is employed. An additional rise in reliability is attained by means of combining Ornstein-Uhlenbeck processes with physically informed models of a source, propagation path, and near-surface conditions with Bayesian procedures for updating the parameters based on data monitoring. Built-in systems for recording a response of the equipment and structures allow for identification and online correction of stochastic models during operation, which ensures adaptive reconfiguration of dampers with variable characteristics and early detection of degradation changes. For the linear part of the infrastructure, spatially correlated multipoint models of seismic excitation are of particular significance making it possible to take non-synchronicity of impacts on extended lines and cascading effects in a network into consideration. Transition from deterministic to stochastic methods of analysis and design of damping systems based on Ornstein-Uhlenbeck processes thus provides a more adequate representation of a natural variability of seismic impacts, allows one to obtain quantitative probabilistic estimates of efficiency of protective measures and to form evidence-based solutions for improving reliability and longevity of energy facilities throughout their life cycle — from design and construction to operation and modernization. As a result, there is a balance between safety, economic efficiency and regulatory requirements in the presence of significant uncertainties of external influences and system parameters [19].

Materials and Methods. The major research task is to design a complete mathematical model of earthquake-resistant protection that would reflect both the stochastic nature of the exciting effects and a nonlinear dynamics of various types of damping devices, supports parametric optimization and provides a probabilistic assessment of the reliability of the equipment being protected. Unlike the common approaches relying on overly simplified concepts of seismic impacts or overlooking a random nature of dynamic processes, the suggested formulation focuses on a statistically consistent description of input signals, precise consideration of non-linearities in energy dissipation subsystems, and strict alignment of the efficiency criteria with the risk and fault tolerance indicators. The major element of a model is a description of a seismic input as a random process with specified spectral-temporal characteristics. To this end, it is advisable that families of nonstationary Gaussian or quasi-stationary processes with an evolutionary power spectrum, accelerogram generators calibrated based on the normative response spectra as well as duration and intensity statistics are employed. This formulation makes it possible to reproduce the intensity variability, frequency drift, and time modulation correctly that are essential for a precise assessment of a response. A dynamic damping subsystem should be modeled in a nonlinear formulation taking the actual constitutive laws into consideration: bilinear and multistep hysteretic models for viscoelastic elements, force dependences for viscous dampers, models with temperature dependence and amplitude of deformations, possible limiting states (ledge, saturation). In order to enhance identifiability and computational stability, it is recommended that reduced descriptions based on operational models with experimentally verified parameter ranges are employed. As the tasks of adjusting dampers in actual structures have a multi-criteria nature, a model should include formalized objective functions and constraints: minimizing transmitted accelerations and displacements, limiting coupling forces, ensuring operational suitability under multi-level excitation scenarios, cost, weight, and manufacturability requirements. The parameters of dampers can be optimized within the framework of a probabilistic formulation by means of the methods of gradient stochastic optimization, evolutionary strategies or Bayesian optimization with surrogate models, which reduces computational costs considerably while maintaining the accuracy of the estimates. Probabilistic reliability assessment should be integrated into the computational circuit through risk-based design metrics. This entails calculating functions for exceeding limit states, estimating a probability of failure based on the selected performance criteria, and confidence intervals for the key responses. In order to boost the efficiency of Monte-Carlo methods, multilevel and multifid schemes, importance weighting, reduction of the dimension of the input space by means of stochastic modes based on extended Chebyshev/Legendre polynomials or Gaussian processes should be employed. From the perspective of numerical implementation, a modular architecture is preferred, combining direct temporal integration of a nonlinear system of equations of motion with adaptive step schemes and stable integrators, specialized solvers for non-smooth hysteretic laws, a probabilistic modeling unit that provides the generation of ensembles of input effects and statistical processing of responses. In order to speed up the calculations, it is advisable that reduction of POD/DEE-based models or trainable surrogates (e.g., physically informed neural networks) with an obligatory a posteriori validation on independent scenarios is employed. The suggested concept enables elimination of the limitations of the existing approaches by means of consistently accounting for the randomness of seismic excitation precisely describing nonlinear damping mechanisms, and directly linking parametric optimization with probabilistic reliability criteria [20]. This lays the foundation for practical design-oriented developments, provides scalability for different classes of structures and types of dampers, as well as reproducible quantitative estimates of the efficiency of protection systems with a controlled degree of uncertainty.

Let us consider a real example. The initial system parameters are as follows:

The turbine unit weight is $m = 50000$kg.

The natural frequency of the oscillating system is $f_0 = 2.5$ Hz.

The damping coefficient (for a linear case) is $\zeta = 0.02$.

The parameters of the Ornstein-Uhlenbeck random process are as follows:

The rate of return to the average $\theta = 1.5\text{sec}^{-1}$, intensity variance is $\sigma = 0.3\,\text{m/sec}^2$, mathematical expectation of excitation is $\mu = 0$.

The parameters of the viscoelastic damper are as follows:

The damping coefficient $c_d = 2 \cdot 10^5 N \cdot \mathrm{sec/m}$, indicator of the strength of non-linearity is $\sigma = 1.8$.

Stationary displacement dispersion with no damper. For a one-mass system undergoing random excitation of the Ornstein-Uhlenbeck type, the dispersion of stationary displacements is expressed as

$$\sigma_x^2 = \frac{\sigma^2}{2\theta\omega_0^2},$$

where $\omega_0 = 2\pi f_0$ is the natural angular frequency of a system.

Let us substitute the initial values:

$$\omega_0 = 2 \cdot 3.14 \cdot 2.5 \approx 15.71 \text{ rad/sec};$$

$$\alpha_x^2 = \frac{0.09}{2 \cdot 1.5 \cdot 15.71^2} \approx \frac{0.09}{739.5} \approx 1.22 \cdot 10^{-4} m^2.$$

Correction of damping in the presence of a nonlinear damper.

While using a viscoelastic damper with a nonlinear characteristic as

$$F = c_d x^\alpha.$$

its influence on the effective damping value in calculating the dynamic parameters is displayed. The effective damping coefficient is estimated approximately using the following formula:

$$\zeta_{eff} = \zeta + \frac{c_d + \sigma_x^{\alpha-1}}{2m\omega_0}.$$

Here σ_x is the the standard deviation of the velocity estimated for a nonlinear system. For the selected parameters

$$\zeta_{eff} \approx 0.02 + 0.089 = 0.109.$$

The decrease in the stationary displacement dispersion due to the increased damping can be approximately estimated as

$$\frac{\sigma_x^2}{\sigma_{x_0}^2} \approx \frac{\zeta}{\zeta_{eff}}.$$

Inserting the numerical values,

$$\frac{\sigma_x^2}{\sigma_{x0}^2} \approx \frac{0{,}02}{0{,}109} \approx 0.18,$$

or, which is equivalent to a 2.4-fold decrease in the RMS deviation of the displacements due to the action of the damper.

Hence the use of a nonlinear viscoelastic damper considerably boosts effective damping of asystem. The RMS values of the displacement response are thus reduced by more than two times helping to reduce seismic risks for a turbine unit, increase its dynamic stability and reliability in conditions of accidental impacts. This approach can be recommended for modernizing vibration protection of large power units in earthquake-prone regions.

Research Results. The study has displayed the high adequacy of the generalized Ornstein-Uhlenbeck process as a stochastic model for describing input seismic impacts employed in analyzing the dynamics of building structures. The use of the model is justified by means of its analytical transparency making it possible to obtain isolated expressions for the major statistical characteristics, including spectral densities, correlation functions, response variances, and probabilities of exceeding critical levels. This serves to greatly facilitate the solution of engineering problems in assessing and predicting the behavior of structures under the action of random dynamic impacts.

Unlike their linear analogues, damping devices with a nonlinear power characteristics have been shown to be capable of considerably increasing the efficiency of energy dissipation during random intense seismic vibrations. This is due to the capacity of nonlinear dampers to amplitude-dependent adaptation of a damping force, which causes a more uniform redistribution of energy across the spectrum of exciting frequencies. This adaptation helps to reduce a probability of local resonances as well as the level of extreme structural responses. The highest energy dispersing capacity is observed for dampers with a nonlinear index α in the range of 1.6–2.0, which has been confirmed by theoretical analysis and numerical modeling.

A comprehensive methodology for probability-based parametric optimization of damping systems has been developed allowing one to take statistical heterogeneity and regional features of seismic impacts into consideration. The suggested approach integrates parameter setting of the stochastic input process model with multi-purpose optimization of vibration protection efficiency with limitations on material resources and compliance with the current regulatory requirements. This methodology entails use of procedures for identifying statistical characteristics of seismic impacts in the area and adapting the parameters of damping devices taking probabilistic criteria for reliability and operational efficiency into consideration.

Discussion and Conclusion. The generalized Ornstein-Uhlenbeck process provides a solid mathematical and computationally efficient foundation for formalizing the stochastic nature of seismic impacts and analytically evaluating the dynamic response of engineering systems. The use of nonlinear dampers with a nonlinearity index α of the order of 1.6–2.0 is considerably superiority in terms of energy dissipation and limiting extreme responses compared to the traditional linear systems. The developed probability-based parametric optimization methodology opens up avenues for flexible adaptation of the characteristics of dampers to various regional features of seismic activity, taking the variability of input processes and specific features of construction sites into consideration.

In the future, it is recommended that experimental research is extended in order to verify mathematical models and to improve the accuracy of parameterization of materials and interface elements. On top of that, theory and practice of uncertainty analysis related to both the characteristics of seismic impacts as well as the physical and mechanical properties of damping devices must be developed. This would improve the overall reliability and engineering validity of design decisions in the field of earthquake-resistant construction and lay a foundation for introducing intelligent adaptive vibration protection systems based on state-of-the-art stochastic models.

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Modern Approaches to Studying the Aerodynamic Stability of Complex Curvilinear Buildings

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Abstract

Introduction. Modern architecture is characterized by the extensive use of buildings with complex curvilinear forms that are high expressive yet require tackling new engineering challenges associated with ensuring their aerodynamic stability. Normative methods for calculating wind loads are largely focused on buildings with a simple geometric shape and fail to account for the flow characteristics of free-form shells. This highlights the need to systematize modern approaches to analyzing wind effects on such structures. The aim of the study is to summarize and compare normative, experimental, and numerical methods for assessing the aerodynamic stability of complex-shaped buildings.

Materials and Methods. The object of the study is a building with a biomorphic three-beam structure characterized by smooth contours and a complex spatial topology. In order to analyze its aerodynamic characteristics, numerical simulation of wind flow was performed using the RWIND Simulation software. The study was conducted in order to identify the flow characteristics and distribution of aerodynamic loads on the surface of a complex-shaped building.

Research Results. As a result of the calculations, distributions of the pressure, velocity, and pressure coefficients over a building surface were obtained. Zones of a local pressure increase and dilution were identified in the areas of volume junctions and roof recesses. It was found that the curvilinear form of a building contributes to a reduction in the overall aerodynamic drag; however, it also induces the formation of local vortex structures, which is to be considered in designing façade and roofing systems.

Discussion and Conclusion. The results confirm the effectiveness of applying Computational Fluid Dynamics (CFD) methods for analyzing the aerodynamic properties of complex-shaped buildings. The integrated use of normative, experimental, and numerical approaches ensures a more accurate assessment of wind effects and contributes to developing a cutting-edge methodology for designing aerodynamically stable architectural structures.

Keywords: aerodynamic stability of buildings, wind effects, complex-shaped buildings, curvilinear shells, computational fluid dynamics, numerical simulation, aerodynamic pressure, vortex flow structures, wind loads on structures

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Оригинальное эмпирическое исследование

Современные подходы к исследованию аэродинамической устойчивости зданий сложной криволинейной формы

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Аннотация
Введение. Современная архитектура характеризуется активным использованием зданий сложной криволинейной формы, обладающих высокой выразительностью, но требующих решения новых инженерных задач, связанных с обеспечением их аэродинамической устойчивости. Нормативные методы расчёта ветровых нагрузок ориентированы преимущественно на здания простой геометрической формы и не учитывают особенностей обтекания свободных оболочек. Это обусловливает необходимость систематизации современных подходов к анализу ветрового воздействия на подобные сооружения. Целью настоящего исследования является обобщение и сравнение нормативных, экспериментальных и численных методов оценки аэродинамической устойчивости зданий сложной формы.
Материалы и методы. Объектом исследования является здание с биоморфной трёхлучевой структурой, характеризующееся плавными очертаниями и сложной пространственной топологией. Для анализа аэродинамических характеристик выполнено численное моделирование ветрового потока с применением программы RWIND Simulation. Исследование выполнено с целью определения особенностей обтекания и распределения аэродинамических нагрузок на поверхность здания сложной формы.
Результаты исследования. В результате выполненных расчетов получены распределения давления, скоростей и коэффициентов давления по поверхности здания. Выявлены зоны локального повышения и разрежения давления в областях сопряжения объёмов и углублений кровли. Установлено, что криволинейная форма здания способствует снижению интегрального аэродинамического сопротивления, однако вызывает образование локальных вихревых структур, которые необходимо учитывать при проектировании фасадных и кровельных систем.
Обсуждение и заключение. Полученные результаты подтверждают эффективность применения методов вычислительной гидродинамики (CFD — Computational Fluid Dynamics) для анализа аэродинамических свойств зданий сложной формы. Комплексное использование нормативных, экспериментальных и численных подходов обеспечивает более точную оценку ветровых воздействий и способствует формированию современной методологии проектирования аэродинамически устойчивых архитектурных сооружений.

Ключевые слова: аэродинамическая устойчивость зданий, ветровое воздействие, здания сложной геометрии, криволинейные оболочки, вычислительная гидродинамика, численное моделирование, аэродинамическое давление, вихревые структуры обтекания, ветровые нагрузки на сооружения

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Introduction. The architecture of the 21st century is characterized by a surge in the number of buildings and structures of complex curvilinear shapes. In conjunction with use of new composite materials and systems of enclosing structures, modern technologies of design, modeling and construction enable implementation of architectural ideas that used to be considered utopian. Renewed interest in curvilinear architectural structures is a testimony to the reemergence of the trends that date back to the late 20th century, when plastic and biomorphic forms started being considered as a tool of combining aesthetics, functionality and engineering rationality [1]. Concert halls with shell coverings, museums with asymmetric volumes, sports arenas with domed and combined roofs, as well as high-rise complexes with facades containing bends, twisted and torsion elements are emerging in the largest global megacities. Such structures are a cornerstone of the modern architectural image of cities becoming visual representations and symbols of a regional cultural identity.

The growing proprotion of buildings with a non-standard geometry is posing new engineering challenges. One of the major factors critical to safety, operational reliability and durability of such structures is their aerodynamic stability under the influence of wind loads [2]. Unlike the traditional rectilinear or cylindrical shapes, curvilinear structures create a complex structure of airflow flow accompanied by intense eddy formation, local separation zones and pressure concentration [3]. These cause loads to distribute unevenly over the surface, enhance oscillatory processes and in some cases might generate resonant effects. On top of that, local high-pressure areas might damage facade systems, translucent structures, and roofing elements. In the context of dense urban development, the interaction between adjacent buildings exarcebates the aerodynamic pattern increasing local vortex zones as well as pressure on enclosing structures [4].

The issue of an adequate assessment of a wind impact on buildings of complex shape is a daunting one. In engineering practice, regulatory methods fixed in national and international standards are still dominant. In Russia, the main documents are SP 20.13330.2016 "Loads and Impacts" and SP 267.1325800.2016 "Buildings and Structures. Design Rules for Wind Impacts", regulating methods for identifying calculated wind pressure, aerodynamic drag coefficients and dynamic characteristics. Overseas similar provisions are contained in Eurocode EN 1991-1-4, ASCE 7-22, AIJ Recommendations for Loads on Buildings (Japan) and GB 50009-2012 (China). These documents define the core principles of calculating wind loads, but their application to objects with complex geometries is restricted as the calculated dependencies have been developed for bodies of simple shapes — prismatic, cylindrical and domed ones.

At the same time, experimental and numerical methods for investigating wind impacts are actively developing. Wind tunnel aerodynamic tests provide a high degree of accuracy and allow identification of local pressure concentration zones, flow separation and formation of turbulent structures. Field measurements on actual structures provide an insight into the actual behavior of buildings in conditions of natural wind, however, they call for considerable financial and time resources.

Numerical methods based on solving Navier-Stokes equations have been developed within the CFD approach that involves discretization of a calculation domain and numerical solution of equations of motion and conservation of mass for a flow being studied. Use of CFD makes it possible to analyze nonstationary three-dimensional aerodynamic processes around objects of almost any configuration as well as to obtain detailed fields of velocities, pressures, and vortex structures. The accuracy of such calculations is largely due to a choice of the turbulent model (RANS — Reynolds-Averaged Navier-Stokes, LES — Large Eddy Simulation and hybrid DES — Detached Eddy Simulation) [5, 6], correctness of the boundary conditions and a degree of detail of a calculation grid.

The state of the art in the field of aerodynamic stability of buildings of complex geometry is thus characterized by a broad range of approaches, but the shortage of a unified methodology to ensure both the accuracy of calculations and their practical applicability. This serves to highlight the need for a comprehensive analysis of architectural and construction aerodynamics considering the relationship between shape, structure and air flows [7]. There is an imperative to systematize the existing methods, compare their capacities and limitations as well as to develop a strategy for further development of this scientific and technical field.

The aim of the study is a comprehensive analysis of the modern approaches to studying wind effects on buildings of a complex curvilinear shape. The paper discusses the provisions of the regulatory framework, experimental methods of aerodynamic testing and modern CFD modeling technologies. Special attention is paid to a comparative assessment of accuracy, computational efficiency and integration of these methods within a single methodological system.

Materials and Methods. The study of the aerodynamic stability of buildings with complex curvilinear shapes relies on the analysis of the three major groups of methods: normative, experimental and numerical (CFD) ones. This division reflects the state of the art in engineering practice and allows comparisons of their accuracy, scope and limitations.

Normative methods for calculating wind effects are based on generaliing statistical data from long-term meteorological observations and results of aerodynamic experiments for typical building geometries. In domestic practice, calculations are regulated by the document SP 20.13330.2016 "Loads and Impacts" where the estimated wind speed and pressure are identified taking the wind area, terrain category and height of the structure into consideration. According to modern research, the existing regulatory dependencies developed mostly for buildings of a simple shape demonstrate a limited scope of applicability in analyzing objects with a complex or curvilinear geometry leading to a decrease in the accuracy of calculations of aerodynamic loads [8].

Experimental methods include aerodynamic tests in wind tunnels and field measurements making it possible to obtain pressure and velocity distributions along the surface of building models [2, 9]. Modern research makes use of highly sensitive pressure sensors, as well as flow visualization techniques such as PIV (Particle Image Velocimetry) and digital anemometry. Use of scale models of fragments of urban development enhances the realism of the resulting data, which is of particular importance for buildings located in a dense urban environment [10]. In spite of their high degree of accuracy, such tests call for considerable material and time costs with their results frequently limited to specific flow scenarios. Experimental methods are thus largely employed in order to verify numerical models and refine turbulence parameters.

Numerical methods (CFD) are a universal tool for analyzing wind impacts on buildings of complex shapes. They make it possible to investigate three-dimensional unsteady flows and evaluate the influence of a shape, angle of attack, and surface roughness on the aerodynamic characteristics [11]. As part of this study, a computational model of the building was deisgned in the Rhinoceros 3D environment and exported to the RWIND Simulation software package that implements a method for finite-volume solution of Navier-Stokes equations in a stationary formulation using the turbulent k–ω SST model [12, 13]. The program automatically generates an adaptive finite-volume grid thickened in areas of an increased curvature and expected flow separation. Depending on a tasks at hand, various turbulent models can be employed - RANS, LES, and hybrid DES [5, 6, 14]. CFD analysis provides detailed pressure fields, velocities, and turbulent energy, visualization of vortex structures, and identificaton of aerodynamic coefficients. Special attention is paid to the quality of a calculation grid and correctness of the boundary conditions the reliability of the results depends on [15].

The integrated application of normative, experimental and numerical approaches provides a comprehensive assessment of the aerodynamic stability of buildings. Normative methods set the initial design guidelines, experimental tests refine the pressure distribution, and CFD modeling indicates the spatial structure of the flow and zones of local aerodynamic effects. This integrated approach lays the foundation for the modern methodology for analyzing and designing buildings of a complex shape ensuring their stability, safety and durability.

Research Results. The object of the numerical analysis is a building with a smooth curved architectural form with no clearly defined faces and a three-ray structure typical of modern biomorphic forms (Fig. 1). According to the plan, the building is a three-ray structure formed by three interconnected oval volumes united in the central part. This geometry makes it possible to investigate the features of the interaction of flows in the areas of volume coupling and those with variable surface curvature. The streamlined contour of the building contributes to the uniform redistribution of the incoming airflow and reduces the intensity of local vortex formations.

Fig. 1. Architectural rendering of the object

The roof surface has a continuous smoothed geometry with three large oval depressions with glazed dome structures inside it. These depressions form closed contours that smoothly interface with the main body of the building, which avoids sudden changes in the curvature and minimizes the likelihood of areas of a local increase in the aerodynamic drag. Visually, the shape of the building resembles a biomorphic shell seamlessly integrated into the surrounding space and adapted to the wind flow direction.

A digital geometric model was designed in the Rhinoceros 3D environment using the NURBS representation of geometry (Fig. 2) making it possible to accurately reproduce the curved shape of the shell with controlled radii of curvature and ensure a high degree of correspondence between the architectural concept and the calculated model [11, 15]. Based on the designed model, the surface was prepared for CFD analysis in the RWIND Simulation software environment [3, 14, 15] where algorithms for generating an adaptive finite-volume grid and modeling wind flows around complex spatial shapes are implemented.

Numerical analysis of aerodynamic stability was performed in RWIND Simulation using the finite volume method for solving Navier-Stokes equations in a stationary formulation [3, 11]. K–ω SST was employed as a turbulent model [12] that provides an accurate description of the flow in the boundary layers and correct modeling of flow separation zones on curved surfaces [3, 14]. A three-dimensional computational domain was defined for the calculation that ensures the free development of the flow with no boundary effects. A uniform velocity profile with a maximum value of 41.2 m/s was set at the input. The outlet boundary was identified by the condition of zero pressure gradient, upper and side surfaces by the condition of free sliding. The surface of the building was assumed to be rough making it possible to take the influence of the microrelief of the roof and facade elements into consideration.

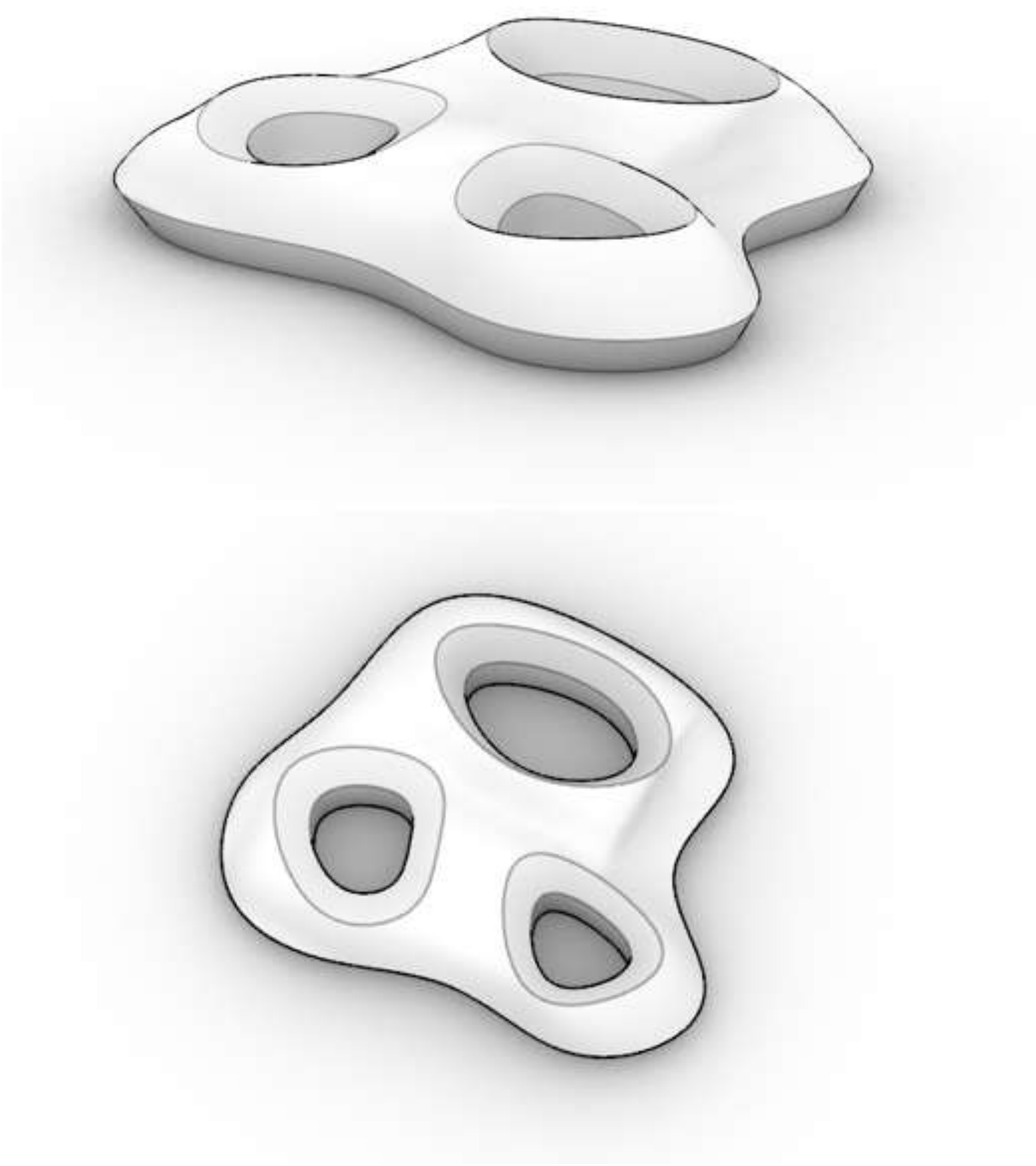

Fig. 2. Digital NURBS-shell model

The results of the numerical simulation include three-dimensional fields of static pressure, aerodynamic coefficient *(c)*, velocities and trajectories of an air flow. Fig. 3 shows the distribution of static pressure over the building surface. The maximum positive values of +574 Pa are observed in the windward sections of the shell, mostly in the front convex zones and in the central part perpendicular to the wind direction. In the areas between the oval depressions and in the rear part of the structure, negative pressure values up to –864 Pa are recorded, which is indicative of the formation of rarefaction zones and local vortex structures.

Fig. 4 shows the distribution of the aerodynamic coefficient *C*. The highest values (C = 1.02) occur in the frontal sections of the roof where the air flow is inhibited. The minimum values (C = –1.54) are observed in the depressions and behind the rear edges of the volumes where recirculation zones are formed, i.e., areas of reverse flow movement accompanied by reduced pressure and eddy formation. Data analysis confirms that there are stable vortex structures in the inter-petal regions causing pressure pulsations and an increase in the dynamic component of the wind load [13].

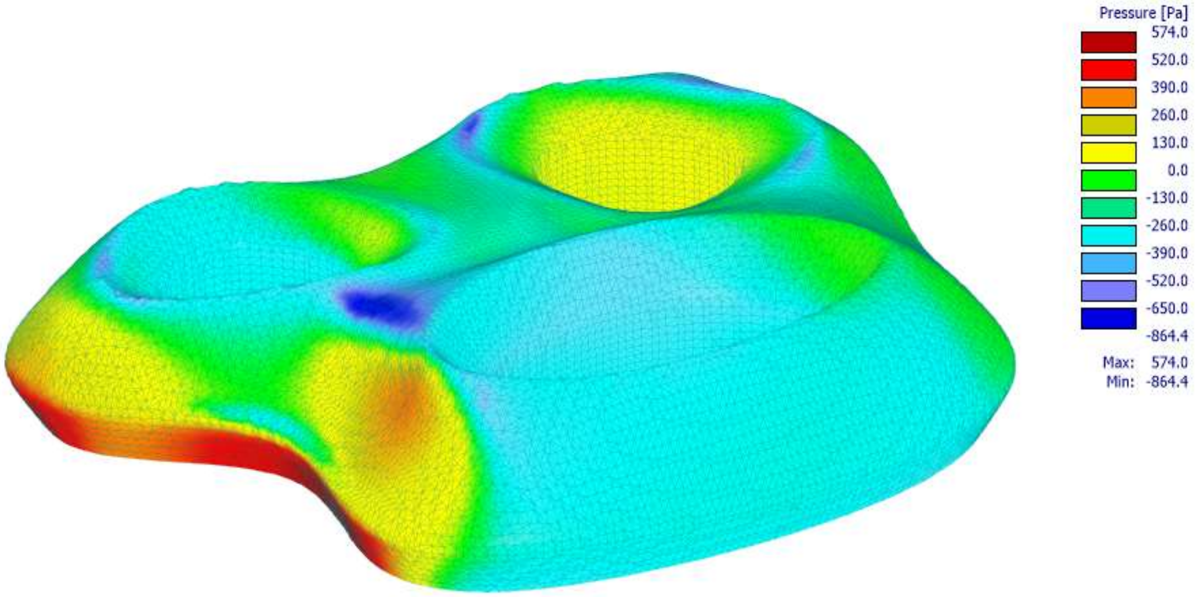

Fig. 3. Static pressure on the building surface

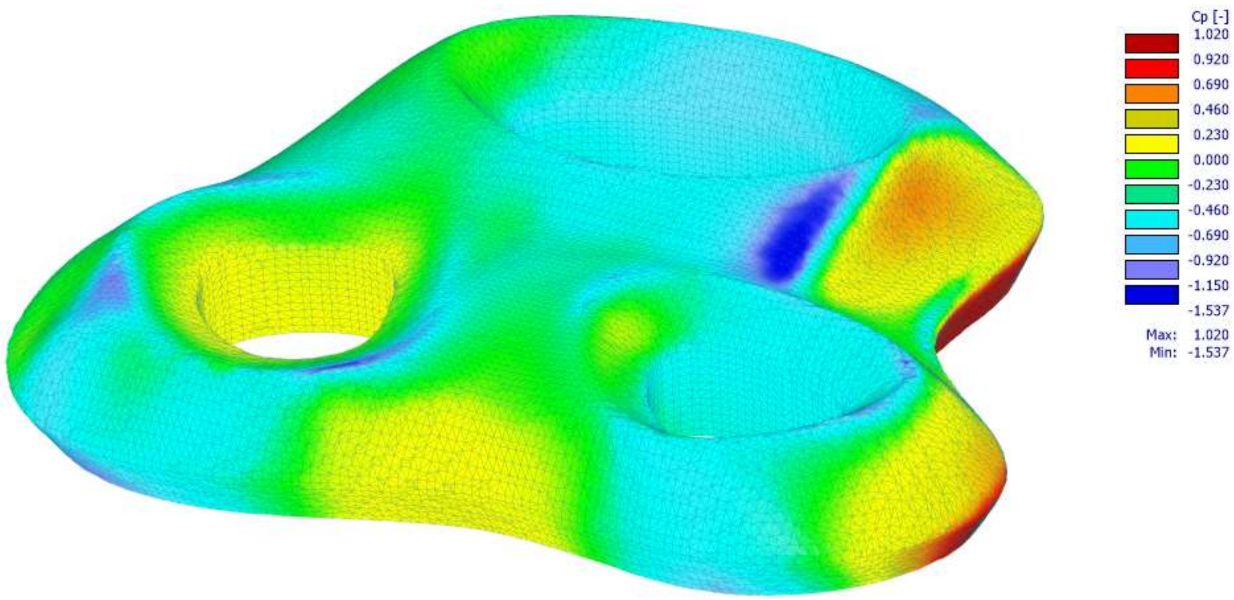

Fig. 4. Pressure coefficient distribution C_p

Fig. 5 shows the velocity distribution in the flow section. Along the surface of the building, the flow velocity decreases to 3–5 m/s in the boundary areas and vortex formation zones, whereas the acceleration to 30–35 m/s is observed between the main volumes. A large-scale vortex wake forms above the central part gradually fading as it recedes. This flow structure is typical for aerodynamically streamlined but geometrically complex shapes combining smooth transitions and the mutual influence of conjugated shell elements.

Numerical modeling has thus confirmed the high aerodynamic efficiency of curvilinear architectural forms in the presence of smooth interface surfaces. The results make it possible to quantify the distribution of wind loads and visualize the flow structure, which is a critical step in optimizing the architectural shell and increasing the reliability of complex-shaped building structures.

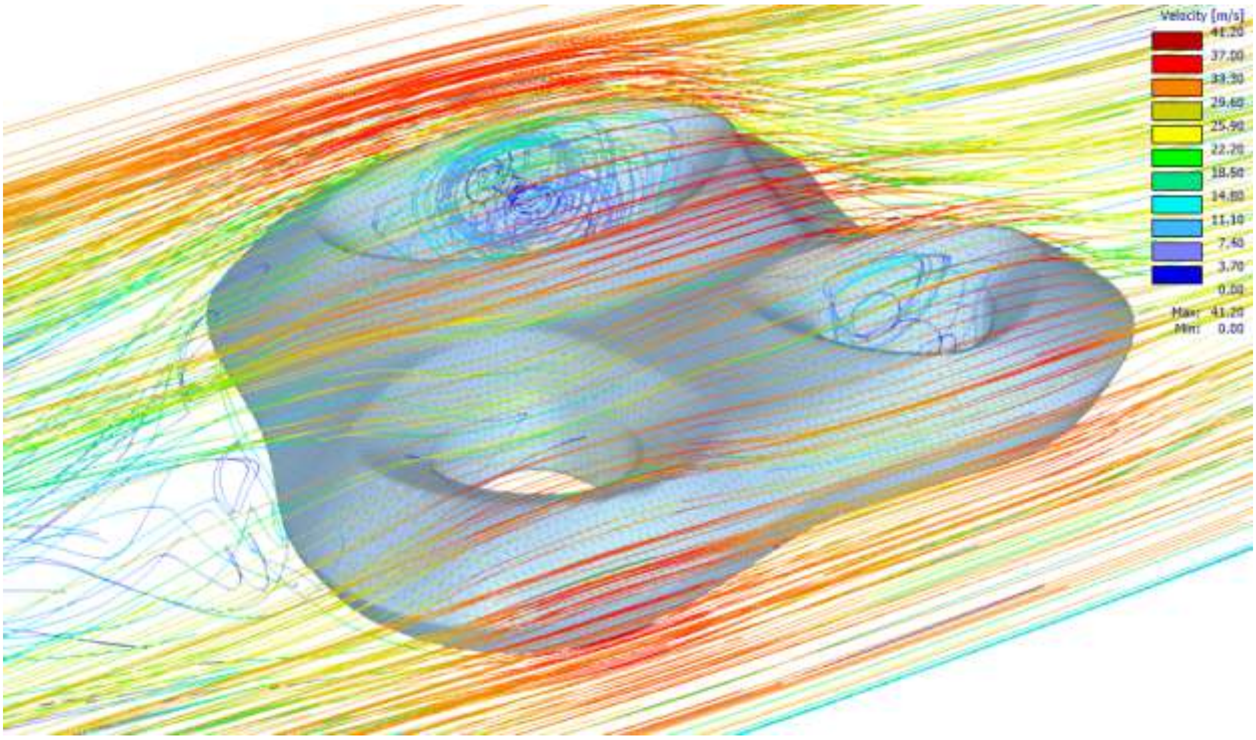

Fig. 5. Velocity distribution in the flow section

Discussion and Conclusion. The research has shown that architectural objects with a distinct curvilinear geometry have not only a high aesthetic and functional value [1], but also place increased demands on the aerodynamic stability and accuracy in calculating wind impacts. The analysis of normative, experimental, and numerical approaches has confirmed that the traditional methods based on the simplified models and averaged coefficients are not sufficiently accurate while calculating complex-shaped buildings. At the same time, use of modern computational aerodynamics tools can serve to considerably increase the reliability of projections and detail a flow pattern.

Using the example of a building with a three-beam shell, it is shown that the combination of Rhinoceros 3D and RWIND Simulation software environments provides a possibility of a comprehensive analysis of the aerodynamic characteristics. Use of the turbulent k–ω SST model [12] made it possible to correctly reproduce vortex structures and identify the pressure distribution over the surface as well as the areas of local loads and potential instability. The results are consistent with data from overseas studies [11, 13] and confirm the efficiency of the CFD approach in analyzing buildings with a free architectural plastics.

Integration of the numerical analysis methods is at the core of the modern methodology for designing buildings of complex geometry ensuring the relationship between an architectural concept and engineering reliability. Use of CFD modeling in the early stages of design makes it possible to identify unfavorable aerodynamic zones, optimize the geometry of the shell and prevent resonant vibrations. This approach contributes to enhancing safety, durability and energy efficiency of structures and is in compliance with the principles of sustainable architectural development.

The prospects for further research are integrating the numerical and experimental methods, designing unified databases of aerodynamic coefficients for typical geometries as well as introducing digital building doubles for monitoring wind impacts in the operating conditions [16]. On top of that, the urgent task is yet to improve the national regulatory framework including SP 20.13330.2016 and SP 267.1325800.2016 bringing their provisions in compliance with the international standards Eurocode EN 1991-1-4, ASCE 7 and AIJ Recommendations.

The implementation of these directions will enable a single comprehensive methodology for analysis and design ensuring improved calculation accuracy, operational reliability and stability of buildings of a complex curvilinear architecture.

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NA Buzalo: scientific supervision, analysis of the research results, revision of the manuscript, correction of the conclusions.

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Original Empirical Research

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Extended Wooden Panels for Flooring and Cladding of Buildings

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Abstract
Introduction. The development of the construction industry in Russia involves both emerging technologies and materials and traditional construction methods. One of the well-known tools in this country is wooden housing construction. Apartment buildings are being built from wooden CLT panels, glued timber, beams made of unidirectional veneer, etc. are commonly employed. Modern architecture seeks to create large open spaces for free planning. In wooden buildings, it is not always possible to organize such spaces due to a limited length of lumber being produced. Studies aimed at designing extended panels from wood are thus gaining momentum. The aim of this study is to develop new designs of wooden panels for cladding and flooring of buildings from standard lumber, plywood and oriented strand boards with spans exceeding the standard length of boards, to identify the limits in the load-bearing capacity of such panels, as well as to conduct their geometric calculation.
Materials and Methods. Two types of box-shaped panels made of wooden planks, plywood and/or oriented strand boards are considered. The load-bearing capacity of the suggested structures is estimated by means of both traditional methods of material strength and computer models.
Research Results. The design of extended panels is described differing from the known overseas analogues and is free from the inherent disadvantages of the latter. Geometric calculation of the suggested structures is performed. The rational size ratios of the sizes of boards that make up the panels are identified. Design limitations for individual elements of products are established. Computer models of the panels are designed and employed in order to identify the applicability limits of the suggested structures.
Discussion and Conclusion. As a result of the research, new designs of wooden panels for flooring and cladding of buildings with possible spans exceeding the standard length of lumber and made with no use of costly materials have been developed. The simplicity of the design makes it possible to organize manufacturing of products in small-size industries with complex and costly equipment involved. The panels can be employed for long open spaces in wooden buildings and structures for a broad range of purposes.

Keywords: panel, wood, lumber, plywood, flooring, cladding, long spans

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Оригинальное эмпирическое исследование

Протяженные деревянные панели перекрытий и покрытий зданий

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Аннотация
Введение. Развитие строительной отрасли России предполагает как появление новых технологий и материалов, так и использование традиционных методов строительства. Одним из хорошо известных методов в нашей стране является деревянное домостроение. В настоящее время возводятся многоквартирные дома из деревянных

CLT-панелей, широко используется клееная древесина, балки из однонаправленного шпона и т.п. Современная архитектура направлена на создание значительных открытых пространств для свободной планировки. В деревянных зданиях не всегда удается организовать подобные пространства из-за ограниченности длин выпускаемых пиломатериалов. В связи с этим актуальными становятся исследования, направленные на создание протяженных панелей из древесины. Целью настоящей работы является разработка новых конструкций деревянных панелей покрытий и перекрытий зданий из стандартных пиломатериалов, фанеры и ориентированностружечных плит с пролетами, превышающими стандартную длину досок, определение пределов несущей способности таких панелей, а также их геометрический расчет.

Материалы и методы. Рассмотрены два типа панелей коробчатого строения, выполненных из деревянных досок, фанеры и/или ориентированностружечных плит. Оценка несущей способности предлагаемых конструкций произведена как при помощи традиционных методов сопротивления материалов, так и на компьютерных моделях.

Результаты исследования. Описана конструкция протяженных панелей, отличающаяся от известных зарубежных аналогов и свободная от присущих последним недостатков. Выполнен геометрический расчет предлагаемых конструкций. Определены рациональные соотношения размеров досок, составляющих панели. Установлены конструктивные ограничения для отдельных элементов изделий. Созданы компьютерные модели панелей, при помощи которых установлены пределы применимости предлагаемых конструкций.

Обсуждение и заключение. В результате проведенных исследований разработаны новые конструкции деревянных панелей перекрытий и покрытий зданий с возможными пролетами, превышающими стандартную длину пиломатериалов и выполненные без использования дорогостоящих материалов. Простота конструкции позволяет организовать выпуск изделий на небольших производствах без сложного и дорогостоящего оборудования. Панели могут применяться для создания протяженных открытых пространств в деревянных зданиях и сооружениях различного назначения.

Ключевые слова: панель, древесина, пиломатериалы, фанера, перекрытие, покрытие, протяженные пролеты

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Introduction. Extended open spaces have been embraced in modern civic architecture. For them, special structures of flooring and cladding – most frequently made of metal – are typically used. In wooden buildings, open spaces are commonly limited by the size of standard lumber and are not beyond six meters. In order to design large facilities, beams made of glued wood or products made of unidirectional veneer lumber (LVL) are to be employed with their spans depending on the cross-section sizes. The use of these beams increases the cost of a frame structure and considerably enhances the thickness of flooring and roofing. Extended box-shaped panels where the power bearing elements are "smeared" over the panel area are thus of interest.

One of the possible variants of such products are Kielsteg panels shown in Fig. 1 and allowing flooring of spaces up to 27 meters long [1]. It should be noted that in this country a similar design was patented as early as in 2010 [2].

These products have some design features that arguably reduce their effectiveness. As wood performs better in compression than in tension, it is advisable that a possibility of using boards of different thicknesses for upper and lower paneling is explored. Plywood ribs bent during the manufacture of panels experience prestressing, which must be considered in design. Finally, the edges converging at the same joint form hard-to-reach spaces inside the panel to accommodate effective insulation in them. It should be noted that laminated wood panels can have a highly diverse structure [3–5], and only two of the possible variants of such products are going to be considered.

Along with the flat coverings, the panels discussed can also be used in vaults. Wooden arches are one of the oldest and most interesting types of engineering structures in use. Initially, they emerged as a result of arched structures for overlapping sufficiently large spaces with no intermediate supports [6]. In ancient Rome, the Pantheon and the Colosseum came to be known as iconic structures using vaults. The structures were further developed in the Middle Ages when thin-walled cladding emerged that was capable of withstanding significant loads with a minimum thickness, e.g., the Notre Dame Cathedral in Paris. Modern architecture makes use of new approaches to designing vaulted structures, including use of composite materials, parametric design by means of a special software, etc.

It is obvious that for a comprehensive review of the suggested panels, it is necessary to address some issues with the main one being design of products and materials used to create them, as well as the geometric calculation of flat and vaulted panels.

a) *b)*

Fig. 1. Wooden floor panel by Kielsteg: *a* — cross section; *b* — an example of the use of panels [1]

It is also important to identify the method of assessing the stress-strain of the panels. Various approaches are employed to this end [7, 8]. However, the finite element method seems to be the most universal one allowing almost all the design features to be considered.

Use of renewable natural environmentally friendly materials in structures of buildings is becoming a modern trend in the construction industry. In the Russian Federation, more and more attention is being paid to wood use in construction, but the limited size of lumber makes it impossible to cover significant spans with no metal or massive glued structures. The creation of new products with lower material consumption compared to traditional structures is an urgent task [9], and their use in cladding of significant spans, including vaulted ones, opens up avenues for creating architecturally attractive objects.

The aim of the study is to evaluate a possibility of using the suggested panels as load-bearing structures of flooring and cladding, to identify their maximum spans and optimal ratios between the dimensions of the components of the product elements.

The tasks to be addressed to this end are as follows:

– geometric calculation of panels and obtaining analytical dependencies to identify the dimensions of all of the components of a product;

– assessment of the stress state of the forcibly curved edges of the middle layer of panels;

– designing parametric computer models of structures.

Materials and Methods. Two types of box-shaped panels with plank sheaths and ribs made of plywood or oriented strand boards (hereinafter referred to as OSB) are discussed. The panels consist of recurrent modules with the size shown in Fig. 2. The modules differ from each other in the shape of the edges. The first module has pre-curved edges, and the second has rectilinear ones. Each of the suggested options has its advantages and disadvantages. In the first case, chamfered boards are used, however, certain effort must be applied to pre-bend the ribs, which might require presses, and the ribs themselves will have lower safety margins due to prestressing. In case of rectilinear ribs, chamfering of the skin boards becomes necessary, but the ribs do not have prestressing.

The basic dimensions of the panels are their height *H*, the thicknesses of the skins and ribs t_1, t_2 and *t*, respectively, as well as the width of the lumber *b*. The remaining dimensions shown in Fig. 2 are identified using the basic ones.

Modules for vaulted cladding come in a larger number of sizes and are more challenging to manufacture (Fig. 3).

A numerical-analytical approach was employed for the stated aim of the study. At the same time, the geometric calculation of the panels is performed analytically, and the obtained dependencies are applied in the corresponding program. Computer models of the panels were created in the SolidWorks software package.

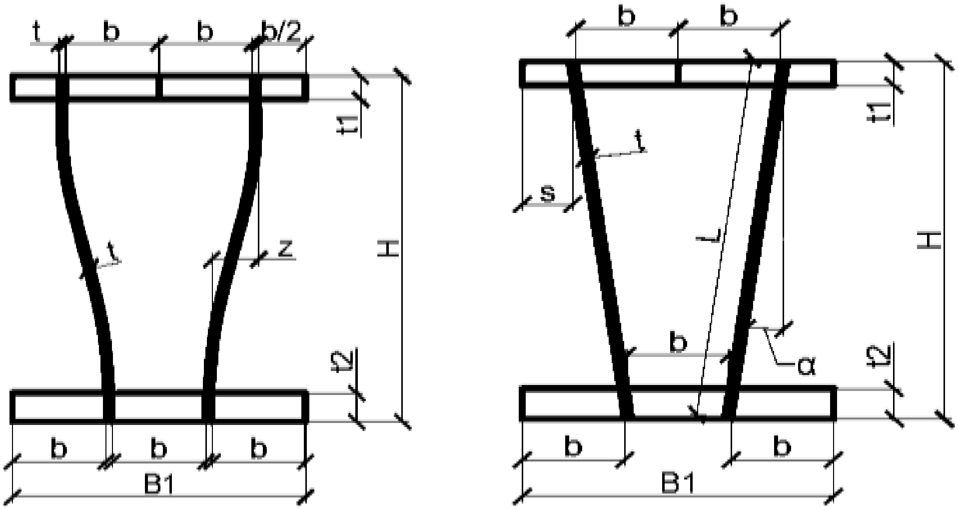


Fig. 2. Flat panel modules

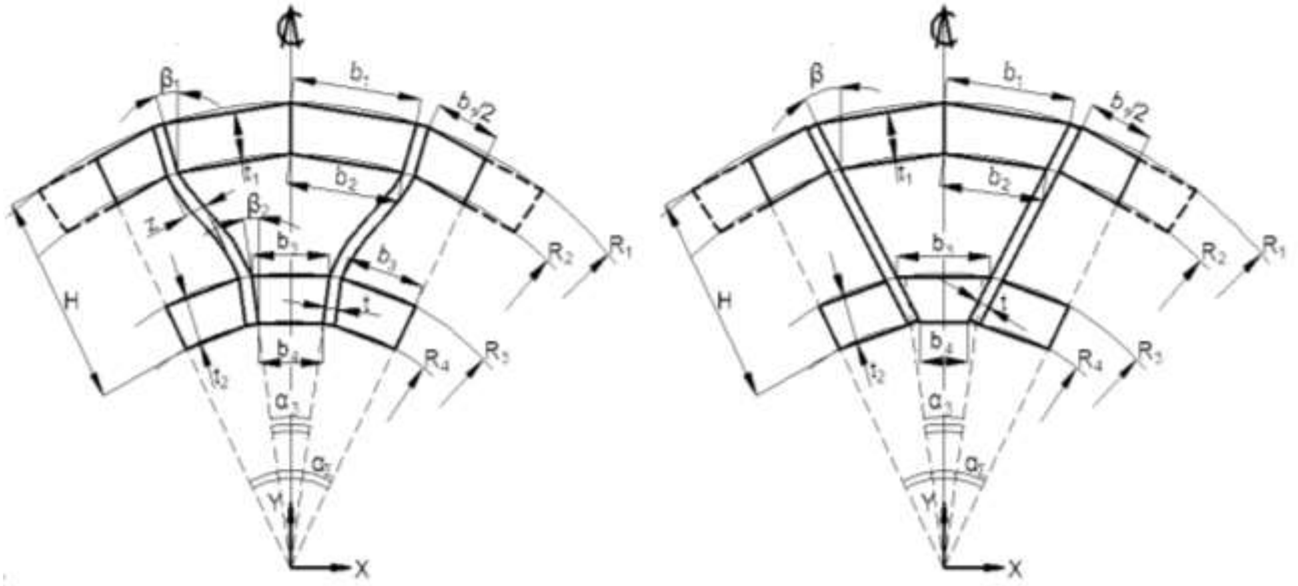


Fig. 3. Vault panel modules

Research Results. The first stage of the study was a geometric calculation of the panels. If in case of flat panels it turns out to be fairly simple, for the arch elements it is a rather laborious process due to a need to identify the central angles of the edges at radii R_1 and R_4, as well as the bevels of the cladding boards depending on the specified parameters. The problem had to be addressed using analytical geometry methods, and, e.g., to identify all the dimensions of the module of vaulted cladding with pre-curved edges, it was necessary to calculate more than fifty parameters.

The solution to the problem of geometric calculation of the vaulted panel is implemented in the Microsoft Excel software. Fig. 4 shows a workbook sheet for a module with curved edges where the width of the board b_3 is initially calculated so that the central corner of the lower skin is equal to the angle α_Σ (the designations are shown in Fig. 3), and a text file is then written with the data necessary for a generative code. A module with rectilinear edges is calculated in the same manner.

The curved edges of the panels are pre-bent during the panel manufacturing process as well as a plate under cylindrical bending. Obviously, to this end, some pressure must be applied to the rib that in this case can be identified using the methods of material resistance. A cantilever beam with a span equal to the length of the rib section between the board sheaths is considered whose free end a concentrated force and bending moment are applied to (Fig. 5).

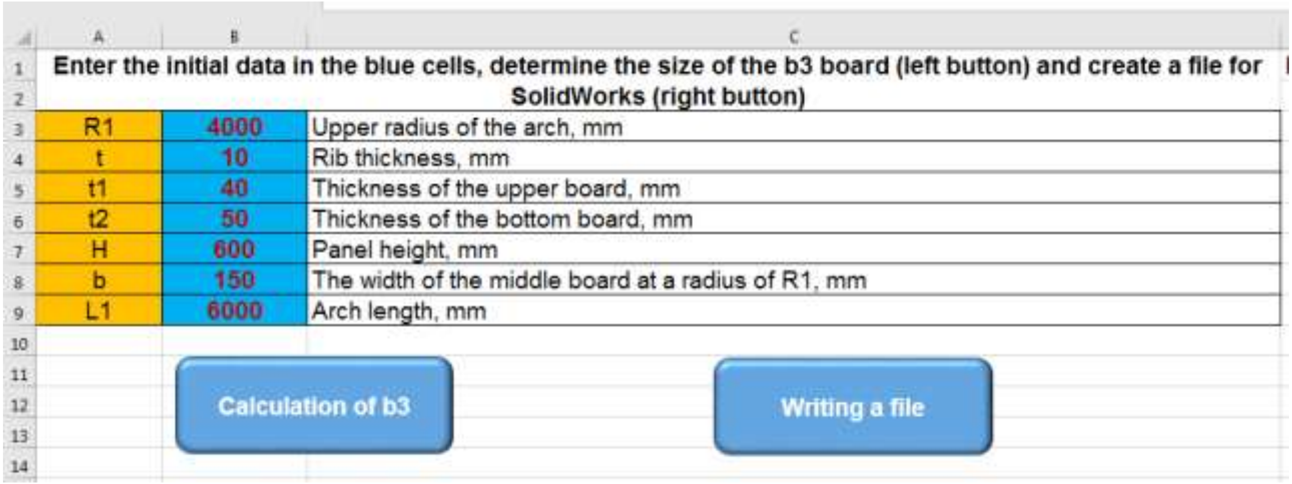


Fig. 4. Program for geometric calculation of a vaulted panel module with curved edges

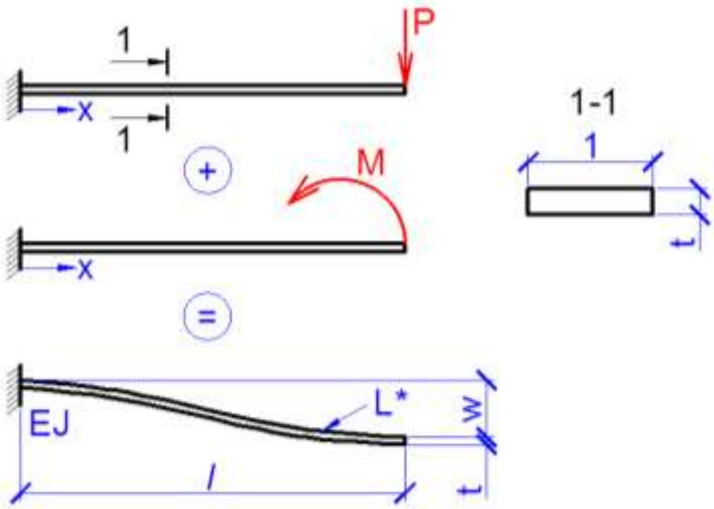


Fig. 5. Identifying the length of a work piece of the curved part of the rib

The ratio of these force factors is selected so that the specified deflection of the beam is ensured while the angle of rotation of its free end is zero. Obviously, in this case, the bending moment can be expressed in terms of a concentrated force, so the equation of the curved axis of the beam will have the form shown below. Given the fact that the stresses in the rib should not be beyond the calculated bending resistance, the values of the maximum deflection of the rib and the force creating it will be identified using the following expressions:

$$w(x)=\frac{Px^2}{Et^3}(3l-2x);\ w_{\max}=\frac{R_u l^2}{3Et};\ P=\frac{R_u t^2}{3l}.$$

Here R_u is the calculated bending resistance of the rib material with the remaining designations clear from Fig. 5.

In manufacturing panels, it is recommended that plywood or OSB sheets used as ribs are positioned so that the fiber direction of the outer layers is parallel to the thickness of a product.

The length of the work piece of the curved edge section is identified using an elliptical integral that takes the following form:

$$L^*=\int_0^l \sqrt{1+\left[\frac{Px}{2EJ}(l-x)\right]^2}\,dx.$$

As a result of a numerical experiment where the distance between the panel skins and the amount of preliminary bending of the rib varied, it was found that the length of the work piece of the curved section of the rib is slightly greater than the one between the skins and can be found using the formula

$$L^*=l+0{,}01w_{\max}^2.$$

All of the above actions were necessary in order to identify the stress-strain of the arches performed in the SolidWorks software package. This software tool provides a broad range of possibilities and can be used for modelling the behavior of structures made of new materials, accounting for the joint work of sheaths and a rib frame [10], as well as for comparing different design options [11]. E.g., the resulting three-dimensional model of an object in case of using global variables can change when the specified variables can change in an external text file, which is handy in numerical experiments.

In order to design a panel model in SolidWorks, it sufficed to form separate modules a so-called assembly was made from that included a required number of modules. Furthermore, in the Simulation software the object was automatically divided into solid-state volumetric finite elements, support anchorages and operating loads were specified followed by a static calculation.

As an example, Fig. 6 shows a module with curved edges and global variables from a Microsoft Excel text file.

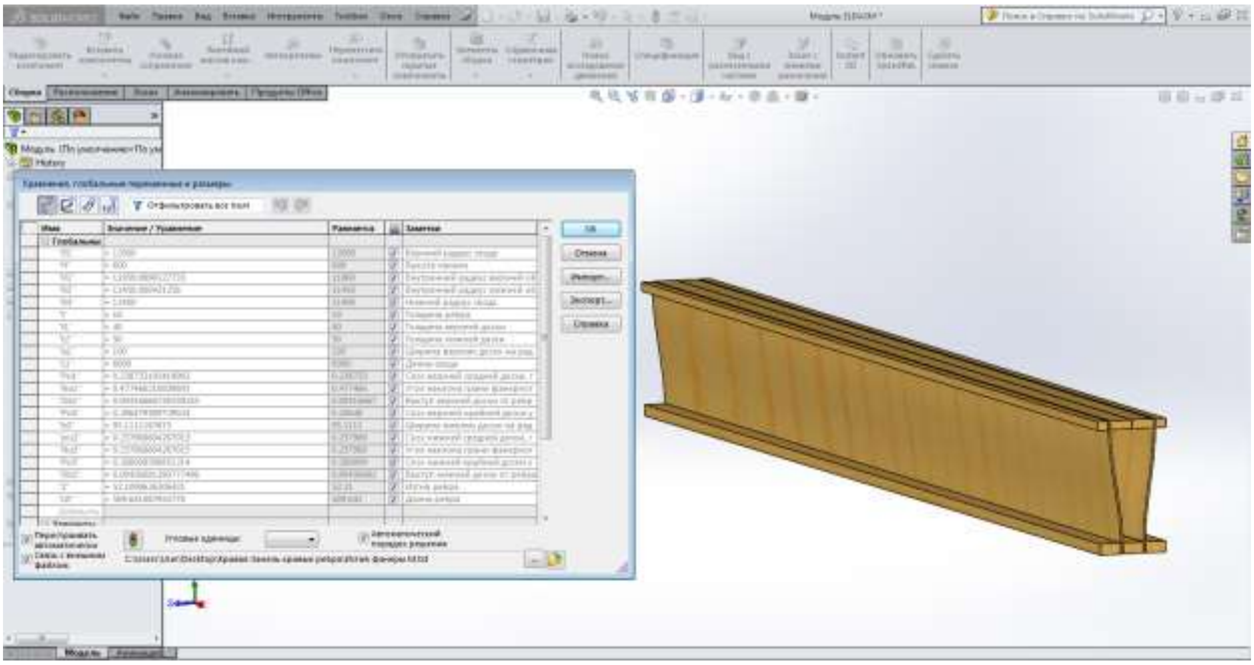
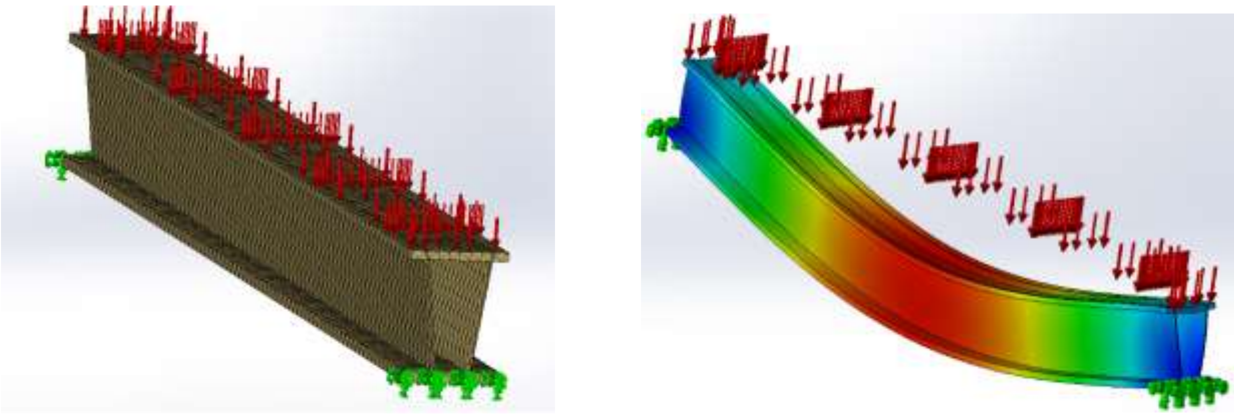


Fig. 6. Type 1 vault module model

In real-world design, it is more handy for engineers to use reference data that would provide rational dimensions of cladding elements depending on the geometric parameters of an object and natural and climatic area of construction. Obtaining such data is possible based on the results of a large numerical experiment where at least two hundred different module variants will have to be considered. The aim of the experiment is to identify the appropriate parameters of the cladding panels that could be written down in the form of simple formulas or tables, i.e., to design an engineering methodology for the above structures.

A possibility of designing such a technique is implicitly confirmed in [1] that examines Kielsteg panels, as well as the results of preliminary computer calculations for operational loads of individual modules (Fig. 7).



a) *b)*

Fig. 7. Computer simulation of Type 2 module: *a* — finite element model, *b* — deformed circuit

Discussion and Conclusion. As a result of the literature review, new types of cladding panels and floors made of wood, plywood and/or OSB have been set forth. Unlike the traditional glued plywood panels, the cladding of the suggested products is made of planks, and the middle layer is a system of ribs made of sheet materials. This solution enables a considerable increase in the span overlapped by the panels and therefore allows for extended open spaces in wooden buildings. The panels can be either flat or curved in a circular arc on the short sides making it possible for them to be employed in vaulted cladding.

As a result of the geometric calculation, the basic size ratios of the panels were identified and a software program was developed in order to obtain the specified characteristics of the vaulted products. Computer models of the panels have been compiled and preliminary calculations performed proving the applicability of the resulting structures as elements of building cladding with spans up to 15 meters. Large spans can also be blocked provided that a construction lift is designed.

Panels of the specified design are also applicable in flooring of buildings, however, in this case the spans will be slightly smaller. This is due to the higher load on flooring. Other types of similar panels are being considered, including those employing thin-walled steel curved profiles as a middle layer. Applications for inventions have been submitted for the products described in the study.

Further research directions involve analytical identification of a rational ratio of the thickness of the boards of the panels and design of an engineering methodology to calculate them with no use of complex computing systems.

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Evaluation of the Load-Bearing Capacity of Cylindrical Stone Vaults Taking their Damage into Account

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Abstract

Introduction. The calculation of cylindrical stone arches of historical buildings is often performed in a core setting. At the same time, the ratio of bending moments M and longitudinal forces N, as well as cracks has a major effect on the load-bearing capacity of arches. The latter does not allow the analysis of the load-bearing capacity of arches by means of the standard methods. The aim of the study is to develop a methodology for assessing the load-bearing capacity of cylindrical stone arches with cracks.

Materials and Methods. The experimental studies confirm that one or even a few cracks are not invariably a sign of exhaustion of the load-bearing capacity of arches. The upper limit of the load-bearing capacity is due to such a number of conditional hinges (cracks), which converts the arch into a kinematic mechanism. It is possible to analyze the operation of vaults with cracks up to their physical destruction by means of the so–called interaction dependencies reflecting the limiting ratios MRd-NRd.

Research Results. The interactive dependencies of M_{Rd}–N_{Rd} were identified experimentally. The experiments also revealed the mechanisms of destruction of the cylindrical vault depending on the ratio M_{Rd}–N_{Rd}. Thus, under the action of only the bending moment, the destruction of the sample occurred along an unconnected section of the masonry; under the action of only the compressive force, as a result of the formation of longitudinal cracks; under the combined action of the compressive force and the bending moment, nature of the destruction depended on the ratio of these forces. The numerical model have been verified that can be used in order to design interactive dependencies.

Discussion and Conclusion. A methodology has been developed for assessing the load-bearing capacity of cylindrical stone arches with cracks using interactive dependencies reflecting the limiting ratios M_{Rd}–N_{Rd}. It is shown that it is possible to directly design interactive dependencies with numerical solid-state models, having previously "fine-tuned" them to the results of a number of simple masonry tests. The actual values of M and N in the cross sections are identified using the rod models of the arches. The load-bearing capacity of the arches is evaluated by comparing a certain combination of M-N with the interaction dependence curve.

Keywords: stone vaults, cylindrical vaults, cracks in stone vaults, destruction of vaults, load-bearing capacity, mechanisms of destruction of vaults.

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Оригинальное эмпирическое исследование

Оценка несущей способности цилиндрических каменных сводов с учетом их повреждений

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Аннотация

Введение. Расчет цилиндрических каменных сводов исторических зданий, зачастую, выполняют в стержневой постановке. При этом доминирующее значение на несущую способность сводов имеет соотношение изгибающих моментов M и продольных сил N, а также наличие трещин. Последнее не позволяет вести анализ несущей способности сводов стандартными методами. Целью исследований является отработка методики оценки несущей способности цилиндрических каменных сводов с трещинами.

Материалы и методы. Выполненные авторами экспериментальные исследования подтверждают, что появление одной и даже нескольких трещин не всегда является признаком исчерпания несущей способности сводов. Верхняя граница несущей способности определяется наличием такого количества условных шарниров (трещин), которое переводит свод в кинематический механизм. Вести анализ работы сводов с трещинами вплоть до их физического разрушения возможно с использованием так называемых интеракционных зависимостей, отражающих предельные соотношения M_{Rd}–N_{Rd}.

Результаты исследования. Интеракционные зависимости M_{Rd}–N_{Rd} определены авторами экспериментальным путем. Также в ходе проведенных экспериментов выявлены механизмы разрушения цилиндрического свода в зависимости от соотношения M_{Rd}–N_{Rd}. Так, при действии только изгибающего момента разрушение образца произошло по неперевязанному сечению каменной кладки; при действии только сжимающего усилия – в результате образования продольных трещин; при совместном действии сжимающей силы и изгибающего момента характер разрушения зависел от соотношения этих сил. Также выполнена верификация численной модели, с помощью которой возможно построение интеракционных зависимостей.

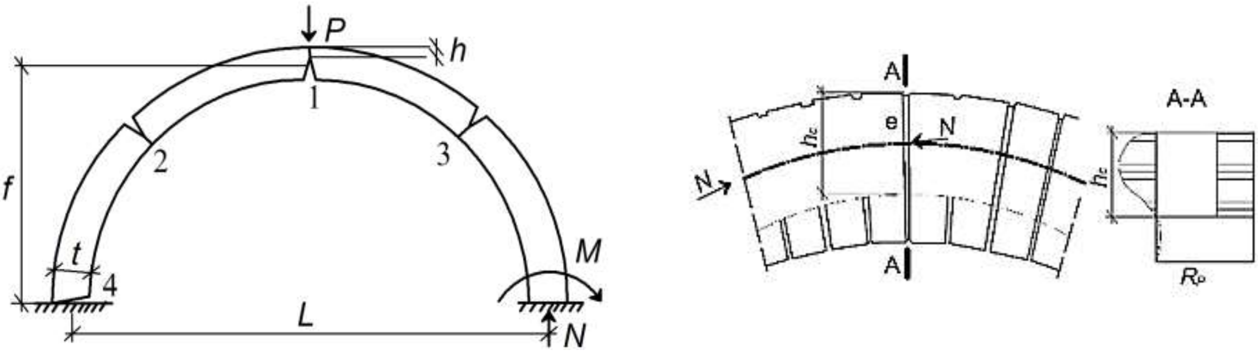
Обсуждение и заключение. Отработана методика оценки несущей способности цилиндрических каменных сводов с трещинами с использованием интеракционных зависимостей, отражающих предельные соотношения M_{Rd}–N_{Rd}. Показано, что непосредственно интеракционные зависимости возможно построить с помощью численных твердотельных моделей, предварительно «настроив» их на результаты ряда простейших испытаний кладки. Фактические значения M и N в сечениях определяются на стержневых моделях сводов. Оценка несущей способности сводов ведется путем сопоставления определенной комбинации M–N с кривой интеракционной зависимости.

Ключевые слова: каменные своды, цилиндрические своды, трещины в каменных сводах, разрушение сводов, несущая способность, механизмы разрушения сводов

Для цитирования. Зимин С.С., Орлович Р.Б., Данилов С.В., Москалькова Ю.Г. Оценка несущей способности цилиндрических каменных сводов с учетом их повреждений. *Современные тенденции в строительстве, градостроительстве и планировке территорий.* 2026;5(1):40–47. https://doi.org/10.23947/2949-1835-2026-5-1-40-47

Introduction. For more than a thousand years stone vaults have been one of the main structural elements of historical buildings. At the same time, analyzing their work is still relevant in assessing the technical condition, repairs, reconstruction and modernization of buildings. Cracks, degradation and layering of masonry, and even falls of its individual fragments, which might be due to uneven foundation precipitation, operational wear, overloads, dynamic impacts, man-made and other factors [1–6] are common defects in arches. Particularly unfavorable is the degradation of mortar joints typically made of weak lime mortars of old buildings. As a result of the violation of the adhesion between the mortar joints and the stones, the tensile and shear strength of the masonry along the unconnected sections declines.

The load-bearing capacity of arches depends not only on the mechanical characteristics of the masonry and accumulated damage but also on the geometry and nature of loading that determine the features of the stress-strain and mechanisms of destruction. In cylindrical arches, the ratio of bending moments M and longitudinal forces N is of primary importance. At the same time, the shape of the arch, which coincides with the pressure curve, is optimal. Deep cracks dividing the arch into a few large contiguous blocks correspond to the hinges of the design schemes and form both stable one-, three-hinged, and instantly changeable multi-hinged schemes[1]. The formation of hinges is caused by the pressure curve extending beyond the core of the section and development of cracks in the stretched zone due to the low adhesion of stones to the solution, as a result, compressive stresses dominate in the weakened sections (Fig. 1). In this case, the height of the stretched cross-sectional area with a height h increases in proportion to the decrease in the height of the compressed area h_c identified using the following expression: $h_c = 2 \cdot (0.5 \cdot h - M/N)$.



a) *b)*

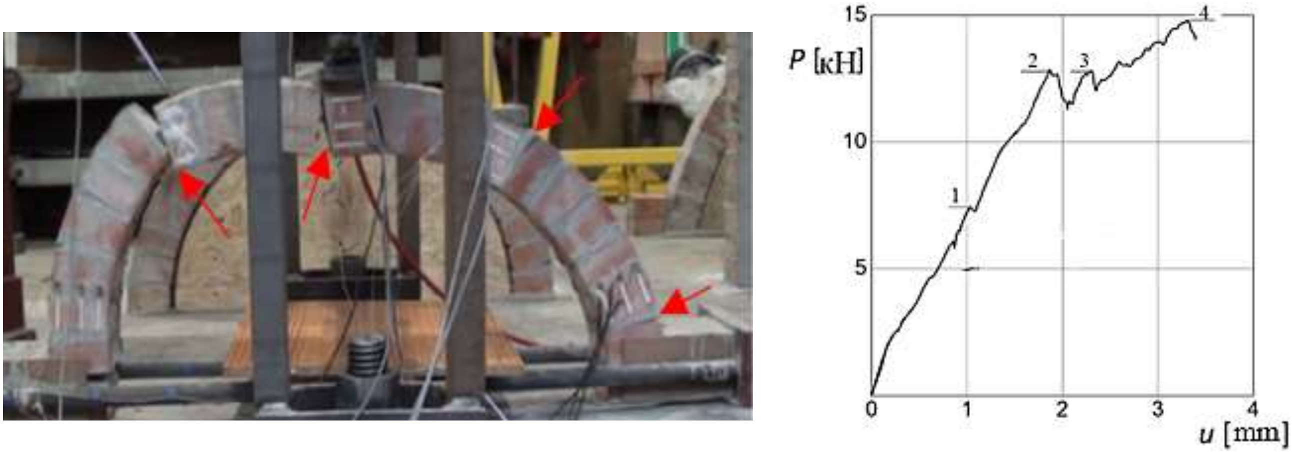
Fig. 1. Mechanism of destruction of cylindrical arches: *a* — scheme of formation of hinges in sections of arches with cracks; *b* — stress state of the section with a crack [10]

Materials and Methods. It is noteworthy that due to the variability of the eccentricity $e = M/N$ along the curve of the arch, location of possible hinges, particularly in case of damage in the form of cracks, is also variable. Therefore while assessing the load-bearing capacity of historical arches, it is critical to select the right design scheme taking not only the boundary conditions, but also the damage into account. The problem is that detailed localization of damages and, above all, cracks with identifying their geometric parameters is possible only from the inner surface of the vault. An inventory of damage from the outside of vaults is typically an issue and sometimes not possible, as this surface is commonly inaccessible for detailed inspection. The latter is due to the fact that inspection of vaults most frequently starts prior to their restoration when the inner surface is plastered and the outer is hidden under the existing floor. This serves to considerably reduce the reliability of surveys and initial data for verification calculations in compliance with the requirements of standards such as GOST 31937-2024 "Buildings and Structures. Rules of Inspection and Monitoring of Technical Condition".

It is noteworthy that cracks in the arches are not invariably a sign of exhaustion of their load-bearing capacity. This is confirmed by the experimental studies of the mechanism of destruction of stone arch samples with a height-to-span ratio of $f/L = 0.5$. The reference zones of the samples are rigid making their rotation unlikely. The tests were carried out with hydraulic jacks in the form of a concentrated load applied symmetrically in the middle of the sample span (Fig. 2). At all of the test stages, up to the destruction, deformation and cracking were recorded by means of a high-resolution high-speed camera. At the same time, vertical deformations of the u-samples were recorded by means of induction sensors. On top of that, relative deformations in the characteristic cross sections of the samples were recorded by means of strain gauges. The recordings of all of the measuring tools were read by means of a single HBM Hottinger MGCplus strain gauge bridge with a frequency of 10 Hz. The destruction of the samples was brittle and accompanied by cracks that were identified with the formation of hinges. Fig. 2*b* shows the experimental dependence of vertical deformations of the u-lock section of the arch on the load value P. At the same time, four characteristic load levels can be distinguished where due to the formation of hinges, the static scheme of the arch changed. At the first level, at a load of P = 6.7 kN, which is about 45% of the destructive load of P_p = 14.8 kN, the first hinge was formed in the middle lock part of the arch. The next two hinges 2 and 3 were formed almost simultaneously at test load levels of 84% and 90% of the destructive load, respectively. The final destruction occurred at a load of P_p = 14.8 kN as a result of the formation of the fourth hinge in the right support heel of the arch after which it turned into an instantaneously changeable system (Fig. 2*a*).

According to the experimental data, presence of a crack in cylindrical arches is not a sign of exhaustion of their load-bearing capacity. This is to be taken into account while examining the arches and calculating their actual load-bearing capacity in compliance with the requirements of the standards. The problem can be solved using the numerical method making it possible to simulate crack development in the most stressed sections of arches (Fig. 3).

[1] Study of Deformations, Calculation of the Load-Bearing Capacity and Structural Reinforcement of Ancient Spacer Systems: Methodological Recommendations. Moscow: Rosrestavratsiya; 1989. 164 p. URL: https://www.restsouz.ru/upload/guidelines/003-BessonovGB-Issledovanie-deformacij-raschyot-nesushcej-sposobnosti-i-konstruktivnoe-ukreplenie-drevnih-raspornyh-sistem.pdf (accessed: 03.02.2026).



a) *b)*

Fig. 2. Mechanism of destruction: *a* — stone arch; *b* — experimental relationship between the magnitude of the load *P* and the deflection of the u-lock section [10]

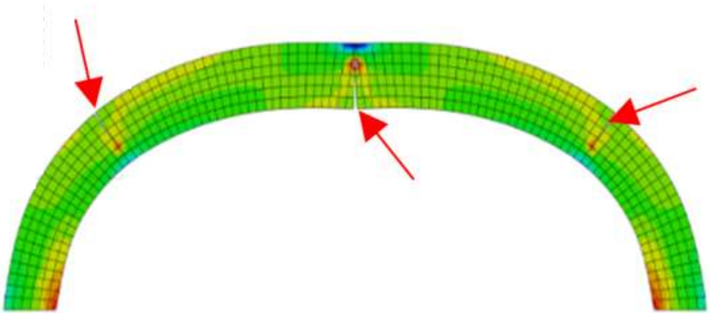


Fig. 3. Stress-strain of the arch at the stage of crack formation (indicated by arrows) during finite element modeling [10]

However, in this case, there are some difficulties taking the geometric and physical nonlinearity into account associated with crack formation and transition of masonry material in the compressed zone of sections with cracks to a plastic state. Hence the kinematic method might be preferred making it possible to evaluate the upper limit of the load-bearing capacity of arches [7-9]. To this end, interactive dependencies are employed between the bending moments M_{Rd} and the compressive forces N_{Rd} where the section transitions to the limiting state (Fig. 4).

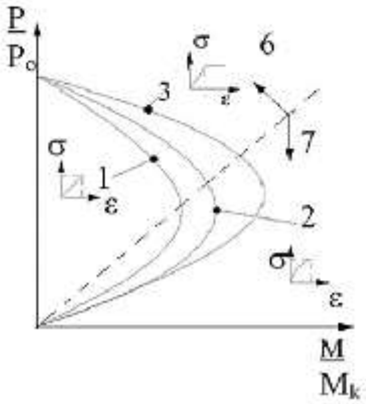


Fig. 4. Interaction curves between the compressive force N_{Rd} and the bending moment M_{Rd} in compressed-bent stone structures with linear (1), parabolic (2) and ideally plastic (3) dependencies σ–ε of masonry [9]

In relation to the evaluation of the load-bearing capacity of arches, the method involves implementing the following procedures. At the first stage, the normal forces *N* and bending moments *M* in each of the arch sections are identified using analytical methods for calculating curved rods. At the same time, the cracks identified during the surveys of the arches are modeled in the form of hinges. At the next stage, the obtained values of *M* and *N* are compared with the interaction dependences of the limiting values of bending moments M_{Rd} and longitudinal forces N_{Rd} exceeding which means formation of a hinge that is introduced into the design scheme. The process continues until the the last hinge where the arch system transforms into a mechanism (Fig. 1*a*).

Research Results. The M_{Rd}–N_{Rd} interaction dependencies can be identified using experimental or numerical methods as exemplified by the study. Experimental studies were conducted on 5 samples of masonry measuring 12×25×105 cm made of solid brick (brick size 12×25×6.5 cm) of the M250 brand on cement mortar of the M50 brand. The samples were loaded using two independent hydraulic jacks creating axial compression N and a transverse load P leading to a bending moment (Fig. 5).

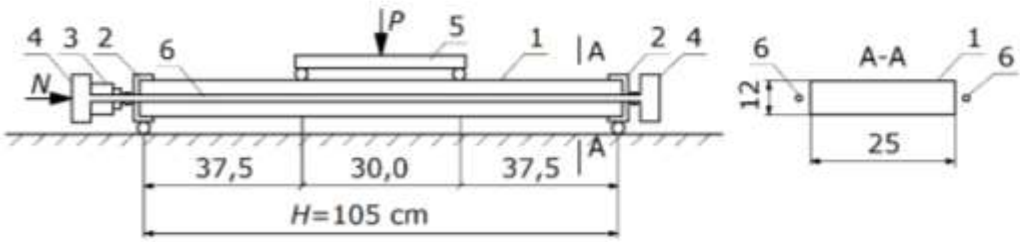
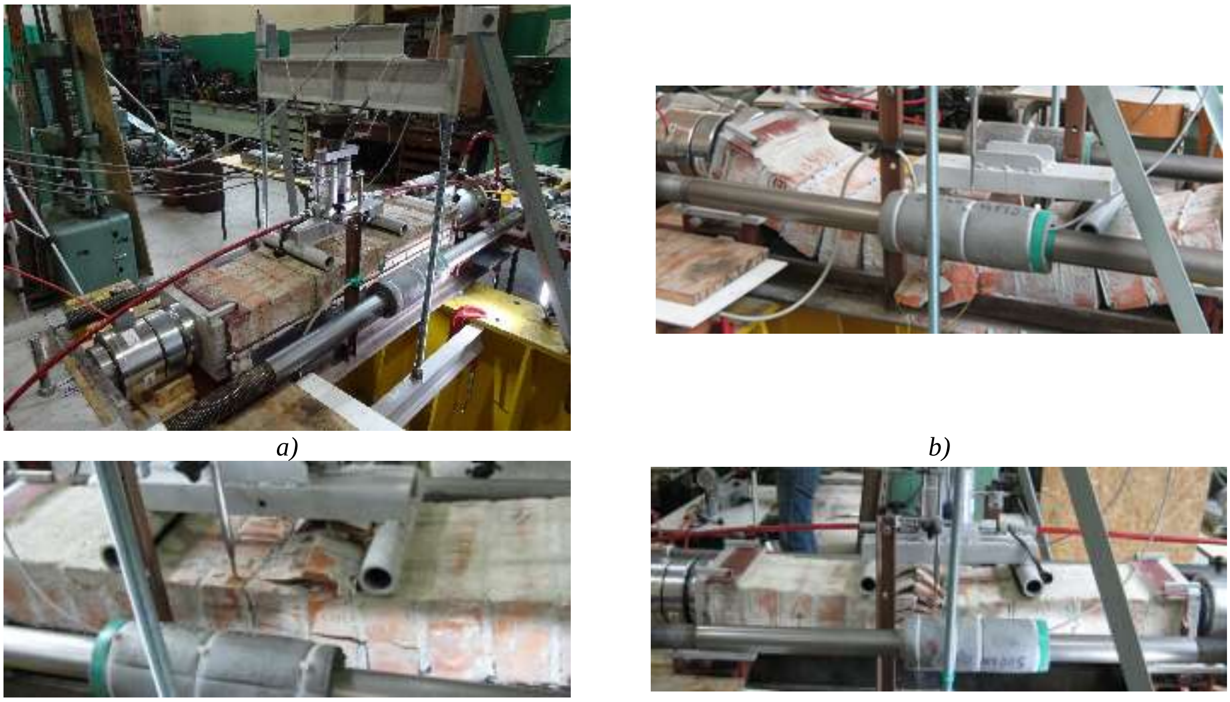


Fig. 5. Test scheme of the experimental stone samples for compressive bending: 1 — test sample; 2 — steel heads with hinges; 3 — hydraulic jack creating force N; 4 — traverses for fastening steel tightens and hydraulic jack; 5 — distribution beam; 6 — steel tightens with a force meter [10]

The first sample was tested for central compression with axial force N, the second one was tested for bending, and the remainder of the samples were tested for compression with bending, and during their loading the force level N remained constant, and the force P was variable until destruction. It was found that under the action of the bending moment alone, the fracture of the sample occurred along an unconnected section of the masonry at a value of M_{Rd} = 1.0 kN·m (Fig. 6*b*). Under the action of the compressive force alone, the fracture occurred as a result of the formation of longitudinal cracks at a value of N_{Rd} = 690 kN (Fig. 6*c*). Under the combined action of the compressive force and the bending moment, the nature of the fracture depended on the ratio of these forces and was most frequently accompanied by fragmentation of the most compressed zone of the samples (Fig. 6*d*).



a) *b)* *c)* *d)*

Fig. 6. Testing of a brickwork sample: *a* — type of test bench; *b* — the nature of the destruction of samples during bending; *c* — the nature of the destruction of samples during compression; *d* — the nature of the destruction of samples during compression with bending [10]

The numerical analysis was conducted using a complex of high-level finite element modeling. At the same time, the design scheme of the model (Fig. 7) corresponded to that of laboratory tests of the samples (Fig. 5).

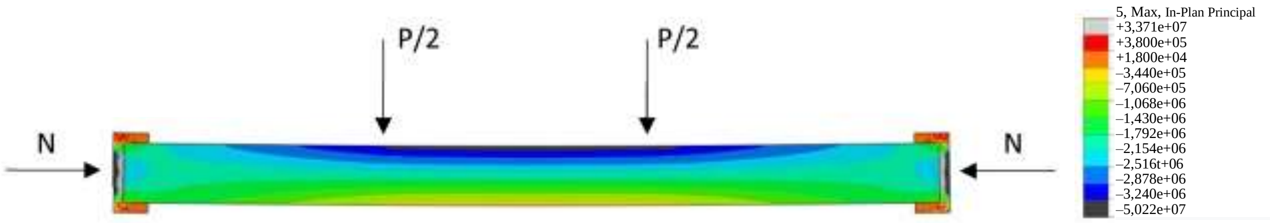
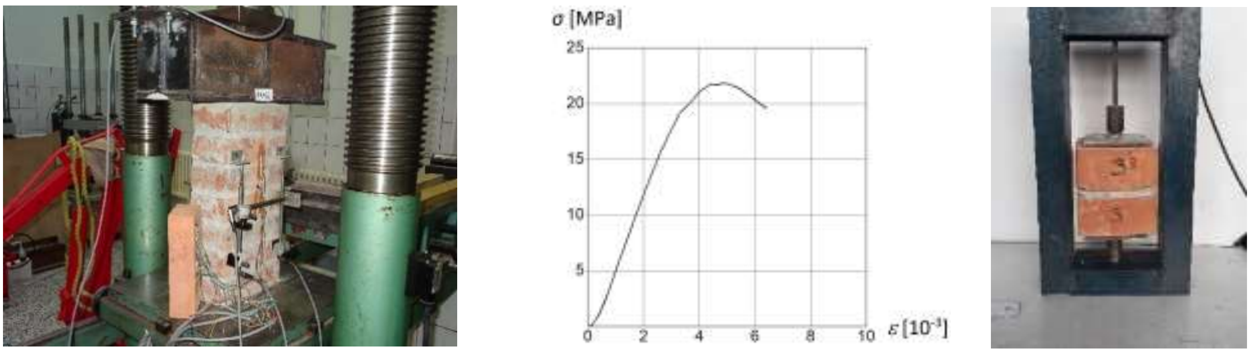


Fig. 7. Calculated stress state of the sample under the simultaneous action of a compressive force N = 120 kN and a bending moment M = 1.24 kNm [10]

The calculation model provided for the possibility of crack formation in the zone of the greatest tensile stresses and their development along the section height. On top of that, the development of plastic deformations of the masonry in the compressed section zone above the crack was taken into account. As the problem was being solved, for each loading level of N = 10 kN and P = 0.05 kN, the modulus of deformation of the material and height of crack development were adjusted. The masonry deformation modulus was assigned in compliance with the experimental dependence σ–ε obtained as a result of compression tests of the standard samples up to fracture (Fig. 8 *a,b*). The tensile strength perpendicular to the mortar joints of the masonry was identified by means of tensile tests of the samples of two bricks connected with a mortar joint (Fig. 8 *c*).



a) *b)* *c)*

Fig. 8. Testing of the masonry samples: *a*, *b* — for compression; *c* — for stretching across the mortar joints [10]

The interaction relationship between the experimental values of M_{Rd} and N_{Rd} is shown in Fig. 9 as an approximating curve 1. In the same figure, an approximation of the results of the numerical studies is shown as curve 2.

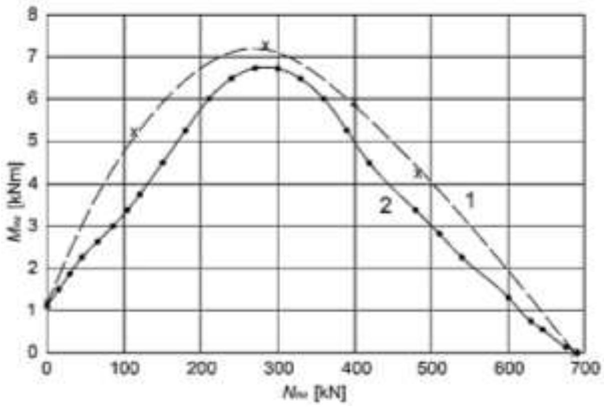


Fig. 9. Interactive M_{Rd}–N_{Rd} dependencies of the masonry samples during compression with bending: 1 — experimental ones; 2 — theoretical ones [10]

According to the above studies, the values of the load-bearing capacity of the experimental samples turned out to be 10–25% higher than those of numerical modeling. This is due to the omission of the volumetric stress state of the compressed section zone with a crack during its plastic deformation in the computational model.

Based on the analysis of the M_{Rd}–N_{Rd} dependence, an increase in the compressive force to the level of $N < 0{,}4N_{Rd}$ neutralizes the negative effect of the bending moment, which results in cracks along the loose section of the masonry.

At values of $N < 0{,}4N_{Rd}$, compressive stresses dominate with the concentration in the most compressed zone of the section leading to fragmentation of the masonry in the area.

Discussion and Conclusion. The set of studies performed has enabled us to conclude that the interactive method can be employed in order to assess the load-bearing capacity of vaulted structures when calculated in a rod arrangement and in the presence of cracks. The interaction dependence itself can be obtained by means of a numerical solid-state model (a rectilinear prism with the possibility of combining the values of *M* and *N*), which is confirmed by the results of its experimental verification.

The algorithm for evaluating the load-bearing capacity of the vault is the following. A number of relatively simple masonry tests are pre-performed, according to the results of which a numerical solid-state model is "fine-tuned". Further on the values of the longitudinal and transverse forces vary in the model and, based on the test results, the M_{Rd}–N_{Rd} interaction dependence is designed. A model of the arch in the core setting is designed separately. A certain calculation situation yields combinations of *M*–*N* values along the length of the arch of the vault. The load-bearing capacity of the vault up to its physical destruction is evaluated by means of comparing a certain combination of *M*–*N* with the M_{Rd}–N_{Rd} interaction dependence curve.

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Original Empirical Research

Ensuring the Strength, Rigidity and Stability of Load-Bearing Structures of a Multi-Story Building of Complex Shape under Seismic Impacts

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Abstract

Introduction. In order to ensure earthquake resistance and to reduce seismic loads, a spatial calculation of the load-bearing structures of a multi-storey building of complex shape was performed. This article analyzes the design system, computational and dynamic model taking the main and special combinations of loads into account.

Materials and Methods. The calculations were performed by means of the analytical method and the finite element method (FEM) in the STARK ES software package.

Research Results. The results of dynamic calculation are obtained for basic and special combinations of loads and corresponding combinations of internal forces in the calculated structures of a multi-storey building of complex shape. A total of 53 loadings were used.

Discussion and Conclusion. The results of the calculation of a multi-storey building of complex shape have shown that the required strength, rigidity and stability of load-bearing structures are ensured in the design situation in question.

Keywords: building, seismic impact, calculation, load, structural system, seismic resistance, calculation dynamic model, strength, rigidity, stability

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Оригинальное эмпирическое исследование

Обеспечение прочности, жёсткости и устойчивости несущих конструкций многоэтажного здания сложной формы при сейсмических воздействиях

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Аннотация

Введение. С целью обеспечения сейсмостойкости и снижения сейсмических нагрузок выполнен пространственный расчет несущих конструкций многоэтажного здания сложной формы. В настоящей статье анализируются конструктивная система и расчетно-динамическая модель с учетом основных и особых сочетаний нагрузок.

Материалы и методы. Расчеты проводились аналитическим методом и методом конечных элементов (МКЭ) в программном комплексе STARK ES.
Результаты исследования. Получены результаты динамического расчета при основных и особых сочетаниях нагрузок и соответствующих сочетаниях внутренних усилий в рассчитываемых конструкциях многоэтажного здания сложной формы. Всего было использовано 53 загружения.
Обсуждение и заключение. Результаты расчета многоэтажного здания сложной формы показали, что обеспечивается требуемая прочность, жёсткость и устойчивость несущих конструкций при рассматриваемой расчетной ситуации.

Ключевые слова: здание, сейсмическое воздействие, расчет, нагрузка, конструктивная система, сейсмостойкость, расчетно-динамическая модель, прочность, жёсткость, устойчивость
Для цитирования. Мажиев Х.Н., Мажиев К.Х., Панасенко Ю.В., Мажиева А.Х., Мажиев А.Х., Мажиев А.Х. Обеспечение прочности, жёсткости и устойчивости несущих конструкций многоэтажного здания сложной формы при сейсмических воздействиях. *Современные тенденции в строительстве, градостроительстве и планировке территорий*. 2026;5(1):48–67. https://doi.org/10.23947/2949-1835-2026-5-1-48-67

Introduction. The design of high-rise buildings involves complexities of assessing the stress-strain of the main load-bearing structures during seismic impacts.

In order to analyze the reliability, strength, rigidity and stability of load-bearing building structures under basic and special combinations of design loads, a spatial calculation of a multi-storey building of complex shape was performed. The building is under construction on a site where intense seismic impacts are likely [1–6, 15].

A scientific and technical assessment of the compliance of design solutions to ensure the safety of buildings with the requirements of regulatory documents of the Republic of Azerbaijan where the building is under construction is provided.

Materials and Methods. The structural system is a 29-storey complex-shaped building with a height of 150.70 m with a 4-storey structure in the stylobate part and two underground levels. The multi-storey building is located on the drained lands of the Caspian Sea on a 160×80 m construction site and is connected to the coastline via a bridge and a tunnel.

The building is designed according to the scheme of a frame-link frame. The main supporting structures (columns and beams of floors, cores of rigidity, slabs and basement structures) are designed from monolithic reinforced concrete; steel columns and beams — for the top of the building.

The structural stability system is made up of rigidity cores extending from the foundation plate to the top of the building. There are eight elevators in the rigidity cores. The cores also provide access to the stairs. The cores are made up of walls that transfer vertical and horizontal forces to the foundation [1–6, 15]. Apart from the walls, the floors are supported by vertical and inclined columns. The main cores have a plan size of 17.40×14.40 m and form a rectangular shape. The thickness of the outer walls is 800 mm, the inner walls are 300 mm.

The foundation of the building is a solid monolithic slab resting on a pile base. The thickness of the slab is 2650 mm, 2900 mm under the towers and 750 mm, 1000 mm and 2350 mm for the rest of the building. Piles with a diameter of 1.5 m under the foundation of the towers and 2.0 m under the foundation of the stylobate/basement.

The roof of the stylobate is made up of a steel structure and its cladding - a glass facade.

Two reinforced concrete cores of rigidity on each side of the building at the 27^{th} floor level are connected by a steel frame structure — a 90 m long mega-truss forming the shape of a crescent. At the junction, the height of the mega-farm is 7.6 m, in the center — 20.45 m. The roof above the upper level has a rounded shape. The connecting beams are mostly steel profiles encased in concrete. The connection of the truss to the reinforced concrete cores is provided by steel plates. These reinforced concrete cores of rigidity are the main elements ensuring the stability, strength and rigidity of the building. The truss elements were designed to handle vertical and horizontal loads. It should be noted that the main steel beams of the mega-farm will be subjected to significant axial forces due to the general deformation of the building and due to horizontal and vertical loads acting on it [7–12].

It is assumed that some of the mega-truss will be assembled on the stylobate floor slab and then installed in the design position. The lifting scheme assumes that parts of the mega-truss and the walls of reinforced concrete cores of rigidity will be built first resulting in a rigidly connected frame. This cantilever frame will support a lifting platform and hydraulic jacks for mounting the middle section of the truss.

The suggested construction sequence is divided into the following stages:

– Construction of the underground part and the stylobate. The first stage entails the construction of the base structures. The walls of the rigidity cores are then mounted followed by the basement columns. After the stylobate and the basement roof have been constructed, it is planned to install the main structures of the steel roof.

– Construction of the towers. The main walls, columns and slabs are up to the level of the 27th floor.

– Installation and lifting of the truss. It is assumed that some of the steel truss will be assembled at ground level and then installed in the design position.

– The upper part of the building. After the completion of the reinforced concrete cores of rigidity and the elements around the mega-farm will be raised to the design position and connected to the cores of rigidity at the level of the 27th floor. After that, jacks will be installed between the levels of floors 21-26. Small steel parts on the 18th, 19th and 20th floors will be constructed using a tower crane. The other floors of the farm (floors 28 and 29) can then be constructed using a tower crane.

In order to assess the reliability of the design solutions employed in the project, the spatial model of the building was calculated for the design loads and impacts. The calculation was carried out by means of the STARK ES software package [12–16].

A spatial shell-rod model was employed as a computational model where the supporting columns, crossbars and truss elements are represented by rod elements of a general type, the coating shell, floor slabs and walls are represented by elements of a flat shell (Fig. 1).

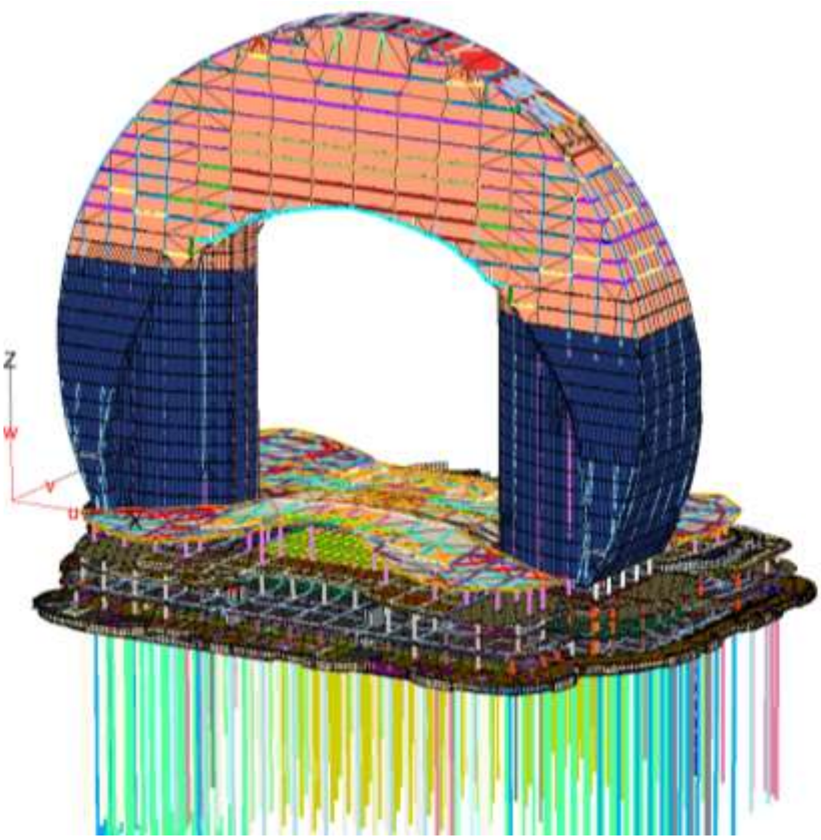

Fig. 1. A finite element model of a multi-storey building of complex shape [15]

The level of responsibility of the building in compliance with GOST 27751-88 "Reliability of Building Structures and Foundations. Main Provisions of the Calculation" is high. The value of the reliability coefficient of responsibility in compliance with GOST 27751-88:

- while calculating forces and stresses (according to the first group of limiting conditions) in building structures, bases and foundations of a building is assumed to be equal to γ_n = 1.1;

-while identifying displacements (according to the second group of limit states) of structures and base deformations, it is assumed to be equal to γ_n = 1.0;

- while calculating for emergency impacts ("progressive" destruction) and seismic impacts — equal to γ_n = 1.0.

Evenly distributed temporary loads on the floor slabs are accepted in compliance with the purpose of the premises. Load reliability coefficients γ_f and load combination coefficients Ψ_i are adopted in compliance with the requirements of AzDTN 2.1-1.

While calculating the building, the following loads and impacts are considered:

- vertical constant loads from the own weight of the load-bearing and enclosing structures of the building;
- long-term loads from engineering equipment;
- temperature loads;
- snow load;
- average and pulsation components of the wind load (terrain type A);
- seismic impact for an 8-point area.

Snow area — I. The estimated weight of the snow cover per 1 m^2 of horizontal surface is 0.8 kPa. In the calculation model the snow load is applied uniformly over the entire surface of the building cover. Snow bags are also considered.

The wind area — VI. Type of terrain A. Standard value of wind pressure ω_0 = 0.73 kN/m^2.

The seismicity of the construction site is assumed to be 8 points (repeatability of 1000 years). The soil class according to seismic properties is III. Fig. 2–3 show the dynamicity coefficient and values of the parameters for calculating the seismic impact.

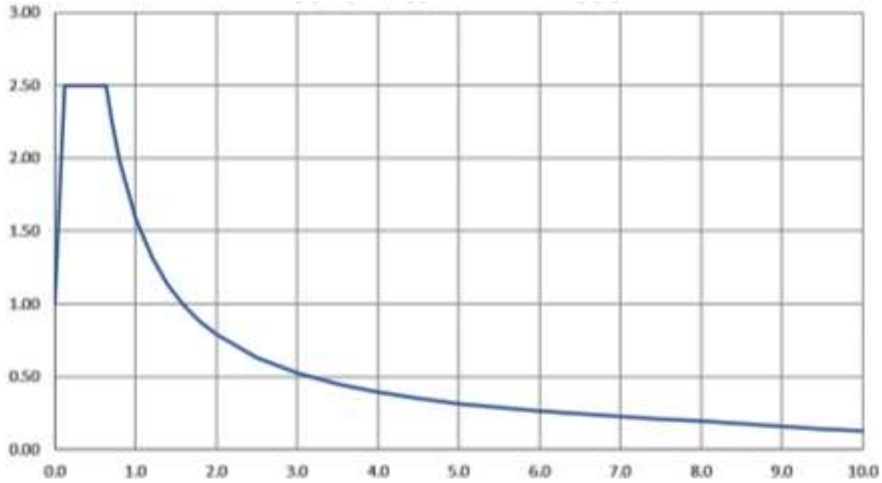

Fig. 2. Dynamic coefficient β(T)

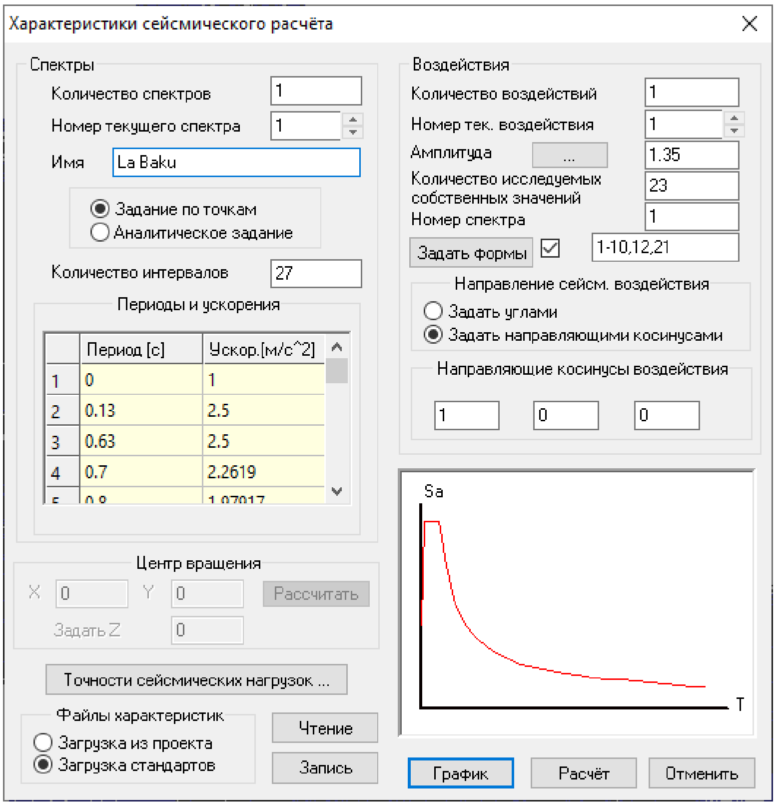

Fig. 3. Characteristics of the seismic calculation

The loads on the design scheme were assumed in compliance with AzDTN 2.1-1 and according to the initial data. A total of 53 loads were used in the calculation model of a multi-storey building of complex shape to account for all of the loads. Diagrams of the application of some of the loads are shown in Fig. 4–7.

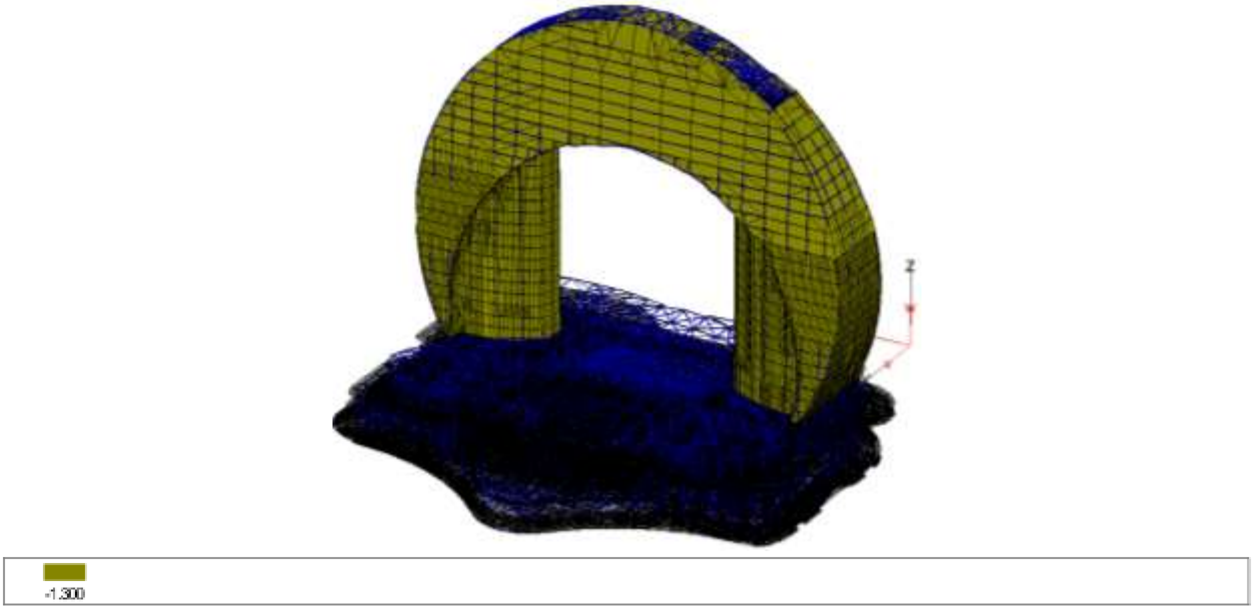

Fig. 4. Scheme of applying the load from the weight of facade structures, kN/m^2 [15]

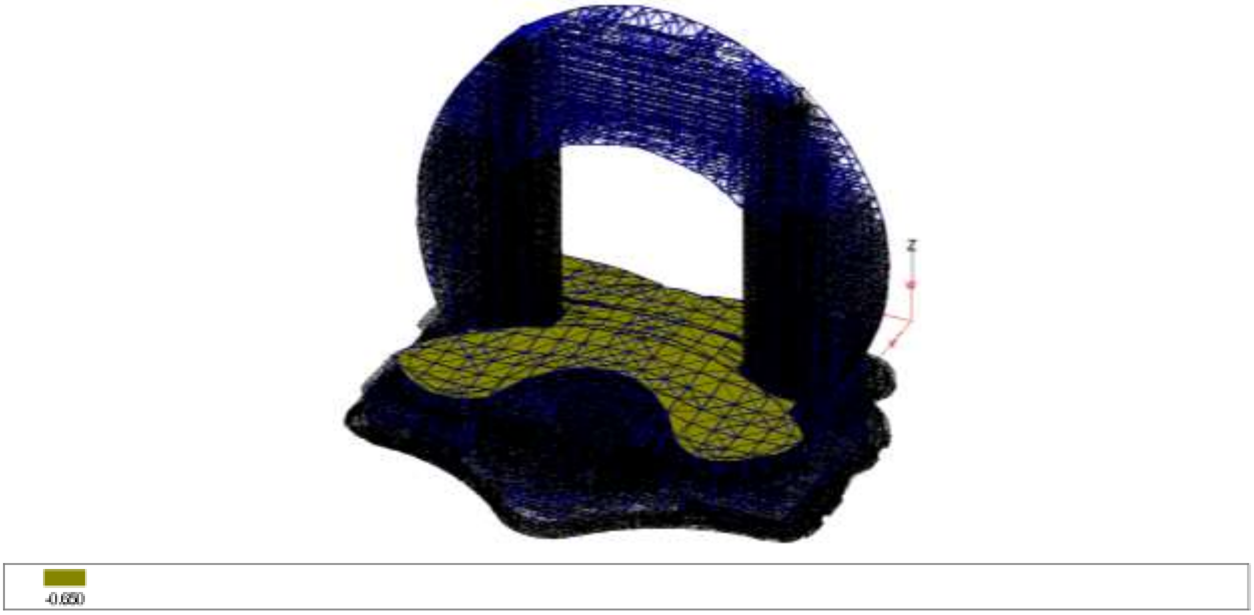

Fig. 5. Scheme of application of the load from the weight of the roof structures of the stylobate, kN/m^2 [15]

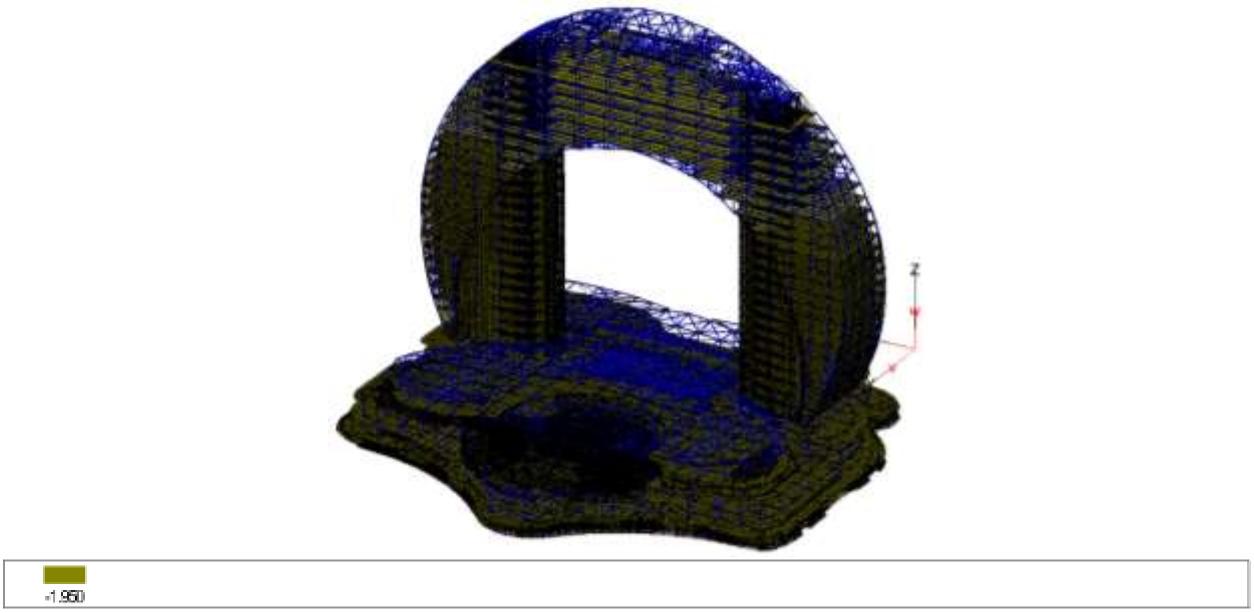

Fig. 6. Scheme of applying the load from the weight of the partitions, kN/m^2 [15]

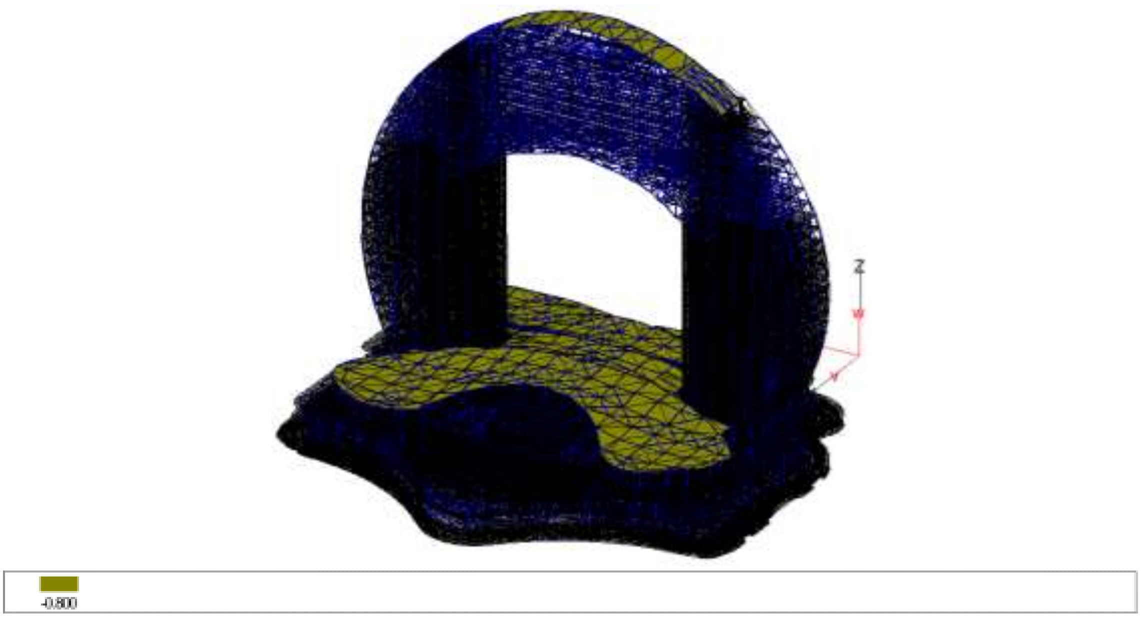

Fig. 7. Snow load application scheme, kN/m^2 [15]

Research Results. The foundation draft was evaluated for the main combinations of the full values of the standard loads. The pattern of the foundation sediment is shown in Fig. 8. The maximum sediment was 50.2 mm.

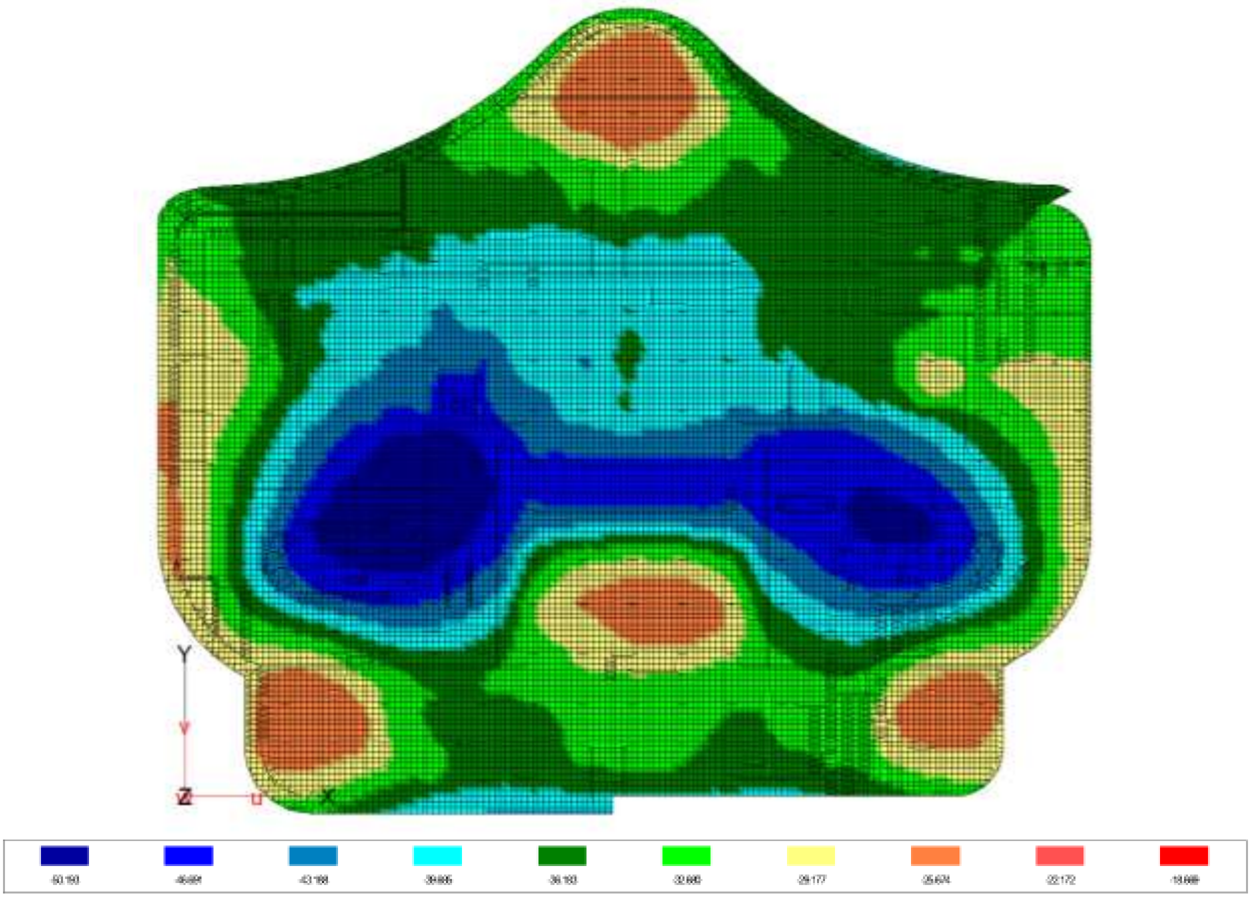

Fig. 8. Pattern of the foundation sediment, *Uz* [15]

The maximum value of the relative difference in the drafts from the main load combinations is s/L = 0.00016, which is not beyond the limit value of 0.005.

The horizontal deviations of the top of the building were evaluated for the main combinations of the total values of the standard loads (Fig. 9).

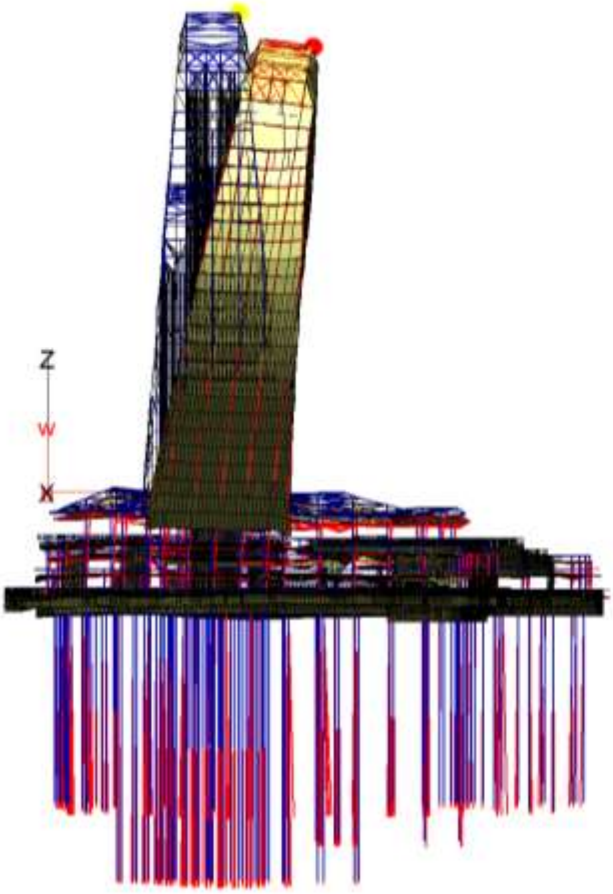

Fig. 9. Horizontal displacement of the top of the building [15]

The maximum horizontal displacement of the top of the building from the standard load values is 163.4 mm, which is not beyond the maximum permissible value of 300 mm.

The results of the calculation of the first form of stability loss are shown in Fig. 10.

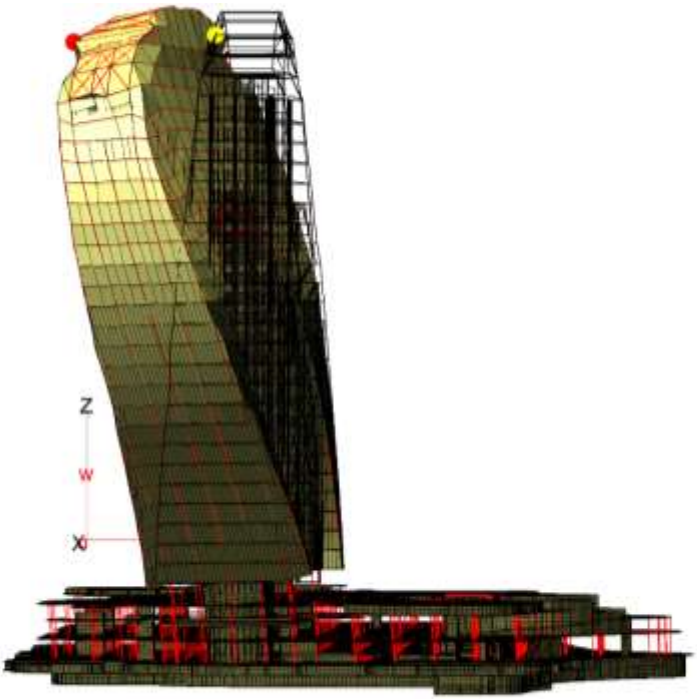

Fig. 10. Form 1 of stability loss [15]

The lowest critical load parameter (stability margin) considering the reliability coefficient of responsibility is 10.4. The overall stability of the load-bearing structures of the building is ensured.

Fig. 11–13 show the first forms of natural vibrations of a multi-storey building of complex shape.

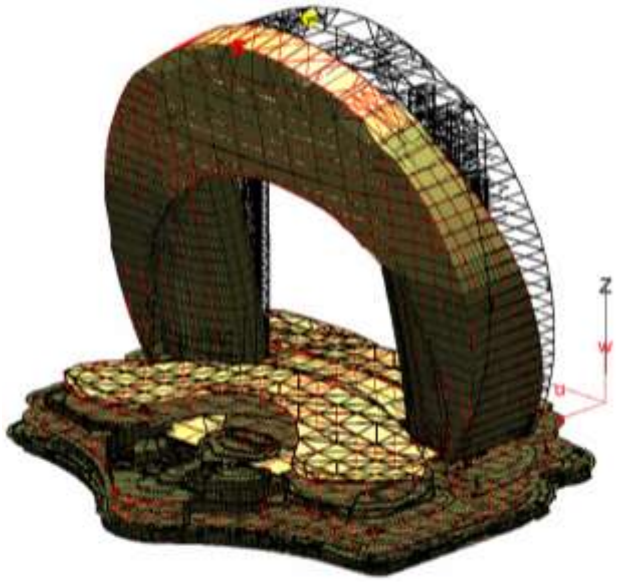

Fig. 11. First form of own oscillations, f_1 = 0.340 Hz [15]

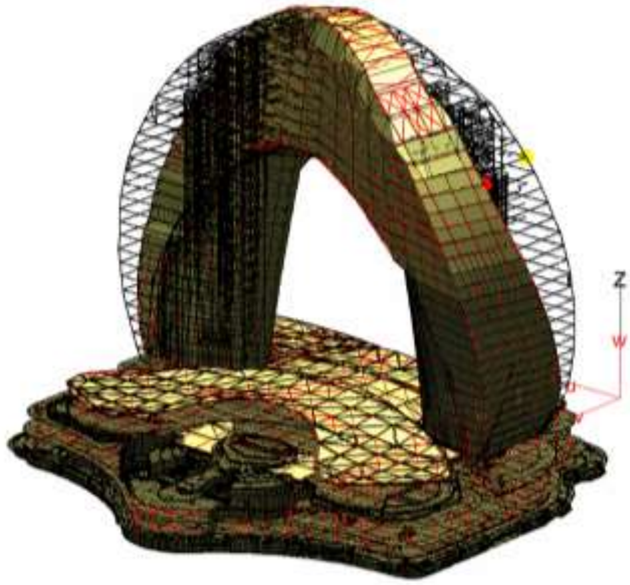

Fig. 12. Second form of own oscillations, f_2 = 0.459 Hz [15]

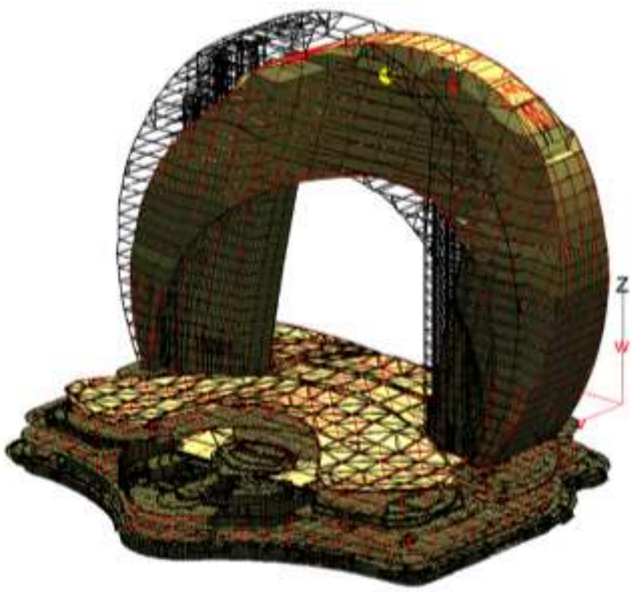

Fig. 13. Third form of own oscillations, f_3 = 0.484 Hz [15]

The vertical deflections of the floor slabs of the western and eastern towers of a multi-storey building of complex shape at different elevations are shown in Fig. 14–17.

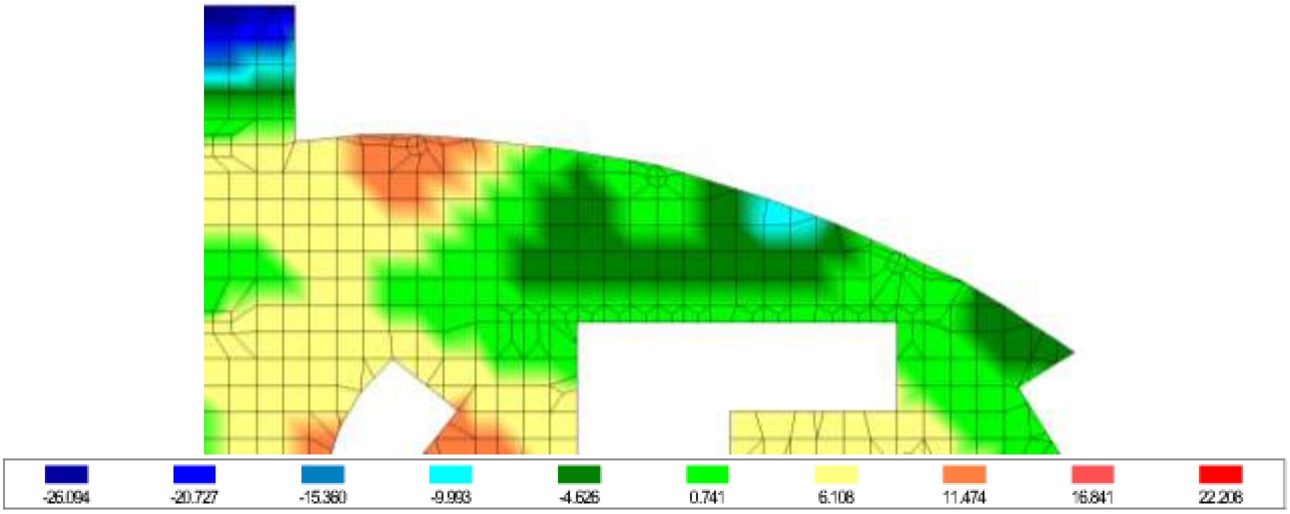

Fig. 14. Maximum relative vertical displacements of the western tower at –6.7 m [15]

The maximum vertical deflection of the floor slab from constant and prolonged loads was 11.47 mm with a console reach of 4.9 m, which is nit beyond the limit value of l/200 = 49 mm.

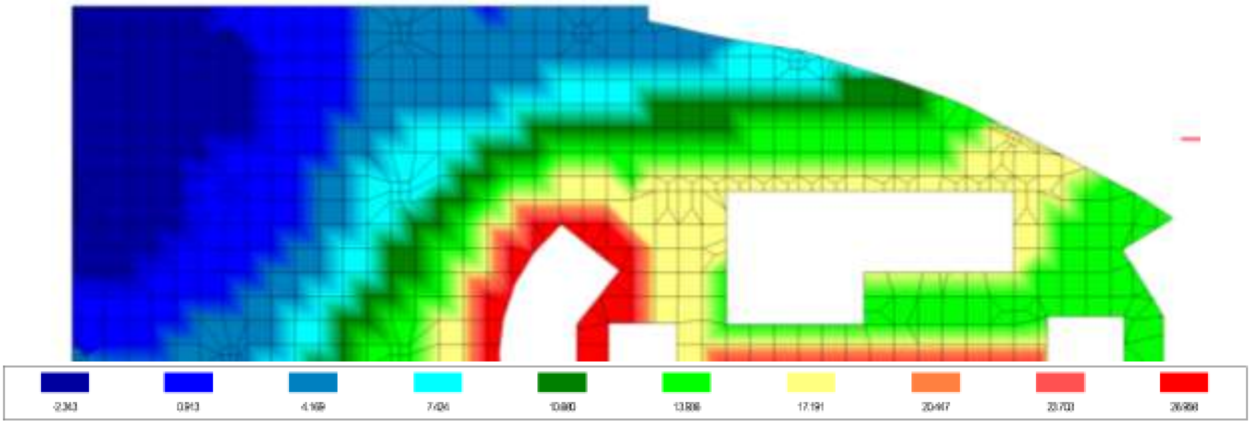

Fig. 15. Maximum relative vertical movements of the western tower at +42.8 m [15]

The maximum vertical deflection of the floor slab from constant and prolonged loads was 1.30 mm with a span of 6.6 m, which is not beyond the limit value of l/200 = 33 mm.

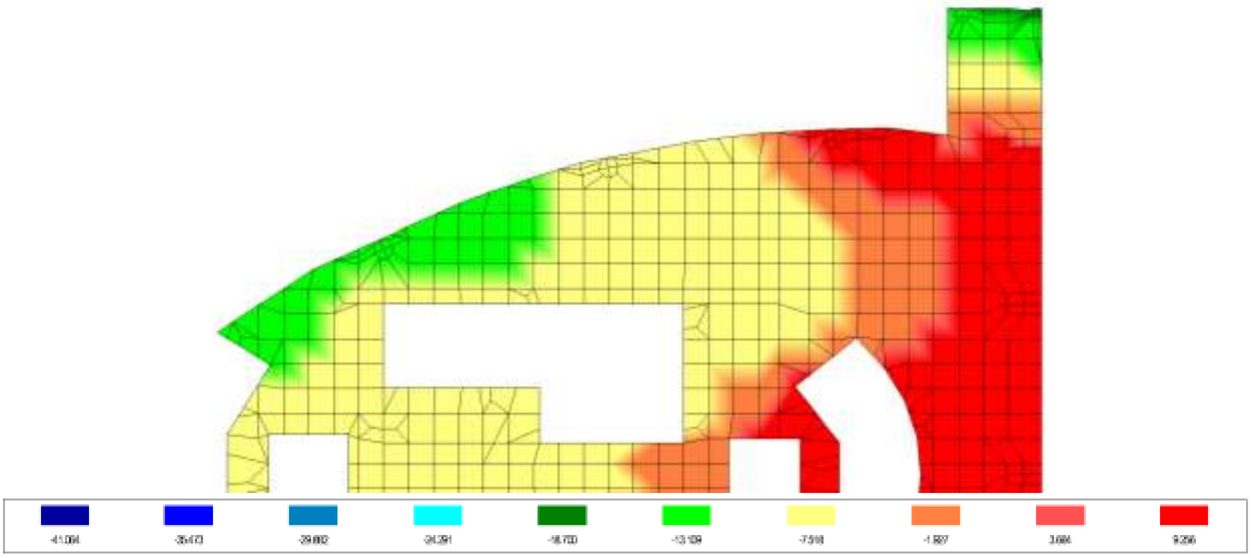

Fig. 16. Maximum relative vertical displacements of the eastern tower at –6,7 m [15]

The maximum vertical deflection of the floor slab from constant and prolonged loads was 13.8 mm with a console reach of 4.9 m, which is not beyond the limit value of *l*/200 = 49 mm.

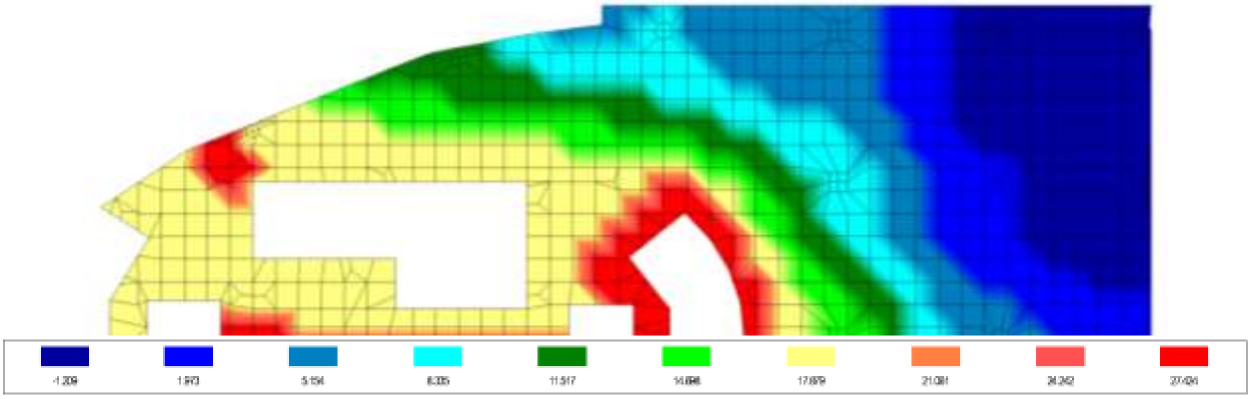

Fig. 17. Maximum relative vertical displacements of the eastern tower at +42.8 m [15]

The maximum vertical deflection of the floor slab from constant and prolonged loads was 1.21 mm with a span of 6.6 m, which is not beyond the limit value of *l*/200 = 33 mm.

The calculation of the amount of reinforcement was performed under basic and special load combinations, including seismic effects considering the conditions of strength and crack resistance. The material characteristics of reinforced concrete structures are shown in Table 1.

Table 1

Characteristics of structural materials

Structure	Concrete type	Longitudinal reinforcement class	Transverse reinforcement class	Thickness of the protective layer			
				SO, mm	SU, mm	RO, mm	RU, mm
Foundation, 750 mm	B45	A500C	A500C	60	90	160	200
Floor slabs	B45	A500C	A500C	35	45	35	45
Walls	B70	A500C	A500C	35	45	35	45
Columns	B70	A500C	A500C	50	50	50	50

Fig. 18-29 shows the results of selecting the necessary reinforcement for a special combination of loads for some structures - parts of the foundation slab of a multi-storey building of complex shape with a thickness of 750 mm, floor slabs of the western and eastern towers at +42.80 m.

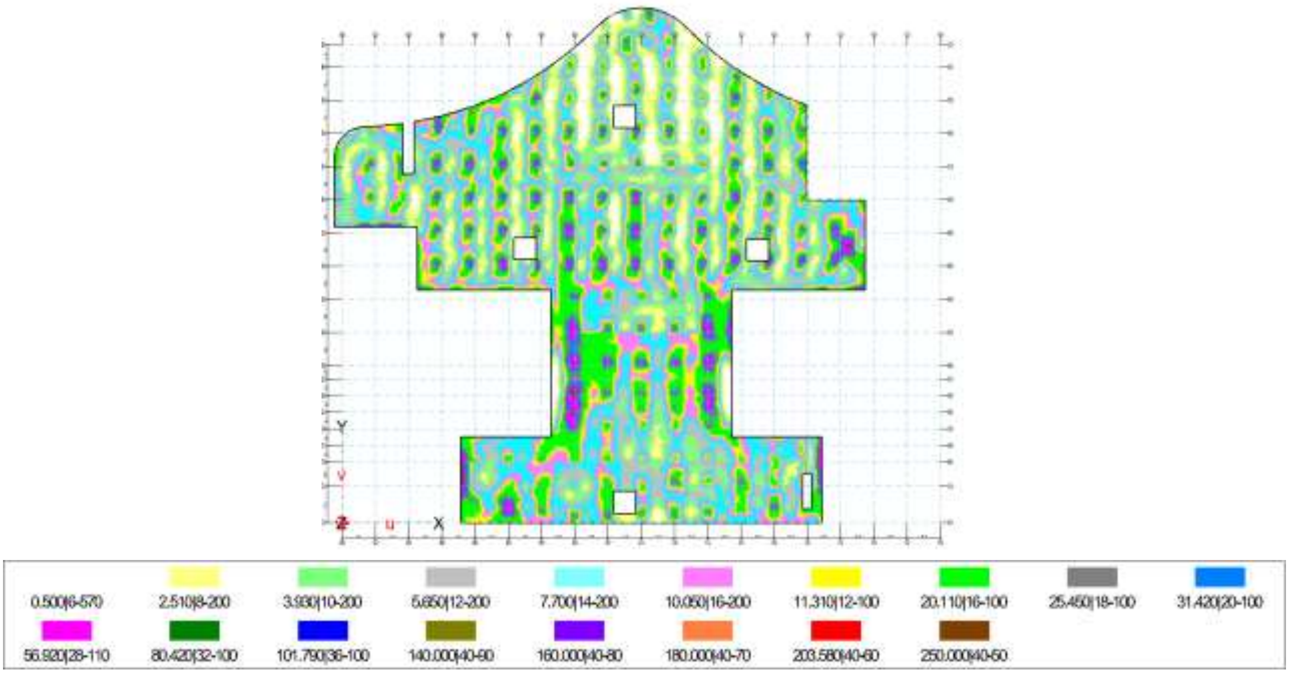

Fig. 18. Reinforcement of the foundation plate. Upper longitudinal reinforcement in the direction of the axis *R*, cm^2/m [15]

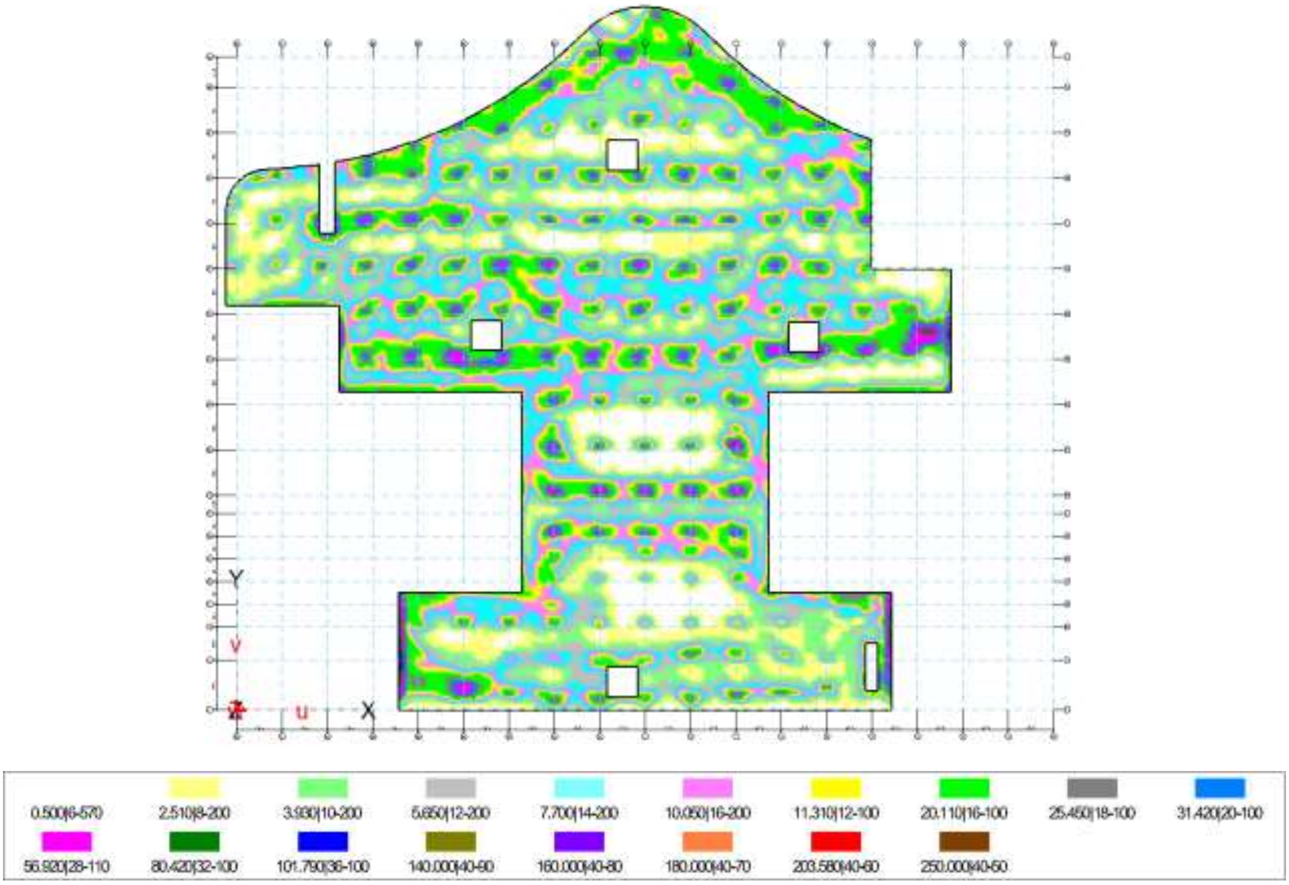

Fig. 19. Reinforcement of the foundation plate. Upper longitudinal reinforcement in the direction of the axis *S*, cm^2/m [15]

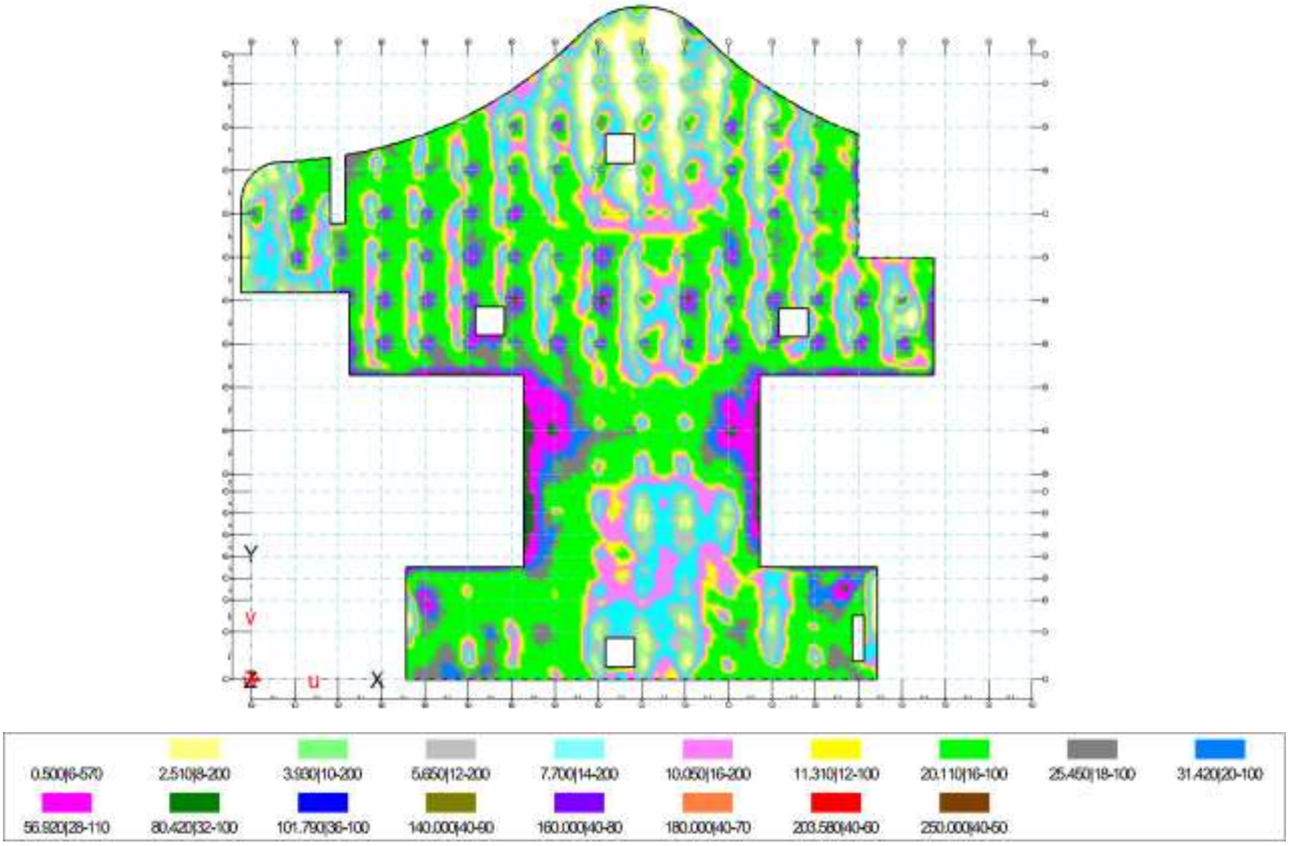

Fig. 20. Reinforcement of the foundation plate. Lower longitudinal reinforcement in the direction of the axis *R*, cm^2/m [15]

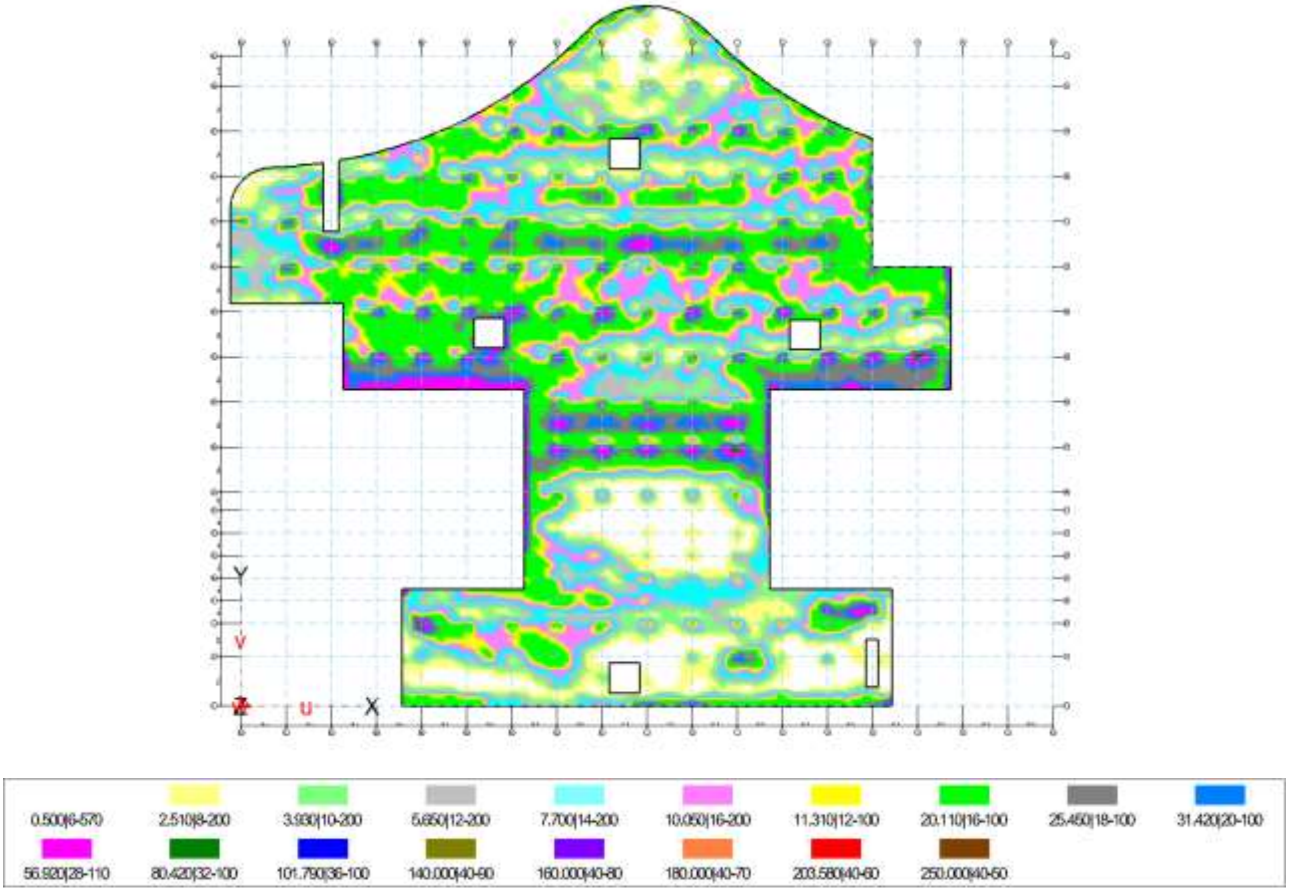

Fig. 21. Reinforcement of the foundation plate. Lower longitudinal reinforcement in the direction of the axis *S*, cm^2/m [15]

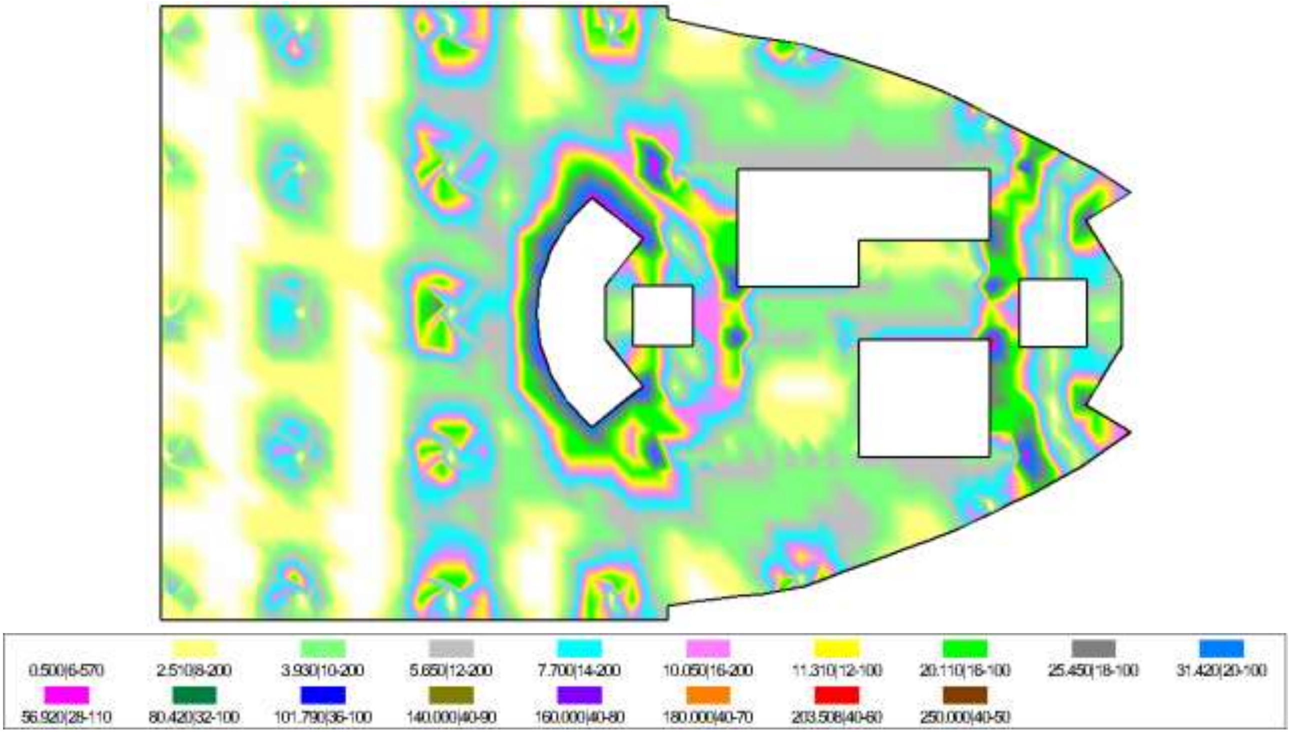

Fig. 22. Reinforcement of the floor slab of the western tower at +42.80 m. Upper longitudinal reinforcement in the direction of the axis *R*, cm^2/m [15]

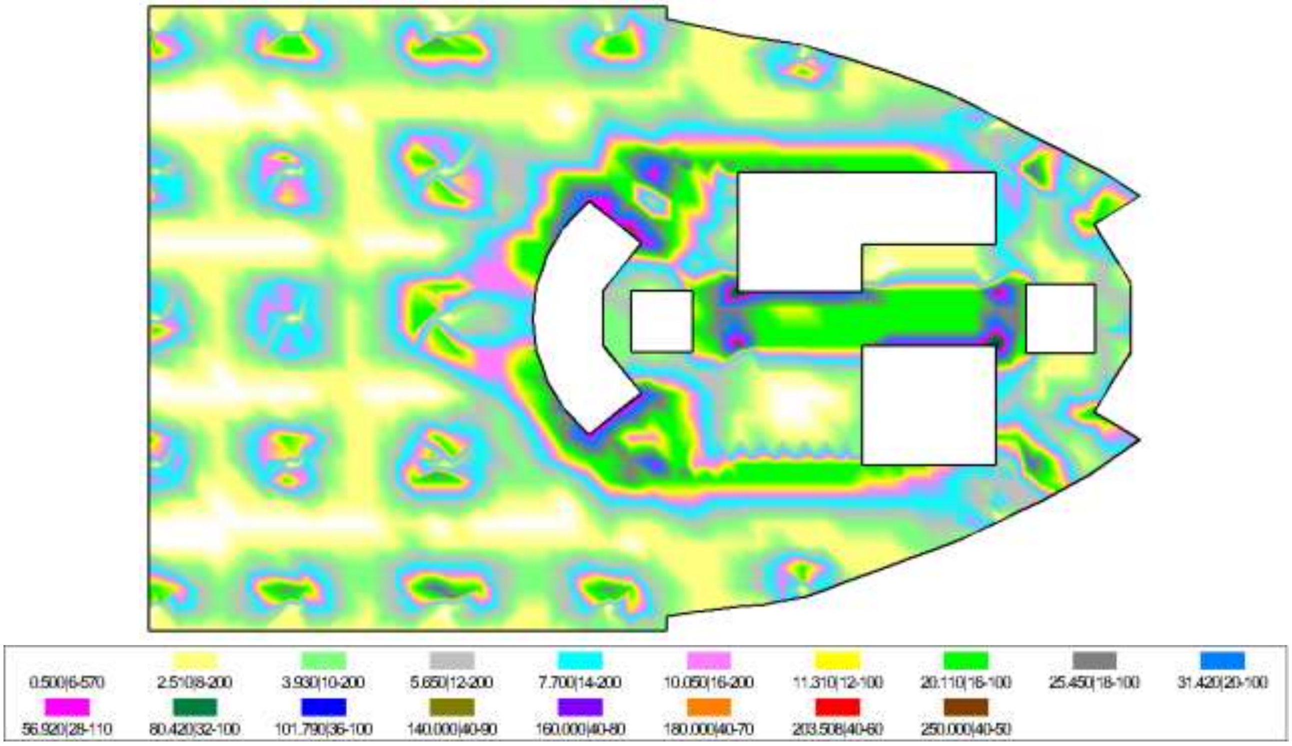

Fig. 23. Reinforcement of the floor slab of the western tower at +42.80 m. Upper longitudinal reinforcement in the direction of the axis *S*, cm^2/m [15]

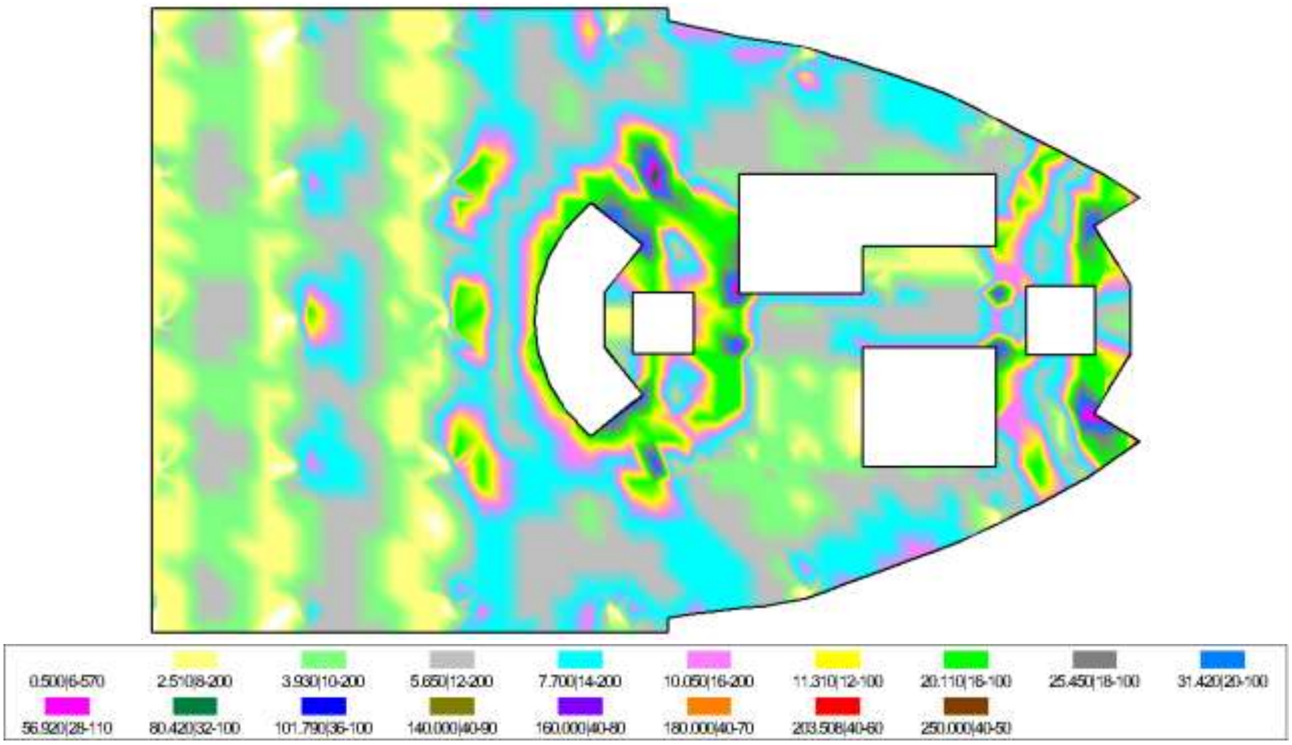

Fig. 24. Reinforcement of the floor slab of the western tower at +42.80 m. Lower longitudinal reinforcement in the direction of the axis *R*, cm^2/m [15]

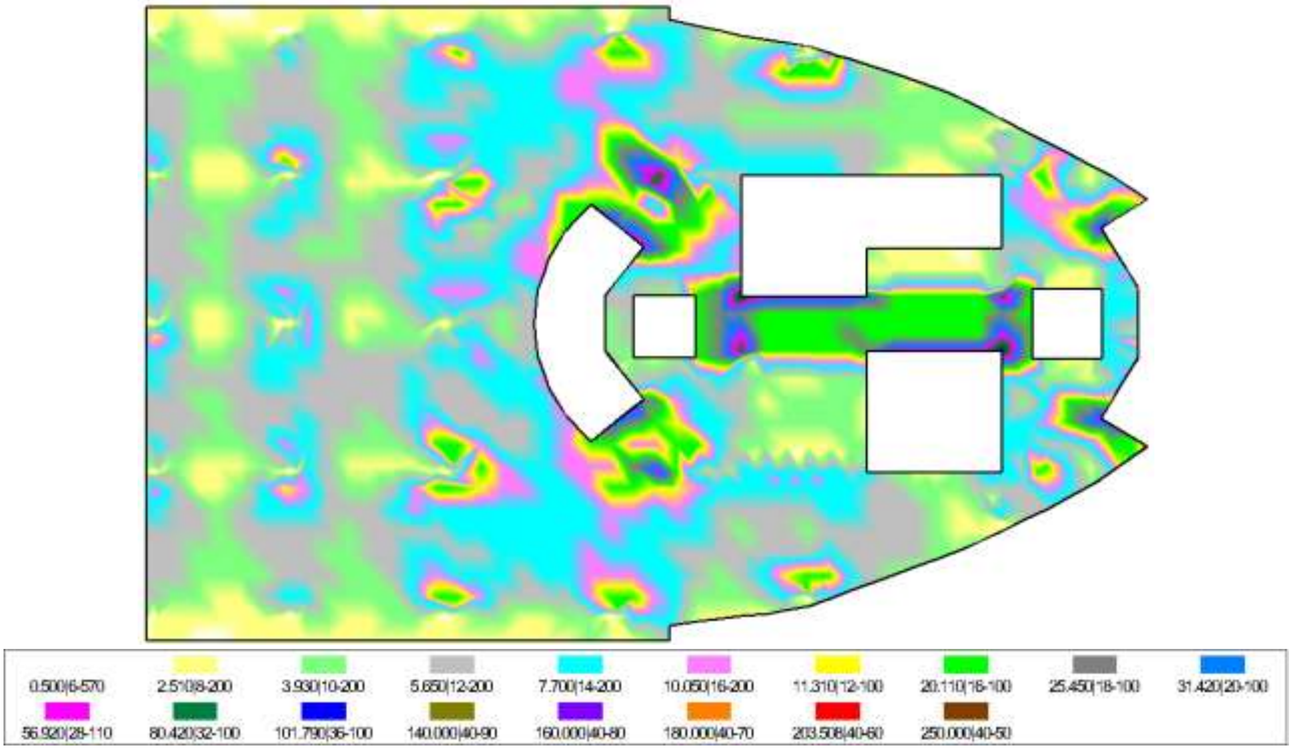

Fig. 25. Reinforcement of the floor slab of the western tower at +42.80 m. Lower longitudinal reinforcement in the direction of the axis *S*, cm^2/m [15]

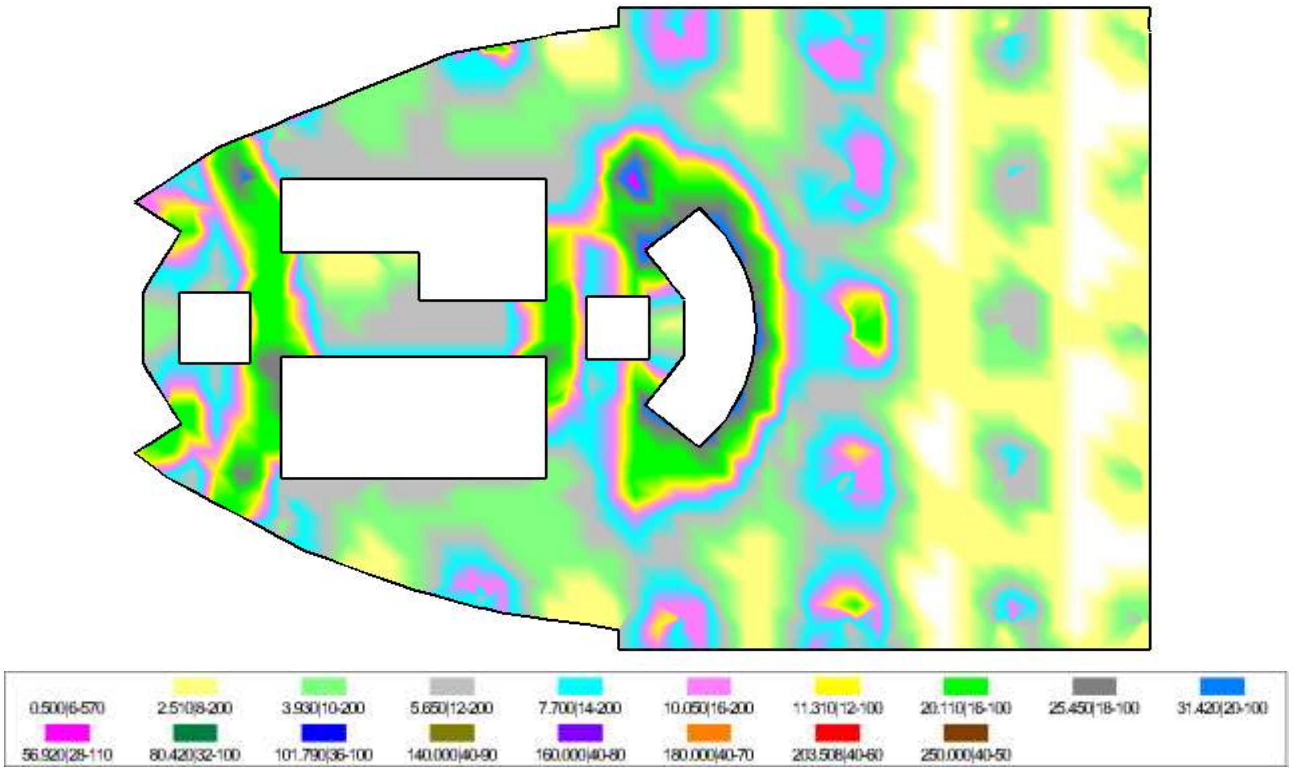

Fig. 26. Reinforcement of the floor slab of the eastern tower at +42.80 m. Upper longitudinal reinforcement in the direction of the axis *R*, cm^2/m [15]

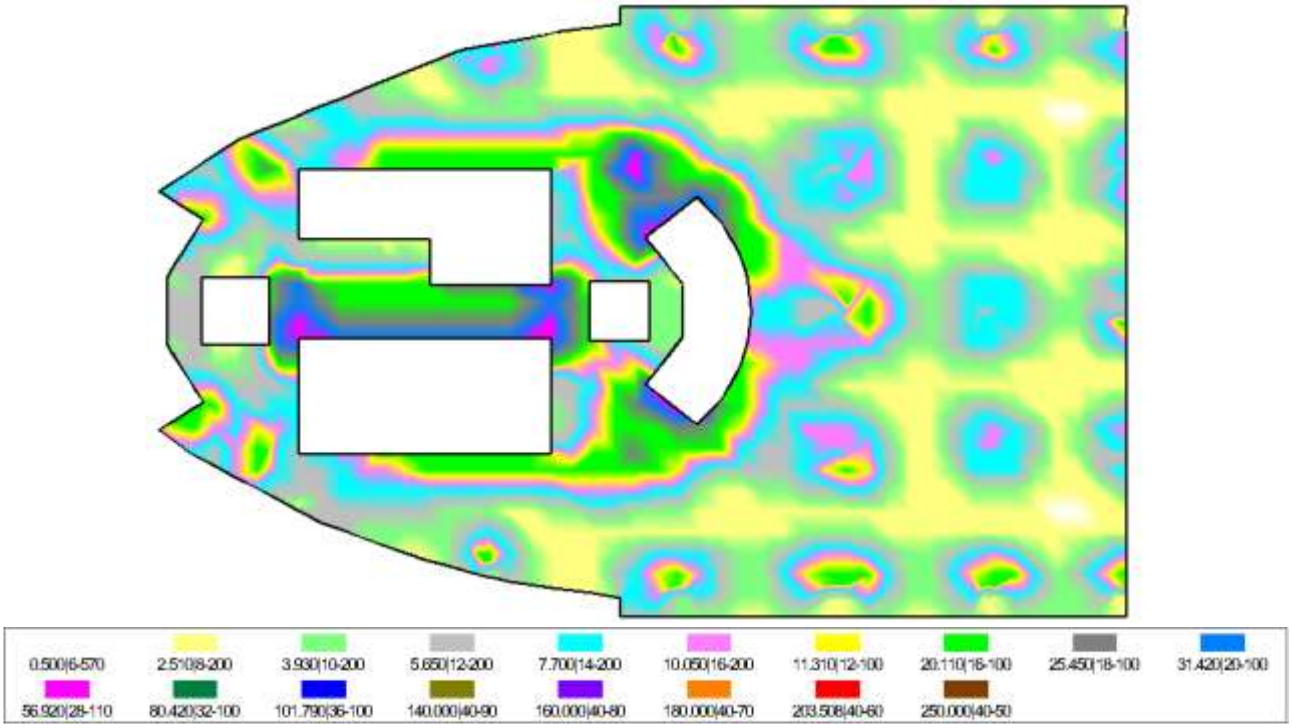

Fig. 27. Reinforcement of the floor slab of the eastern tower at +42.80 m. Upper longitudinal reinforcement in the direction of the axis *S*, cm^2/m [15]

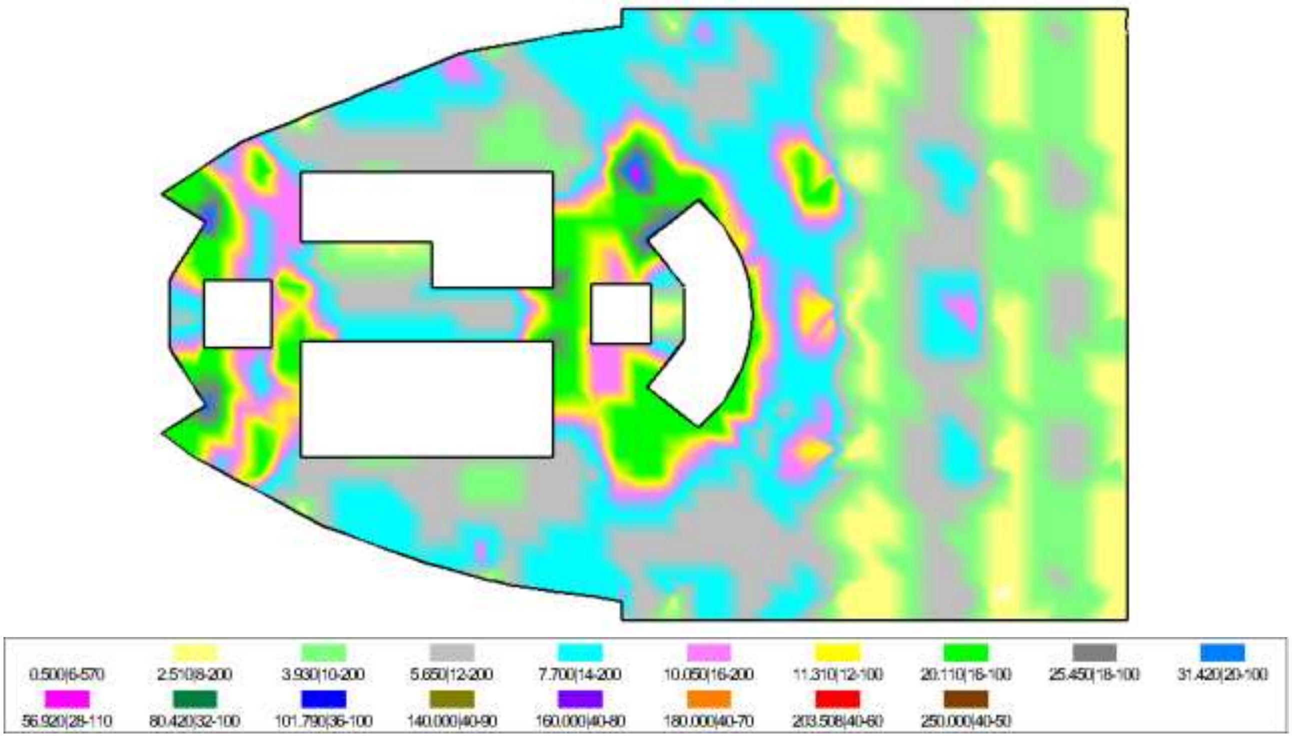

Fig. 28. Reinforcement of the floor slab of the eastern tower at +42.80 m. Lower longitudinal reinforcement in the direction of the axis *R*, cm^2/m [15]

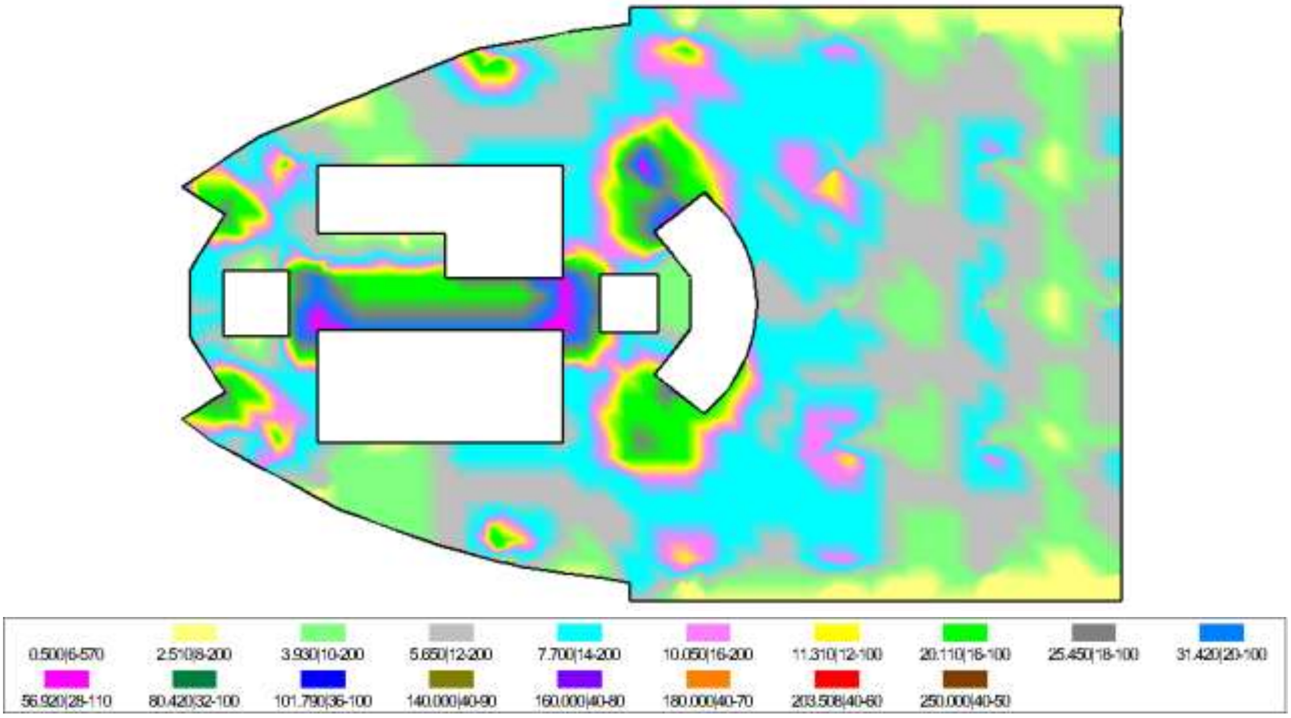

Fig. 29. Reinforcement of the floor slab of the eastern tower at +42.80 m. Lower longitudinal reinforcement in the direction of the axis *S*, cm^2/m [15]

As part of the calculations of the load-bearing structures of a multi-storey building of complex shape, the following works were performed:

1. Analysis of project documentation and research materials. spatial design scheme taking into account the structures of the pile foundation.

2. Collecting loads on structures taking into account the requirements of the norms of the Republic of Azerbaijan, recommendations for the purpose of snow and wind loads.

3. Calculations of the stress-strain and natural oscillations of the spatial calculation schemes of the FEM including verification calculations including for a special combination of loads taking into account seismic effects.

4. Checking the overall rigidity of the load-bearing system of a building, foundation sedimentation, and vertical plate deflections under major combinations of loads and impacts.

5. Calculation of the main elements of load-bearing structures by limiting conditions.

Discussion and Conclusion.

An analysis of the results of spatial calculation of the load-bearing structures of a 29-storey building of complex shape with a frame-braced frame, stiffness cores and a unique mega-truss confirmed the operability of the design decisions made under conditions of seismic impact with an intensity of 8 points. The calculations were performed using the finite element method in the STARK ES PC taking into account 53 loads including special combinations of loads.

The theoretical significance of the study is a comprehensive assessment of the stress-strain of a combined structural system joining monolithic reinforced concrete cores and a 90 m long steel mega-truss. The results obtained confirm the effectiveness of using spatial finite element models to predict the dynamic characteristics of buildings of complex geometric shape. The results of the modal analysis (the first three natural oscillation frequencies: 0.340 Hz, 0.459 Hz and 0.484 Hz) can serve as reference points for verifying similar calculation models and clarifying regulatory approaches to identifying the dynamic parameters of high-rise buildings on weak soils of coastal territories.

The practical significance of the study is due to a possibility of direct application of the results obtained in the design of multifunctional complexes in earthquake-prone areas, including on the drained lands of the Caspian Sea. The key findings important for engineering practice are as follows:

- The maximum foundation draft (50.2 mm) and the relative draft difference (0.00016) are not beyond the regulatory limits, which confirms the reliability of the pile foundation and the rigidity of the foundation plate.
- Horizontal displacements of the top of the building (163.4 mm) are significantly lower than the allowed 300 mm, which guarantees the comfort of operation and the safety of facade systems.
- The deflections of the floor slabs (from 1.2 to 13.8 mm) comply with the requirements of the second group of limit conditions.
- The critical stability parameter (10.4) indicates a high margin of overall stability of the system.
- The selected reinforcement (Fig. 18–29) is sufficient to comply with the requirements of AzDTN 2.3-1, AzDTN 2.16-1 and ensures the load-bearing capacity and crack resistance of reinforced concrete slabs, walls of stiffness cores under basic and special combinations of loads including seismic ones.
- The strength of metal beams complies with the requirements of AzDTN 2.18-1.
- The forces in the piles are not beyond the permissible values.

The suggested structural solutions – stiffness cores combined at the 27th floor level by a steel mega-truss — can be recommended for high–rise buildings of complex shape as an effective way to perceive horizontal seismic loads and ensure spatial rigidity.

Hence the structural solutions adopted in the design of a multi-storey building of complex shape ensure reliability, strength, rigidity and stability of load-bearing building structures with basic and special combinations of design loads and impacts, and the results can serve as a foundation for optimizing the design of such unique structures.

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KKh Mazhiev: participation in the implementation of the study, analysis of the results, correction of the manuscript, formulation of the conclusions.

AKh Mazhieva: participation in the implementation of the study, in the development of a computational and dynamic model of the building, analysis of the results, formulation of the conclusions.

YuV Panasenko: development of a computational and dynamic model of the building, calculations, implementation of the main stages of the study, processing and analysis of the results, formulation of the conclusions.

AKh Mazhiev: participation in calculations, implementation of the main stages of the research, processing and analysis of the results, formulation of the conclusions, preparation of the illustrations, original manuscript and general design of the manuscript.

AKh Mazhiev: participation in calculations, implementation of the main stages of the study, processing and analysis of the results, formulation of the conclusions.

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TECHNOLOGY AND ORGANIZATION OF CONSTRUCTION

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Subject Field Overview

EDN: HSTYXP

General Trends in the Development of Construction Technologies

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Abstract

Introduction. The construction industry depends largely in compliance with the laws of the market with no in-depth analysis of its development trends as a system. Government regulation of the industry fails to make a full use of an evidence-based predictive analysis, but rather is more frequently guided by international experience in the form of small data. The aim of the study is to bridge this gap by means of a general overview of the research related to the general patterns of the development of construction technologies.

Materials and Methods. The research included the search for information from open sources, its analysis and synthesis in order to identify the general trends in the development of construction technologies. Materials from the authors' research were employed. The analysis was conducted using the laws of the development of technical systems.

Research Results. The stages of the evolution of building technologies including prefabricated, monolithic, and precast-monolithic methods are discussed. Ways of improving building materials by means of increasing their physical and mechanical properties, reducing weight, and lowering harmful emissions and costs are also identified. It is noteworthy that the improvement of these materials by their direct relationship with structures results in their dynamic development. It is found that the improvement of materials due to the direct relationship in the system with structures also leads to their dynamic development, they become more durable, lightweight, multifunctional and influence architectural and planning solutions increasing useful selling space. Issues hindering the development of digital technologies for the manufacture of structures are noted: control of early hydration of 3D-printed concrete and a relationship with rheology, ensuring interlayer adhesion, strength, introduction of automated reinforcement and generally the relationship between technology, material and performance characteristics in terms of both structural strength and durability. The basic requirements for the design of buildings and structures and their parts are designed: saving space, materials and energy through integrated design, which includes the integration of all the building systems (structural, mechanical, hydraulic, air and electrical) into a single system. The development of the technology of large-block (modular) construction is considered including the research of SUSU employees on the technology of sinking concrete. Attention is paid to the global experience of modular construction and the direction of development of modular integrated systems.

Discussion and Conclusion. It is concluded that the general trends in the development of construction technologies include: acceleration of large-block (modular) and monolithic construction by improving materials (high-functional concretes, enlarged reinforced frames, fibers), use of automated efficient mechanisms, prefab elements, equipping modules with engineering networks; reducing the complexity and increasing the manageability of construction production by reducing labor costs in the proposed construction technologies, automation and digitalization of the major processes; use of information modeling technologies, neural networks, and rational layout of the interior of a building in complex design; improving the functionality and aesthetics of facade technologies.

Keywords: construction technologies, monolithic construction, 3D printing, modular construction, high-functional concretes, digitalization of construction projects, energy saving of buildings, prefab structures, robotization of construction.

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Обзор предметного поля

Общие тенденции развития строительных технологий

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Аннотация

Введение. Развитие строительного комплекса происходит, главным образом, по законам рынка без глубокого анализа тенденций его развития как системы. Государственное регулирование отрасли не в полной мере использует прогнозный анализ на научной основе, а чаще ориентируется на мировой опыт в виде мало связанных данных. Цель работы – восполнить этот пробел общим обзором исследований с привязкой к общим закономерностям развития строительных технологий.

Материалы и методы. Исследование включало в себя поиск информации из открытых источников, её анализ и обобщение с целью определения общих тенденций развития строительных технологий. Использовались материалы авторских исследований и практического опыта строительства. Анализ проводился с использованием законов развития технических систем.

Результаты исследования. Определены этапы эволюции технологий – сборных, литьевых (монолитных) и сборно-монолитных) – и пути совершенствования строительных материалов с повышением их физико-механических свойств и одновременным снижением массы, вредных выбросов, стоимости. Выявлено, что совершенствование материалов за счет прямой взаимосвязи в системе с конструкциями приводит также к их динамичному развитию, они становятся более прочными, легкими, многофункциональными и влияют на архитектурно-планировочные решения, увеличивая полезное продаваемое пространство. Отмечены проблемные вопросы, сдерживающие развитие цифровых технологий изготовления конструкций: контроль ранней гидратации 3D-печатного бетона и связь с реологией, обеспечение межслойного сцепления, прочности, внедрение автоматизированного армирования и, в целом, связь между технологией, материалом и эксплуатационными характеристиками как с точки зрения структурной прочности, так и долговечности. Сформулированы основные требования к разработке проектов зданий и сооружений и их частей: экономия пространства, материалов и энергии за счет комплексного проектирования, включающего объединение всех систем здания (структурных, механических, гидравлических, воздушных и электрических) в единую систему. Рассмотрено развитие технологии крупноблочного (модульного) строительства, в том числе и научные исследования сотрудников ЮУрГУ по технологии опускающегося бетона. Уделено внимание мировому опыту модульного строительства и направлению развития модульных комплексных систем.

Обсуждение и заключение. Сделаны выводы о том, что к общим тенденциям развития строительных технологий можно отнести: ускорение крупноблочного (модульного) и монолитного строительства за счет совершенствования материалов (высокофункциональных бетонов, укрупненных армокаркасов, фибры), применения автоматизированных эффективных механизмов, префаб-элементов, оснащения модулей инженерными сетями; уменьшение трудоемкости и повышение управляемости строительного производства за счет снижения трудозатрат в предлагаемых строительных технологиях, автоматизации и цифровизации ведущих процессов; использование в комплексном проектировании технологий информационного моделирования, нейронных сетей, рациональной компоновки внутреннего пространства здания; повышение функциональности и эстетичности фасадных технологий.

Ключевые слова: строительные технологии, монолитное строительство, 3D-печать, модульное строительство, высокофункциональные бетоны, цифровизация строительных проектов, энергосбережение зданий, префаб-конструкции, роботизация строительства.

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Introduction. The development of construction technologies is in line with the general patterns of development of technical systems: S-shaped development; completeness of system components; "energy conductivity" of a system; coordination of the rhythm of system components; uneven development of system composition; transition to a suprasystem; displacement of a human and an increase in the level of controllability; dynamization. These patterns have a multiplicative character, i.e. the implementation of one law occurs through the action of others. These trends can also be traced in construction while investigating the interaction of the "materials — structures — building parameters" system. The aim of this literature review is to identify the relationships and patterns of interaction between the elements of the system in order to concentrate on the most important and promising areas of development of the construction industry, which will be beneficial for all parties involved in a construction complex and student instruction.

Materials and Methods. The study employs the classical method of reviewing scientific literature with an analysis of a variety of factors and features identifying the relationships of elements in the construction system, promising areas of development, undesirable effects and possible ways to minimize them. Based on the considered information sources, the research data was systemized and generalized, hypotheses were set forth. According to the sources, the most important and informative ones were selected, including foreign and domestic ones, as well as describing the article authors' own developments.

Research Results. Based on the analysis of the global experience provided in the information sources, in the construction sector it is possible to note the evolution of the major technology options for the production of building structures (prefabricated, cast (monolithic) and prefabricated ones):

- hand-made masonry of small and large blocks, wooden products;
- mechanized installation of enlarged products (shields, panels, modules, blocks of large–span coatings);
- mechanized installation of enlarged products of increased or full factory readiness (with windows, trim, engineering systems) — the so-called prefab elements;
- automated and robotic assembly of prefabricated products.

These are not only materials for prefabricated construction that are evolving, but also methods of connecting prefabricated elements: from wet processes and labor-intensive welding, they are moving to dry and technological connections (bolted, pin, coupling, loop, etc.).

In injection molding technology using concrete solutions and other building mixtures, the same stages are observed with the displacement of humans and the increase in controllability:

– manual injection molding technology;
– mechanized production, transportation and laying of mixtures;
– use of formwork units and reinforced formwork blocks;
– robotic technology for 3D printing and installation of buildings from pre-printed 3D blocks.

At the same time, materials are being improved with an increase in their physical and mechanical properties and a reduction in weight, harmful emissions, and cost. On top of that, the development of materials is moving from the macro to the micro and further to the nanoscale. The prediction and design of material properties first takes place at the level of macrostructures, then molecules, atoms and elementary particles. Traditional concretes are being replaced by highly functional, high-strength, self-sealing ones with extensive use of mineral and chemical additives. New functions of materials are emerging, such as self-purification by adjusting the microstructure using photocatalytic additives.

Heat-efficient hollow bricks and stones are replacing solid ones. According to the development line of technical systems, the next step should be a capillary-porous material (CPM) with a certain structure, e.g., increasing the vapor permeability of a material from the inner surface to the outer due to the size of the pores. Pores can be filled with another substance, such as glue that will be released from the surface layer of the wall blocks holding them together. Due to the controlled microstructure, the principle of self-renewal of paints on the facade can be implemented.

Various building composites with special properties are broadly employed: self-cleaning; healing of cracks; absorption of greenhouse gases. Simple reinforcement of cast structures is being replaced by an adjustable prestressed or ferromagnetic fiber with the ability to control the magnetic and thermal fields.

The evolution of materials is the foundation for improving structures. Structures are evaluated by specific indicators of the ratio: mass to span, height or overlapped area; operational characteristics (strength, heat resistance, sound insulation, durability, etc.) to their own weight, cost of production and installation. These simple relations expressing the law of increasing the degree of ideality show how progressive changes occur in building materials, structures and structures over time.

Recently the term "digital concrete production" has been coined [1]. Digital methods for production of concrete and binders have become the subject of a great number of studies and industrial activity, and the industrialization of technologies such as 3D printing is becoming more and more realistic. The potential for revolutionary changes in construction is growing not only by means of reducing costs, but also by increasing environmental sustainability and functionality. The problems associated with materials for printed concrete are plenty. The major ones are control of early hydration and a relationship with rheology, provision of interlayer adhesion, strength, introduction of automated reinforcement and generally a relationship between technology, material and performance, both in terms of structural strength and durability. An interdisciplinary approach is crucial as this field brings lots of disparate areas together and has so far been driven only by areas such as architecture and construction.

New structures, becoming more durable, lightweight, and multifunctional, affect architectural and planning solutions, increasing the useful selling space. A simple example of such a solution is obvious while replacing a frame with crossbars protruding beyond the ceiling plane with a frame with "secret" crossbars forming a flat ceiling together with the ceilings (e.g., houses of the MKT construction built in Miass, Chelyabinsk region). So, if the bolt protrudes beyond the ceiling plane by 30 cm, in a 10-storey building when smooth ceilings are installed, a full floor is lost. In office, commercial, and sports buildings, loss of space is associated with installation of dimensional engineering systems at a ceiling or floor level. Reducing the thickness of external enclosing structures by means of innovation provides significant savings in usable space in multi-storey buildings. The "3 for 2" concept (3 floors for 2) is aimed at saving space, materials and energy by means of integrated design [2]. As the world adapts to the twin trends of climate change and urbanization, high-rise office buildings in hot and humid climates are the first in line for a significant change in design approach. An integrated design approach pays special attention to reducing the size of engineering communications introducing a new paradigm for optimizing use of space, materials and energy in buildings: integrated integration of all building systems - structural, mechanical, hydraulic, air and electrical ones - throughout the life cycle of a building from early design to construction and operation.

The "3 for 2" concept has been implemented in a pilot project of a non-profit international school in Singapore[1]. In Singapore's new commercial buildings, up to a third of the enclosed volumes are typically occupied by technical systems and structural elements taking up valuable space that could otherwise be reserved for residents. The standard centralized air conditioning systems currently in use are among the main consumers of this space. The 3 for 2 project demonstrates air conditioning technologies that were previously seldom used in commercial buildings in Southeast Asia, such as passive cooling beams and distribution ventilation systems.

The review "The Science of Concrete: the Past, Present and Future of Innovation" [3] notes that concrete, as the most commonly used building material, is rapidly evolving, but at the same time is facing challenges in terms of environmental impact, financial needs, public recognition and image. Studies of radical changes in three major aspects of concrete use are relevant: reinforcement, binder content, and manufacturing methods. It is assumed that, along with introducing robotic production methods, digital technologies can be key to introducing a number of innovations: reinforcement with no reinforcement bars using non-convex granular media; concrete structures optimized for compression using topology optimization, architectural geometry and 3D printing or origami-style formwork; genuinely digital concrete due to a combination of mass data collection and deep learning.

However, interlayer bonding is paramount for printed concrete as well as for traditional concrete. The bonding strength during the multilayer laying of self-sealing concrete was studied in [4]. SUSU studies [5, 6] indicated a positive effect of

[1] *United World College, Singapore.* URL: http://www.systems.arch.ethz.ch/de/research/synergistic-buildings/3for2-beyond-efficiency.html (accessed: 18.07.2025).

acoustic treatment of concrete on its water absorption, which increases the adhesion to old concrete, and also improves the quality of technological joints using printed matrices, various adhesives, additives, and slag-alkaline concretes.

For 3D printing of concrete, the problem of adhesion is addressed by installing reinforcement or introducing fiber into the mixture. As the use of steel reinforcement is compulsory in most building structures, there is an urgent need to develop reinforcement technology for 3D-printed structural elements. In [7], a justification was carried out for 3D printing of steel reinforcement using arc welding with a metal electrode in a protective gas environment. The mechanical characteristics of the printed rods demonstrated comparable mechanical properties to conventional steel reinforcement of the same diameter.

We find it reasonably more technologically advanced to make use of an additional 3D printer manipulator that unwinds and cuts wire reinforcement into the necessary segments, or to make use of fibers from various materials. The results of such developments were reported at the First RILEM 2018 International Conference on Digital Concrete.

It was noted that most studies of printed concrete had been conducted on small models and failed to take large-scale effects into account. Currently, bridges have been implemented among large-scale digital concrete (DFC) structures (e.g., in China, the Netherlands, and Spain), as well as two-storey houses printed in 22 days in China. Large-scale tests of elements manufactured using DFC technology have shown [8] that caution is required as tests of materials related to DFC are being developed, and the large-scale effects of DFC have not been studied in practice. Therefore, it is recommended that large-scale tests in the range from 1:5 to 1:1 are conducted if DFC is applied to responsible structures.

Another significant downside of printed concrete is its low productivity of digital DFAB production in construction. In [9], the costs and time for the robotic construction of a wall was analyzed. In this example, it was found that productivity was higher while using the robotic construction method for complex walls (e.g., an exterior and decorative one), which indicates a possibility of obtaining significant economic benefits from using additive DFAB specifically for construction of complex structures. However, DFC technology is not applicable for high-rise buildings. However, this does not mean that the traditional technology of prefabricated and monolithic housing construction will not be automated and robotized. Back in 1995, an all-weather automated system for the construction of high-rise reinforced concrete buildings was presented at a conference in Belgium [10]. For the first time in the world, it had been used in the construction of a 15-storey reinforced concrete building in Chiba Prefecture in 1995 (Fig. 1).

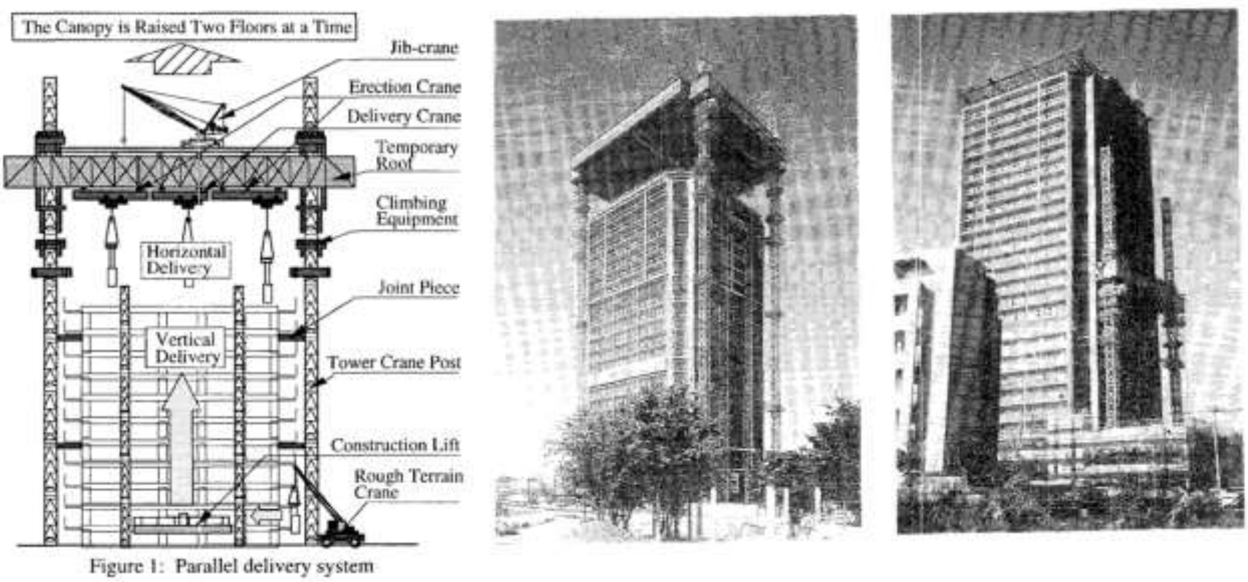


Fig. 1. Automated high-rise building construction system [10]

This system includes four major elements:

1) synchronously rising all-weather temporary structure;

2) parallel material supply system;

3) factory production and unification of building materials;

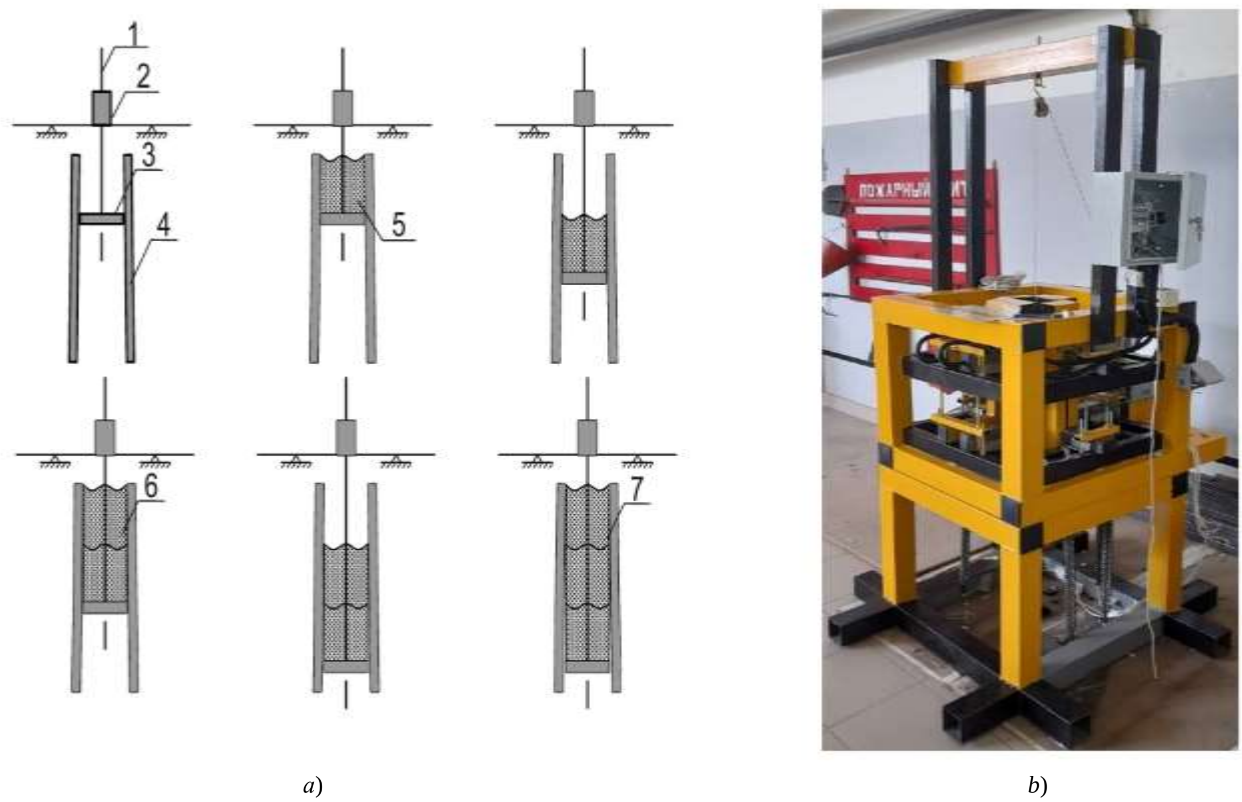
4) material management system.

It ensures high quality, improves working conditions, reduces construction time, labor force and waste, and increases the overall productivity. As shown in Fig. 1, in the BIG CANOPY project a parallel element supply system was installed for three automated overhead cranes to be set up and one large construction lift under an all-weather lifting temporary mounting beam.

The project makes use of a materials management system using a database linked to the CAD system, unification of building materials, factory manufacturing, as well as parallel installation. The major advantages of the technology are increased productivity and quality; shorter construction time; improved production conditions (workers can safely and comfortably work under a temporary roof); reduction in the amount of garbage.

A similar technology (only related to injection-molded structures rather than prefabricated ones) has been patented and is being developed at the SUSU Department of Construction Production and Theory of Structures [11–13]. The shuttering system is made up of two vertical decks with minimal expansion at the bottom. Vertical decks are stationary. The horizontal deck located between them is lowered or raised with hydraulic jacks. The horizontal deck is connected to the lifting or lowering mechanisms with a metal vertical rod. The new technology is a kind of anti-sliding formwork system that moves upward.

The process is performed as follows: in the initial state, the horizontal deck of the formwork is lowered relative to the top of the vertical deck. The composite material (concrete mix) is fed into the formwork system where the mix is placed in the space between the horizontal and vertical deck. There is a film between the concrete deck system which moves downwards with the mixture. This method removes effects such as adhesion and cohesion leaving only a minimum friction between the film and the vertical deck and provides protection against premature drying of the concrete. The first layer of the concrete mixture gains a specific strength with a possibility of heat treatment, acoustic and other effects on the working seams. The horizontal deck is lowered with jacks along with the first layer down. Then the second layer is laid on the first one and having gained strength, it descends. The third layer is laid (Fig. 2). Then the lower sufficiently hardened layers extend beyond the vertical deck, etc.



a) *b*)

Fig. 2. *a*) method of forming reinforced concrete structures by means of lowering concrete (patent 2566540): 1 – metal rod or cable; 2 – lifting and lowering equipment (hydraulic jack); 3 – horizontal deck; 4 – vertical deck; 5 – first layer of concrete; 6 – second layer of concrete; 7 – third layer of concrete; *b)* experimental SUSU setup for the technology development

The technology of sinking concrete can be employed for construction of deep underground structures (mines, hazardous waste storage facilities, underground military installations, etc.), underwater structures on a shelf, manufacture of block rooms in mobile workshops for modular construction, etc. Structures lowered by means of the counterweight method can provide lifting of aboveground structures by implementing the top–down construction technology. The advantage of the new monolithic technology is the immobility of the formwork system and a possibility of equipping complex systems for processing and monitoring the condition of the concrete mixture and concrete under protection from external influences, up to full automation of production processes.

Automation of the installation of multi-storey buildings was proposed back in the USSR by replacing free installation with cranes for forced installation with manipulator cranes[2] [14]. A traditional mounting crane provides only one of the 16–20 operations of the assembly of prefabricated structures, and the remainder are performed manually by workers whose labor costs are 4–5 times more than the operating time of the crane. Experiments were performed with an automatic crane based on the BKSM-5–5A tower crane, but a low positioning accuracy of ± 20 cm was obtained, which prevented a transition to industrial development accuracy of ± 20 cm was obtained, which prevented a transition to industrial development.

In Japan, a two-stage system is employed controlled by a single operator and consisting of free feeding of the prefabricated element by a conventional tower crane and a robot (manipulator) installing the element at the design location.

In Russia, three variants of forced installation have been developed. The idea behind the new method is that in order to increase the productivity and accuracy of installation, the mounted element is gripped rigidly and fed into the design position with registration of linear and angular displacements along the three axes X, Y, Z (in six coordinates). At the same time, there is a task of ensuring the stability of the elements after their installation and uncoupling of the crane handle. This is achieved by using the spatial self-fixation method using locking traps of various designs, which at the same time reduces the requirements for positioning accuracy. A manipulator crane with a rigid connection between the working body and the transportation device has been developed that has a minimum required number of degrees of freedom of the movement device, and thereby a minimum number of drive mechanisms.

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However, the suggested sparsely populated technologies were not commonly used in the USSR due to the lack of digital design, the complexity of the design of cranes, gripping devices, and the need to retrofit prefabricated elements with self-locking devices. As the rest of the operations (suturing, welding ties, sealing) are performed manually in the traditional way, there is a law of uneven development of parts of the system and misalignment. As shown by the further development of the construction industry, complete robotization was achieved not in the assembly, but in the injection molding technology of 3D printing, and only when the power of computing complexes had allowed to carry out digital building projects [15], and the rheology of concrete composites had been managed [3].

The record of Chinese builders who had worked on a prefabricated 10-storey building in 29 hours was confirmed by means of the method of the fastest construction of large blocks (prefab blocks, modules). Block rooms were designed in a few cities of the USSR (Krasnodar, Minsk, Verkhnyaya Salda, etc.) but were limited by the lifting capacity of cranes and transport dimensions. The incentive for the development of that type of housing construction was a number of advantages of those houses over large-panel ones [16, 17]: manufacturing volumetric blocks of full factory readiness in factory production with built-in furniture, plumbing and electrical equipment; transferring 85% of labor costs to a factory or house-building complex; reducing labor costs on the construction site to 25%; reducing the number of mounting elements by 4–5 times; reducing construction time by 3–4 times; reducing labor intensity by 2.5–2.8 times; reducing concrete consumption by 25–28%; significantly reducing overhead costs and costs for temporary facilities, etc.

The sizes and weight of the reinforced concrete blocks were small. The Krasnodar House-Building Plant produced blocks in the form of a ribbed box of the "lying glass" type measuring 3□5 m and weighing 5–9 tons depending on the

[2] Vilman YuA Technology of Construction Processes and Construction of Buildings. *Modern Progressive Methods: a Textbook.* Moscow: ASB; 2011. 336 p.

equipment. The modular product of JSC SZ "OBD" in Krasnodar has presently been significantly improved and reaches a weight of 20 tons and allows for the construction of a 16-storey building (16,000 m^2) in 4 months. Minsk "cap" type blocks of approximately the same size and weight were used for three-dimensional block houses of the 3A-OPB series. Since 2012, the block house construction plant "Choice–OBD" has been operating in Voronezh for the production of reinforced concrete modules of the „cap“ type [18].

The development of the construction industry has now made it possible to significantly increase the size and weight of modules for block construction. The Moscow plant of Concern MonArch LLC (Fig. 3) produces the world's largest room blocks with dimensions up to 15,5×7,5×3,75 m, with an area of over 100 m^2 and weighing up to 40 tons in a basic frame and 55–58 tons with complete finishing.

Fig. 3. Technological portal conveyor (Q=100 t) for conveyor assembly of MonArch Group modules

The technology allows for the installation of 25 modules (1800 m^2) per day, 450 thousand square meters of ready-made housing per year. For the first time the world has witnessed the production of three-storey modules for stairwells and elevator shafts. Due to the maximum robotization and automation of factory module manufacturing processes, there has been a significant reduction in the construction time of houses. The first stage of the Vnukovo plant was launched in July 2023.

Among the major difficulties of MonArch's modular construction technology, the complex logistics of overnight delivery of modules using two lanes of traffic and the need for powerful cranes (typically manufactured in Germany or China) is to be noted.

The MonArch concern is currently producing about 2000 modules per year and participates in the renovation program. In 2025, a kindergarten with 300 places was built in two weeks’ time from 114 large-sized modules in Sireneviy Boulevard in Moscow.

Foreign modular construction systems have been known since the days of Habitat 67 residential complex (1967) and Nakagin Tower in Tokyo (1972) mainly make use of metal-framed blocks such as ADK Modulraum (Germany), Vision Modular, Assael (England), etc. The metal frame is also used for production of block cabins for cruise ships.

Special MEP modules can also be supplied in block design [19]. Modular engineering systems — "mechanical, electrical, and plumbing systems" (MEPs) — involve the pre-fabrication and assembly of engineering system components under controlled factory conditions before they have been delivered to a construction site to be installed. This approach provides lots of benefits, including increased efficiency, cost savings, faster project deadlines as well as improved quality control.

These modules combine various MEP components such as pipelines, ductwork, electrical wiring, and equipment into a single unit. Equipment installed in the MEP units: frames with heat exchangers, pumps or other engineering equipment with integrated pipelines and electrical systems; fully equipped bathrooms with plumbing, electrical and HVAC components (Heating, ventilation, and air conditioning); suspended mechanical racks with pipelines, ducts and electrical systems for efficient distribution in a building or a structure.

Problems related to environmental friendliness, alternative energy sources and lifestyle changes (e.g., due to the pandemic) have led to a surge in the production of buildings using modern construction methods, particularly in residential construction. These methods involve use of new technologies as an alternative to traditional construction in smart buildings. Against the background of the development of industry 4.0, there is an urgent need for integrated design using machine learning, neural network and generative algorithms [20].

On top of that, it is essential to pay attention to the research results where optimization tools were employed in order to develop energy-efficient and rational construction schedules [21]. As the growing global awareness of environmental issues is on the rise, the construction industry is forced to make use of innovative materials and methods in order to make the construction process more energy efficient and environmentally friendly. Project managers should employ optimization tools in their planning procedures in order to address these issues in the early stages of project justification.

Discussion and Conclusion. Thus, among the main trends in the development of construction technologies, the following are to be noted:

– rapid evolution of building materials with an emphasis on high-functional concretes, new composites, capillary-porous materials and environmentally friendly waste materials with the possibility of their recycling and reuse;

– digitalization of projects and technologies: BIM design (with the development of 3D, 4D, 5D...); transition to electronic document management (GOST 70108-2025 "Executive Documentation. Formation and Maintenance in Electronic Form"); 3D printing of structures and formwork; automation of supply processes, construction control, organization and management; precise positioning and automation of construction machinery;

– improvement of factory readiness, dimensions, and architectural and structural quality of prefabricated elements (the so-called precat and prefab technologies);

– development of modular construction in Russia from large-sized frame and reinforced concrete blocks with partial and complete finishing;

– improving the functionality and aesthetics of facade technologies: architectural and structural facade panels; self-cleaning and self-healing of facade materials; intelligent facades;

– development of monolithic construction using highly functional concretes, enlarged reinforcing frames, fibers, high-performance mechanisms, as well as prefab elements for complex multi-part building elements in order to speed up construction;

– complete robotization of some finishing processes;

– implementation of energy-saving and "green" solutions in engineering systems of buildings (not only unique ones);

– improving the quality of landscaping, infrastructure, and the urban environment quality index.

As a result of the review of the global experience, the general trends in the development of technologies in construction have been formulated that are to be noted by construction companies concentrated on continuous development and aspiring for the leading positions in the industry.

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Original Empirical Research

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Analysis of Timing Results during Continuous Concreting of a Massive Foundation Slab

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Abstract

Introduction. Risk assessment of early cracking during hardening of massive monolithic reinforced concrete foundation slabs due to temperature gradients enhances the relevance of studies of a host of factors related to a concreting technology given the technical capabilities of workers and suppliers of concrete mixtures, as well as weather conditions. While developing technological regulations for concreting with a calculation of the thermal stress state in the early period in order to reduce and control heat dissipation, studies in the field of assignment and regulation of time parameters of the process of forming the body of the foundation slab taking into account the prescription features of concrete mixtures and weather conditions are relevant. The aim of the study is to obtain new data for calculating the time parameters of concreting massive structures using concrete pumps with technical characteristics that are not available in the regulatory framework.

Materials and Methods. The paper presents the results of timing the process parameters of continuous concreting of a massive foundation slab with a volume of 1642 m^3 in 13.6 h. The numerical values of the concrete mixture pumping speed, maneuvering time of concrete mixer trucks as well as the coefficients of transition from technical to operational performance of concrete pump trucks with a technical capacity of 170 and 180 m^3/h are obtained. The use of concrete pumps with such a capacity at an actual unloading speed of concrete mixer trucks of up to 2.3 m^3/min ensures an actual pumping performance coefficient of up to 0.81, which basically corresponds to normal operation.

Research Results. The values of the concrete pump utilization coefficient over a time period ranging from 0.478 to 0.841 with an average value of ≈ 0.66 were obtained. The actual average productivity of one concrete pump during the concreting period was ≈ 61 m^3/h.

Discussion and Conclusion. With a distance from the concrete pump truck to the concrete mixer truck waiting area within 25–50 m, the maneuvering time does not depend greatly on the distance, according to 69 measurements, it is no more than 5.76 min with a reliability of 0.95 and is identified by the convenience of an area for maneuvering concrete mixer and access roads. The results can be used in developing process regulations for continuous concreting of similar massive structures.

Keywords: foundation slab, continuous concreting, pumping speed, operating cycle parameters, concrete pump

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Оригинальное эмпирическое исследование

Анализ результатов хронометража при непрерывном бетонировании массивной фундаментной плиты

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Аннотация

Введение. Оценка риска раннего трещинообразования при возведении массивных монолитных железобетонных фундаментных плит вследствие температурных градиентов предопределяет актуальность исследований многочисленных факторов, связанных с технологией бетонирования, с учетом технических возможностей производителей работ и поставщиков бетонных смесей, а также погодных условий. При разработке технологических регламентов бетонирования с расчетом термонапряженного состояния в ранний период с целью уменьшения экзотермии и управления кинетикой тепловыделения актуальными являются исследования в области назначения и регулирования временны́х параметров процесса формирования тела фундаментной плиты с учетом рецептурных особенностей бетонных смесей и погодных условий, а также расчет термонапряженного состояния в ранний период. Цель исследования: получение новых данных для расчетов временны́х параметров бетонирования массивных конструкций с использованием автобетононасосов с техническими характеристиками, данные о которых отсутствуют в нормативной базе.

Материалы и методы. Приведены результаты хронометража технологических параметров непрерывного бетонирования массивной фундаментной плиты объемом 1642 м^3 за 13,6 ч. Получены численные значения скорости перекачивания бетонной смеси, времени маневрирования автобетоносмесителей, коэффициентов перехода от технической к эксплуатационной производительности автобетононасосов с технической производительностью 170 и 180 м^3/ч. Использование автобетононасосов с такой производительностью при фактической скорости разгрузки автобетоносмесителей до 2,3 м^3/мин обеспечивает коэффициент фактической производительности при перекачивании до 0,81, что в принципе соответствует нормальной эксплуатации.

Результаты исследования. Получены значения коэффициента использования автобетононасосов по времени от 0,478 до 0,841 при среднем значении ≈ 0,66. Фактическая средняя производительность одного автобетононасоса за период бетонирования составила ≈ 61 м^3/ч.

Обсуждение и заключение. При расстоянии от автобетононасоса до площадки ожидания автобетоносмесителей в пределах 25–50 м время маневрирования мало зависит от расстояния, составляет с обеспеченностью 0,95 по данным 69 замеров не более 5,76 мин и определяется удобством площадки для маневрирования автобетоносмесителей и подъездных путей. Полученные результаты могут быть использованы при разработке технологических регламентов на непрерывное бетонирование аналогичных массивных конструкций.

Ключевые слова: фундаментная плита, непрерывное бетонирование, скорость перекачивания, параметры рабочего цикла, автобетононасос

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Introduction. Over the recent years, there has been a surge in the number of floors of residential buildings constructed, particularly in major cities in the conditions of dense urban development. According to [1], in 2019 the average number of floors in housing construction was 17.7 floors per 1 m^2 of a constructed building with the average number of floors of 19.7 in the Rostov region. In 2022-2025 buildings of up to 32 floors tall are under contruction in Rostov-on-Don with buildings up to 45 floors tall being considered. Flat foundation slabs (FSs) or slab grillings along a pile field are commonly used as foundations for such objects. The recommended thickness of the FS is from 0.5 to 2m according to section 7.10 of SP 52-103-2007 "Reinforced Concrete Monolithic Structures of Buildings" and up to 3 m according to Section 5.2.7 SP 430.13225800.2018 "Monolithic Structural Systems. Design Rules". In SP 430.13225800.2018, Section 5.2.7 allows for "the first approximation of the thickness of a flat foundation slab on a natural foundation to be equal to 1/65 ÷ 1/50 of the building height". Therefore the FSs for medium and higher buildings of the Rostov region will relate to massive monolithic reinforced concrete structures. According to SP 435.1325800.2018 (Section 9.3.4), "the method of laying a concrete mixture is to ensure the solidity of a structure"; according to Section 5.12, "the method of feeding a concrete mixture is to be selected according to the calculated value of the concreting intensity"; according to Section 6.2.8, "the rate of flow of a concrete mixture to a facility and pumping is to ensure the continuity of the supply of a concrete mixture. Technological breaks should not be over 20 minutes long".

According to [2, 3], the quality of massive monolithic concrete structures is impacted by a host of factors related to concreting technology considering the technical capabilities of work manufacturers and suppliers of concrete mixtures, as well as weather conditions. The multitude of factors makes it imperative to address the issues necessary for developing technological regulations (TRs), while the necessary data and instructions are not always provided in corresponding regulatory documents. Temperature gradients, which cause tensile stresses and pose a threat of early cracking due to a heat release of hardening concrete, are a severe problem. In compliance with STO NOSTROY 2.30.214-2016 "Monolithic Concrete and Reinforced Concrete Structures. Work Requirements. Rules and Methods of Control", heat release is regulated by means of reducing exothermy and controlling kinetics, which, while calculating the risk of early cracking [4–6], requires that the time parameters of the process of forming the body of the OP are identified in a precise manner considering the prescription characteristics of concrete mixtures [7] and weather conditions making research in the field of appointment and regulation of the concreting time parameters of a process relevant while developing TPs calculating the thermally stressed state in the early period. A clear algorithm of technological calculations in developing TPs is needed, among other things, to the importance of identifying the rational overlap time of layers depending on the preservation of the concrete mixture, temperature and humidity conditions, manufacturer's technical capacities (concreting rate) and can vary greatly [8, 9]. It is of particular importance to identify the time limit properly for overlapping layers and concreting rate in dry, hot weather [10].

According to [11, 12], the intensity of continuous concreting of a structure can be up to 226 m^3/h, and according to [7] — even up to 450 m^3/h depending on the number of concrete pumps (CPs), and the total duration of continuous concreting can be up to 35 hours and even up to 3 days long. The well-grounded indicator of "concreting intensity" primarily based on one CP is one of the most critical parameters for developing a concrete processing plant and a concrete-laying complex [10]. Provided that supply of concrete mix to the facility is properly organized, concreting intensity will be determined by the technical performance of a concrete mixer truck (CMT) during unloading (unloading speed), as well as of a concrete pump (CP) (pumping speed) and the efficiency of organization (CMT utilization factor in time). In order to identify the required number of CPs and CMTs, data on their operational performance is needed depending on the pumping speed and CMT utilization factor in time. The technical capacity of CPs during unloading, according to the instructions for transporting and laying concrete mixtures in monolithic structures using CMTs and CPs, ranges from 0.5 to 2 m^3/min or up to 120 m^3/h, and the unloading speed, according to the recommendations for the delivery of concrete mixtures by motor vehicles, can be up to 2.5 m^3/min or up to 150 m^3/h [13]. According to [14], the technical capacity of CMTs is up to 200 m^3/h. The operational productivity of the concrete pump, which determines the concreting intensity, depends on a host of factors and is generally determined by the coefficient of transition from the technical productivity of the concrete pump to the operational one [15, 16], as well as the coefficient of utilization of the concrete pump over time [13].

According to [10], during FS concreting with a total volume of about 1,500 m^3, 10 m^3 of the concrete mixture was directly supplied by a concrete pump with a technical capacity of 120 m^3/h with a delivery range of up to 50 m in an average of 7.5 minutes, which is approximately 80 m^3/h and corresponds to a transition coefficient from technical to operational capacity of $80/120 \approx 0.67$. In technological maps, the value of the coefficient that accounts for organizing (the CMT utilization factor in time) is typically assumed to be 0.65 [10, 13].

The aim of the study is to obtain new data for calculating the time parameters of concreting massive structures using technological equipment (concrete pumps) with technical characteristics not available in the regulatory framework, particularly, in the Government Elementary Estimated Norms (GEEN). The paper shows the results of identifying and analyzing some parameters of a technological process, including the concreting intensity, during continuous concreting with a volume of 1642 m^3 at a facility in Rostov-on-Don.

Materials and Methods. Timekeeping during continuous concreting of 2 m thick concrete with a volume of 1642 m^3 using a concrete mixture (CM) with a workability mark of P4 according to GOST 7473-2010 "Concrete Mixtures. Technical Specifications", ABN Zomlion 59X-6RZ (ABN 1) and KCP58ZX170 (ABN 2) with a technical capacity of 180 and 170 m^3/h, respectively, when CM is supplied to CP with a volume of 7 to 16 m^3. The weighted average CP volume is 12.6 m^3, while calculating the parameters in the TR, the value of 14 m^3 was taken. The following parameters were recorded: the time of CM pumping at a distance of up to 50 m $\tau_п$ as the time from the onset of unloading the CP to the end of pumping (according to the signal of the CMT); the total time of the working cycle of the CP, as the time from the onset of unloading the previous CP to that of a subsequent CP; the time of maneuvering the CP at a distance of 15 to 25 m, as the difference between the total operating cycle time and the pumping time of the CM. Concreting took place in dry, clear weather at a temperature of 15-20°C. Overall, 35 measurements were made according to CMT 1 and 34 measurements according to CMT 2. The discharge rate of the CP and the pumping rate of the CM were assumed to be equal and given by the formula:

$$v_{\text{р}} = v_{\text{п}} = \frac{V_{\text{АБС}}}{\tau_{\text{р}}},$$

where $V_{\text{АБС}}$ is the CP volume, m^3.

Research Results. Table 1 shows the results of measuring the duration of technological operations.

Table 1

Results of the measurements of the parameters of the working cycle, min

CMT	Cycle parameters					
	$\tau_{\text{ц}}$		$\tau_{\text{п}}$		$\tau_{\text{м}}$	
	range	average	range	average	range	average
1	6.73–13.06	9.68	4.58–8.4	6.11	2.15–6.08	3.56
2	6.42–17.2	10.11	4.03–10.95	7.09	1.37–8.83	3.22
According to TR		15.0		9.0		6.0

Table 2 shows the results of calculating the CP unloading rate (CM pumping time).

Table 2

Results of the measurements of the CP unloading rate (CM pumping rate), m^3/min

CP volume, m^3	Unloading speed (pumping speed)	
	Range	Average
7	1.615–1.736	1.675
10	1.709–2.182 1.36–1.77*	1.936 1.565*
11	1.688–2.115	2.11
12	1.503–2.278 1.568*	1.917 1.568*
13	1.866–1.984	1.925
14	1.278–2.227 1.49–1.83*	1.959 1.61*
15	1.731–2.284	1.898
16	1.72–2.02	1.834
According to the TR		1.6

Note: * — according to the data in [10]

Strictly speaking, the total time of CP unloading (CM pumping time) does not comply with the law of normal distribution (Fig. 1). For CMT 1, there is a distribution with a positive asymmetry, for CMT 2 there are two peaks, which can also account for apart from to technical influence of technological and organizational aspects, e.g., the concrete workers' productivity of during laying and compaction.

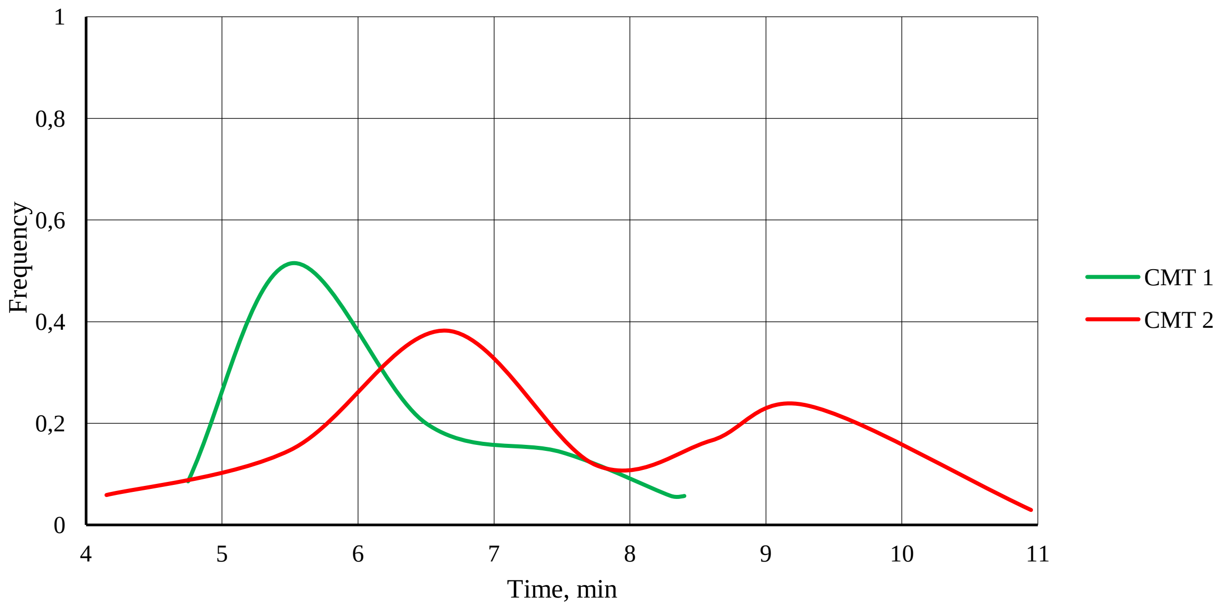

Fig. 1. Frequency of CM pumping

In this case, the CP unloading rate varied in a quite wide range, from 1.28 to 2.28 m^3/min (Fig. 2). The unloading speed was almost independent of the CP volume.

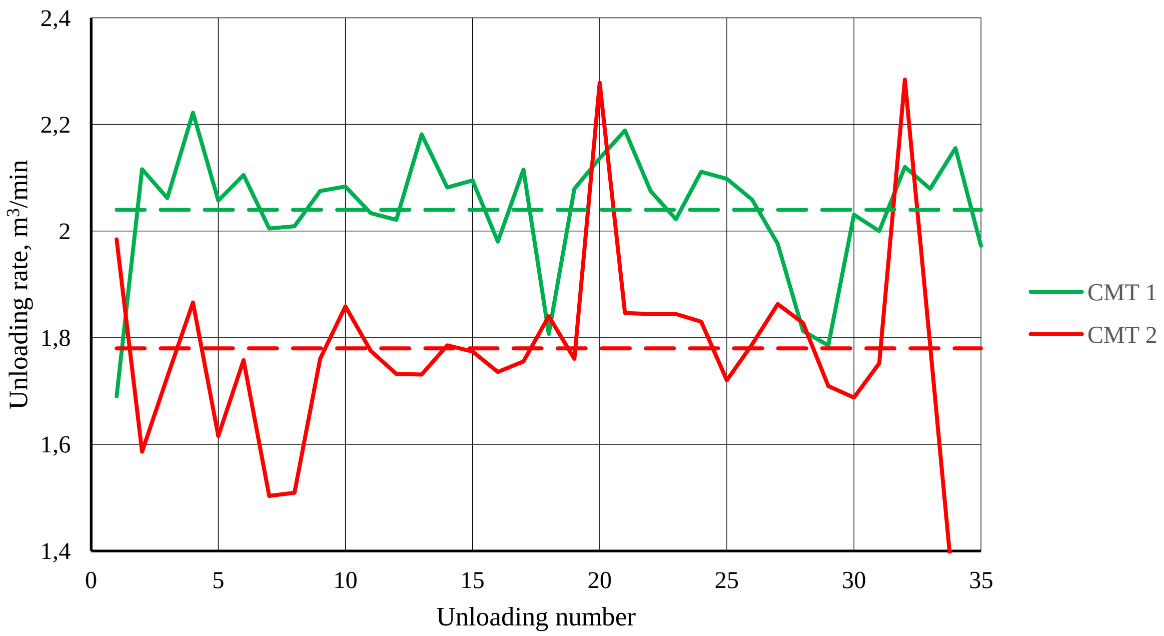

Fig. 2. CP unloading rate

The average pumping rate of CM was 2.04 m^3/min for CMT 1 in the range from 1.69 to 2.222 m^3/min, and 1.78 m^3/min for CMT 2 in the range from 1.278 to 2.284 m^3/min. The pumping rate of CM with a security of 0.95 was 1.815 m^3/min for CMT 1, 1.422 m^3/min for CMT 2 (Fig. 3), the average value for CMT 1 and CMT 2 of 1.62 m^3/min almost coincided with the value of 1.6 m^3/min adopted during the development of the TR. The pumping speed of the CM does not depend on the volume of the CP.

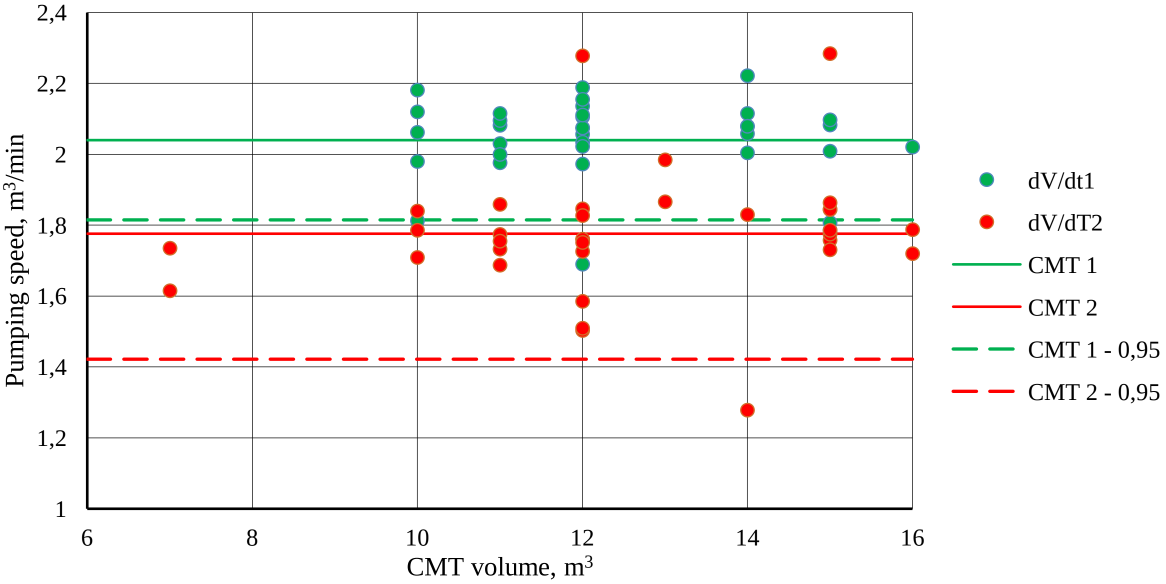

Fig. 3. Dependence of the pumping speed of CM on the volume of CP

In this case, the average coefficient of a transition from the technical to operational productivity for CMT 1 was

$$k = \frac{v_{п,ф}}{v_{п,т}} = \frac{2.04}{3} = 0.68,$$

and for CMT 2 — 0.63.

The coefficient of a transition from the technical to operational productivity should not be associated with the technical performance of the CP. In this case, the expediency of using an CP with a technical capacity of more than 2.83 m^3/min (170 and 180 m^3/h) is justified by the peak values of the pumping capacity coefficient of no more than 0.85.

Fig. 4 shows the dependence of the coefficient β accounting for the organization (the CP utilization factor in time) and the CP volume. The coefficient was calculated using the formula:

$$\beta = \frac{\tau_п}{\tau_ц} = \frac{\tau_ц - \tau_м}{\tau_ц}.$$

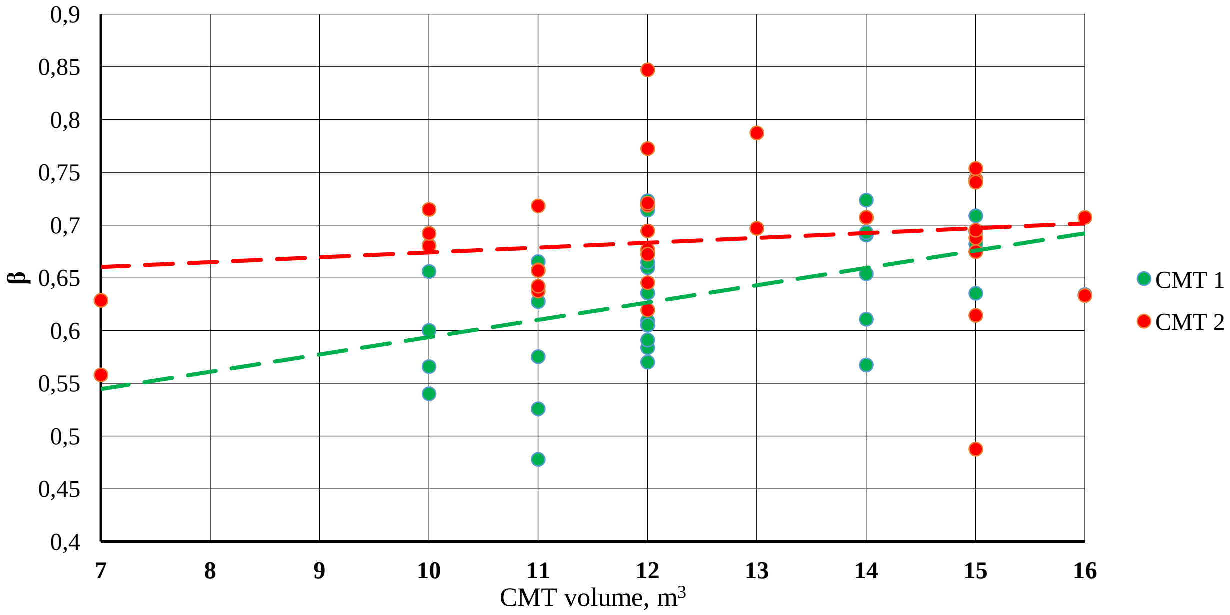

Fig. 4. Dependence of the CP utilization factor on time (organization) from the CP volume

The range of values of the β coefficient for CMT 1 was from 0.478 to 0.744 with an average value of 0.634. The range of values of the β coefficient for CMT 2 was from 0.487 to 0.847 with an average value of 0.685. The average value for both CMTs was 0.659. The dependence of the β coefficient on the CP volume is naturally significantly influenced by the rate of the CP change.

The value of the coefficient of actual efficiency of the CMT during the period of "stabilization" was

$$k_э = k\beta = \frac{0.659 \cdot (0.68 + 0.63)}{2} = 0.43$$

According to data in [10], this value was 0.41.

The maneuvering time of the CP providing CMT 1 and CMT 2 was 3.56 minutes and 3.22 minutes (the best maneuvering conditions), respectively, and with a safety of 0.95 — no more than 5.28 and 5.76 minutes. According to [10], the average maneuvering time of the CP was 3.38 minutes with a range from 2.8 minutes to 4.45 minutes under the best maneuvering conditions compared to those in question. It can be concluded that when the distance from the CMT to the CP waiting area is within 25–50 m, the maneuvering time does not depend greatly on the distance, but is more due to the convenience of the CP maneuvering area and access roads. Cramped working conditions are the key factor for the time parameters of concreting (Fig.5).

Fig. 5. Working site of CMT 2

Fig. 6 shows the dependence of the CMT productivity on concreting hours and data on the average concreting rate

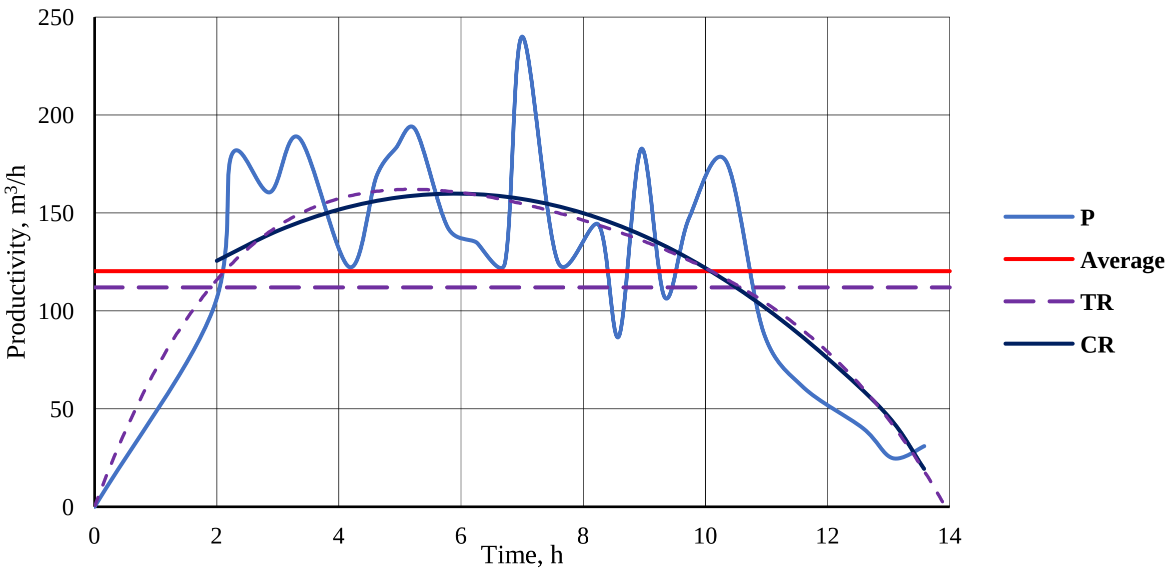

Fig. 6. CMT productivity by concreting hours (P) and average concreting rate (CR)

Technology and organization of construction

The dependence (dotted line in Fig. 6) the average concreting rate, m^3/h, of the time during the process with an approximation confidence index of $R^2 = 0.977$ is described by the equation:

$$CR = 93\tau - 19\tau^2 + 1.6\tau^3 - 0.05\tau^4$$

The deployment and shutdown periods are clear, i.e., about 2 hours each. According to [17], while concreting the bottom plate of the box-shaped foundation of the Lakhta Center tower, a concreting rate of 440 m^3/h was attained about 4 hours later. In this case, during the "stabilization" period, the peak performance of the "maximum/minimum" varied up to 2.5 times. The total concreting time of the FS was actually 13.58 hours, the estimated time for the CR was 15 hours. Average productivity: actual one — 122 m^3/h; estimated by the TR — 112 m^3/h. The average performance factor of the CP taking into account the deployment and shutdown periods was ≈ 0.35. According to [10], this value is specified in the GEEN.

With an average technical capacity of 175 m^3/h for both CPs, the actual average productivity over the concreting period was ≈ 61 m^3/h. According to [10], during the "stabilization" period, with an average CMT productivity coefficient of 0.41 and the technical productivity of 120 m^3/h, the actual average productivity during the concreting period was ≈ 49 m^3/h. According to [17], while concreting the bottom plate of the box-shaped foundation of the tower of the Lakhta Center complex with a volume of about 20.3 thousand tons. m^3 in 49 hours, the average concreting intensity was ≈ 414 m^3/h or ≈ 23 m^3/h per CMT (the CMT parameters are not specified).

Discussion and Conclusion. New data has been obtained for calculating the time parameters of concreting massive structures by means of technological equipment (concrete pumps) with the technical characteristics not available in the regulatory framework, particularly, the GEEN. As a result of the timekeeping during concreting of a thickness of 2m and a volume of 1642 m^3, the values of the unloading rate and the pumping rate of CM were obtained: the average for CMT 1 was 2.04 m^3/min in the range from 1.69 to 2.222 m^3/min, for CMT 2 — 1.78 m^3/min in the range from 1.278 to 2.284 m^3/min. With a security of 0.95, the pumping rate of CM was 1.815 m^3/min for CMT 1, and 1.422 m^3/min for CMT 2. The average coefficient of a transition from the technical to operational productivity was ≈ 0.68 for CMT 1 and ≈ 0.63 for CMT 2. The expediency of using a CP with a technical capacity of more than 2.83 m^3/min or 170 m^3/h is due to the need for normal operation of a machine, even at peak loads. The CP utilization factor in time ranged from 0.478 to 0.847, with an average value of 0.66 for both CPs. With an average technical capacity of 175 m^3/h of both CPs, the actual average productivity over the concreting period was ≈ 61 m^3/h.

The maneuvering time of the CP at a distance of up to 25 m averaged 3.22...3.56 minutes depending on the convenience of the sites, with a security of 0.95 — no more than 5.28 ...5.76 minutes. When the distance from the CMT to the CP waiting area is within 25–50 m, the maneuvering time does not depend greatly on the distance, it is due to the convenience of the CP maneuvering area and access roads.

The research results make it possible to rationally assign time parameters for concreting massive foundation slabs and can be employed in the TR development.

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JI Koryanova: detailed planning, organization and implementation of the research, analysis of the research results, preparation of the manuscript, formulation of the conclusions.

VV Shut: operational planning and implementation of the research, processing of the research results, analysis of the research results.

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CONSTRUCTION MECHANICS
СТРОИТЕЛЬНАЯ МЕХАНИКА

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Original Empirical Research

Numerical Analysis of Stress-Strain Conditions of a Reinforced Concrete Bridge Section

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Abstract

Introduction. The Russian Federation has adopted a long-term program for the large-scale construction of highways, which will require the construction of a large number of bridges of small, medium and large spans. International experience shows that it is advisable to build road bridges from prestressed reinforced concrete. Moreover, the most effective ones are the span bridge sections of a box-shaped cross-section that are different from girder structures by better aerodynamics, lower labor costs during construction and more attractive external aesthetics. In the literature on numerical analysis of the stress-strain of monolithic reinforced concrete structures, very little information is provided on calculating span bridge structures taking into account concrete creep. The aim of the study was to develop a technique for finite element modeling of long-term deformation of a box section span using an authorized software package. The data from the computational experiments were verified using the ANSYS Mechanical software package.

Materials and Methods. The finite element method in the form of a displacement method in combination with the theory of linear viscoelasticity is employed as a mathematical tool for modeling prolonged deformation of the investigated reinforced concrete structure. In order to formalize concrete creep, S.V. Aleksandrovsky's elastic creeping body model was used. The computational process of numerical integration of the resulting operator-matrix equation is based on the principle of superimposition of effects and the use of the trapezoid formula. The computational experiments were performed on the Microsoft Visual Studio platform and the Intel Parallel Studio XE compiler with the built-in Intel Visual Fortran Composer XE text editor. In order to visualize the simulation results in the form of pictures of the distribution of displacement and stress fields, the descriptive graphics of the Matlab system are employed.

Research Results. A program has been developed and verified for the finite element calculation of reinforced concrete beam structures in a three-dimensional formulation using a discrete reinforcement scheme, according to which the reinforcing frame is modeled by means of two-node beams, and the concrete array is modeled by means of volumetric multilinear finite elements. It is found that for the considered typical box-shaped bridge sections, the adopted pre-voltage scheme is ineffective as it fails to provide the required bending.

Discussion and Conclusion. The results of the calculations of the box section in a linearly elastic formulation obtained using the developed software package and the ANSYS Mechanical software package are compared. A satisfactory coincidence of displacement and stress values at the investigated points has been identified. The stress-strain state of the box section at the stage of prestressing and subsequent loading is investigated. The conclusion is made on the expediency of scientific support at the design stage of such bridge sections in order to increase their load-bearing capacity.

Keywords: finite element method, concrete creep, prestressing, reinforced concrete bridge section of a box section

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Оригинальное эмпирическое исследование

Численный анализ напряженно-деформированного состояния секции железобетонного моста

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Аннотация
Введение. В Российской Федерации принята долгосрочная программа по масштабному строительству автомобильных дорог, что потребует возведения большого количества мостов малых, средних и больших пролетов. Международный опыт говорит, что автодорожные мосты целесообразно строить из преднапряженного железобетона. Причем наиболее эффективными являются пролетные мостовые секции коробчатого поперечного сечения, отличающиеся от балочных конструкций лучшей аэродинамикой, меньшими трудозатратами при возведении и более привлекательной внешней эстетикой. В литературе, посвященной численному анализу напряженно-деформированного состояния монолитных железобетонных конструкций, приведено крайне мало сведений о расчете пролетных мостовых строений с учетом ползучести бетона. Целью исследования являлась разработка методики конечно-элементного моделирования длительного деформирования пролетной секции коробчатого сечения, с помощью авторизированного программного комплекса. Данные вычислительных экспериментов верифицированы с использованием программного комплекса ANSYS Mechanical.
Материалы и методы. В качестве математического аппарата для моделирования процесса длительного деформирования исследуемой железобетонной конструкции применен метод конечных элементов в форме метода перемещений в сочетании с теорией линейной вязкоупругости. Для формализации процесса ползучести бетона использована модель упруго-ползучего тела С.В. Александровского. Вычислительный процесс численного интегрирования результирующего операторно-матричного уравнения базируется на принципе наложения воздействий и использовании формулы трапеций. Вычислительные эксперименты выполнены на платформе Microsoft Visual Studio и компиляторе Intel Parallel Studio XE с встроенным текстовым редактором Intel Visual Fortran Composer XE. Для визуализации результатов моделирования в виде картин распределения полей перемещений и напряжений применена дескрипторная графика системы Matlab.
Результаты исследования. Разработана и верифицирована программа для конечно-элементного расчета железобетонных балочных конструкций в трехмерной постановке с использованием дискретной схемы армирования, согласно которой армирующий каркас моделируется двухузловыми балочными, а массив бетона — объемными полилинейными конечными элементами. Установлено, что для рассматриваемой типовой мостовой секции коробчатого сечения принятая схема предварительного напряжения малоэффективна, так как не обеспечивает требуемого выгиба.
Обсуждение и заключение. Выполнено сравнение результатов расчетов коробчатой секции в линейно упругой постановке, полученных с помощью разработанного пакета программ и программного комплекса ANSYS Mechanical. Установлено удовлетворительное совпадение значений перемещений и напряжений в исследуемых точках. Исследовано напряженно-деформированное состояние коробчатой секции на этапе создания предварительного напряжения и последующего нагружения. Сделан вывод о целесообразности научного сопровождения на этапе проектирования подобных мостовых секций с целью повышения их несущей способности.

Ключевые слова: метод конечных элементов, ползучесть бетона, предварительное напряжение, железобетонная мостовая секция коробчатого сечения

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Introduction. At present the most extensive field of application of prestressed reinforced concrete is bridge construction. At the same time, in the practice of domestic design of span sections of overpasses and viaducts, steel structures are preferred. This is primarily due to the problems associated with the "tight" deadlines for construction of road junctions, convenience of transporting metal structures and high costs associated with manufacturing large-span monolithic prestressed reinforced concrete sections and their subsequent installation. Nevertheless, the experience of Western as well as South Asian countries indicates the expediency of making use of prestressed reinforced concrete as a building material for span sections of automobile bridges. On top of that, in order to address the above problems, reinforced concrete sections are traditionally manufactured directly on site followed by sliding them onto bridge supports.

It is well known that concrete is subject to creep deformation, which greatly depends on its "age", size of a structure, nature and sequence of application (removal) of an external load as well as the temperature and humidity state of the environment. It has been experimentally found that creep considerably affects the redistribution of internal forces in concrete and reinforced concrete structures [1, 2]. Additionally, the displacements caused by creep deformation frequently exceed those caused by applying a load at the initial moment of time, as was the case with the reinforced concrete bridge in the island state of Palau in 1996 [3]. In spite of a great number of works dedicated to numerical calculations of reinforced concrete structures taking concrete creep into account, there is no information available in literature on finite element modeling of long-term deformation of bridge sections within the framework of the theory of linear viscoelasticity in combination with the principle of superimposition of impacts. In [4] in particular, an analysis of the long-term deformation of a reinforced concrete beam of a box-shaped cable-stayed bridge was performed using the MIDAS/Civil software package that makes use of a technique based on lowering the current modulus of elasticity of concrete using a creep coefficient.

Hence the problem of numerical analysis of long-term deformation of reinforced concrete bridge sections has been insufficiently investigated. Based on this, the aim of the study can be formulated as follows: to develop a technique for finite element modeling of reinforced concrete bridge sections of a box section, taking linear concrete creep into consideration.

Materials and Methods. The developed mechanical and mathematical model of concrete creep is based on the modern theory of elastic-creeping body [5, 6]. For a finite element analysis of monolithic reinforced concrete structures, taking concrete creep into consideration, the relationship between stresses and deformations is represented in a matrix operator form [6]:

$$\{\sigma(t)\} = [E(t)](1 - R)\{\varepsilon(t)\}, \quad (1)$$

where $\{\sigma(t)\}$, $\{\varepsilon(t)\}$ are the vectors that are columns of stresses and deformations corresponding to a moment in time t; $[E(t)]$ is the elasticity matrix (in the general case, the dimension 6 × 6); $R\varepsilon_{ij} = \int_{\tau_1}^{t} R(t,\tau)\varepsilon_{ij}(\tau)d\tau,\ i,j = 1{,}3$ is a linear integral operator that establishes a correspondence between current deformations ε_{ij} and the history of long-term deformation $\varepsilon_{ij}(\tau)$.

The so-called hereditary function is introduced in integrative terms $R(t,\tau)$ [2].

In the suggested scheme for numerical integration of the resulting integral operator equation (1), the principle of superposition is applied, and the entire loading history is stored in computer memory in the form of arrays of nodal displacements. The Polygon educational and research program was written and debugged based on the developed mathematical software in the Intel Fortran programming language [5, 6].

Research Results. It is noted in [6] that for large spans of reinforced concrete bridges, monolithic sections of a box-shaped cross-section are commonly used. For superstructures with a roadway up to 20 m wide, a single box-shaped cross section with developed consoles is employed.

Let us calculate the stress-strain of a prestressed section of a box-type reinforced concrete bridge span. As a prototype, let us take the data from the example from [7, 8]. The shape and dimensions of the section section are shown in Fig. 1. The span of the bridge beam is 20 m. We assume that the section is loaded with its own weight, the force from the pretension of the cables and the evenly distributed pressure on the roadway.

Given the symmetry of geometry and loading, we will consider the ½ of the section. The corresponding finite element model is shown in Fig. 2. In order to discretize the investigated area of the section, volumetric eight-node finite elements (FE) were employed.

Support pads are located along the edges of the section with 3.6 m in width between them.

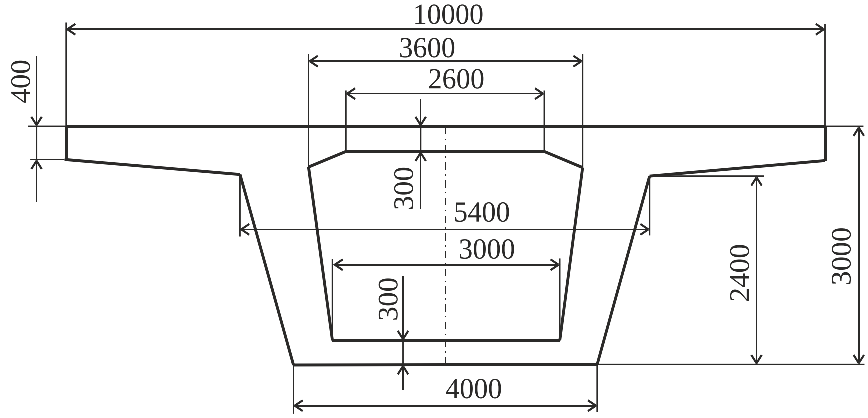

Fig. 1. Cross section of a box-shaped superstructure [7, 8]

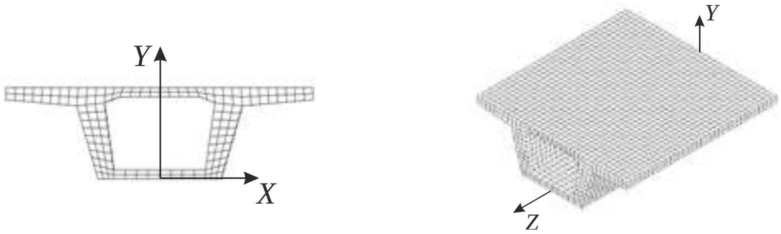

Fig. 2. Finite element model of ½ of a superstructure section

The diagram of the trajectories of "active" reinforcement along the Z axis is shown in Fig. 3 (the dimensions are in meters). The method of tension on concrete using cable reinforcement with no adhesion to concrete is considered. The pre-tensioning force of the cables is 245.6 kN.

First, a linearly elastic analysis of the stress-strain state of the span section will be performed. Fig. 4 and 5 show the distribution of vertical displacements u_y and longitudinal stresses напряжений σ_z in the ½ part of the section due to the action of its own weight.

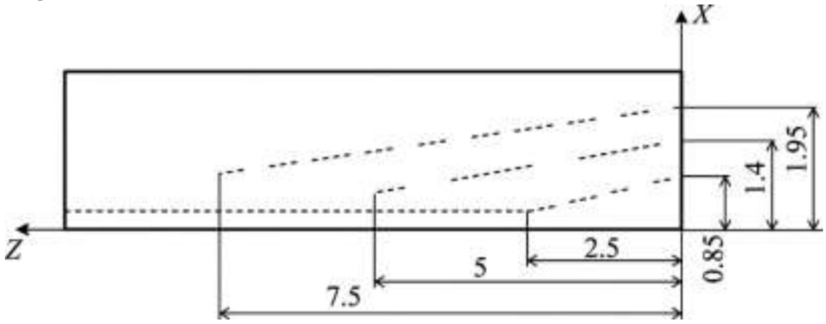

Fig. 3. Scheme of "active" reinforcement (a trapezoidal one)

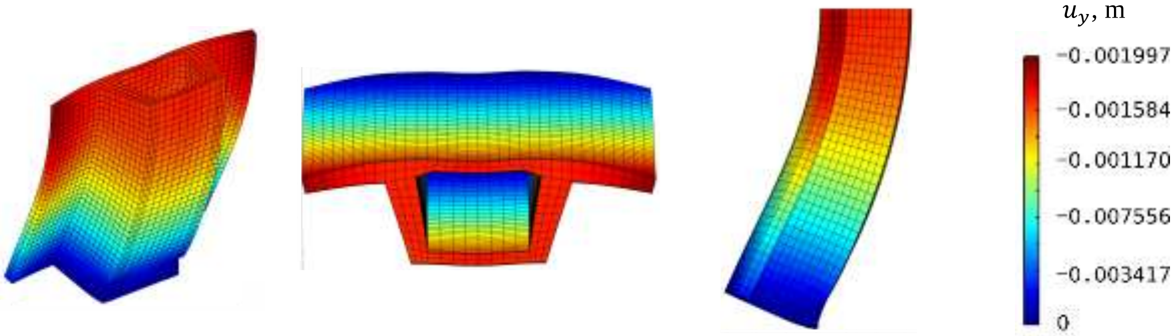

Fig. 4. Picture of the distribution of movements u_y in the ½ of a section from the action of its own weight (Polygon)

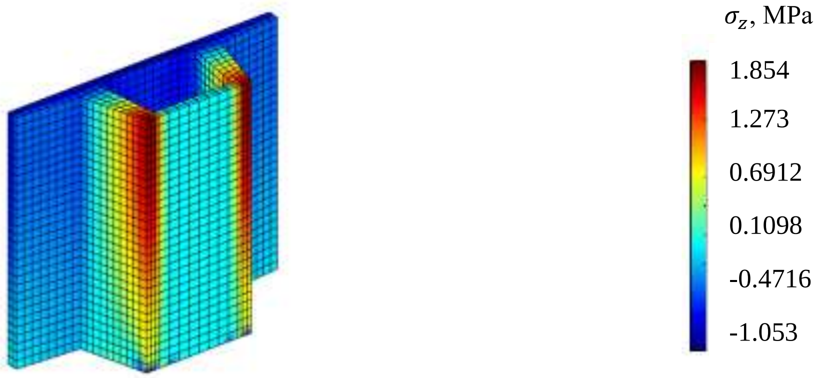

Fig. 5. Picture of the distribution of movements σ_z in the ½ part of the section from the action of its own weight (Polygon)

The values of the maximum deflection and maximum tensile stress in this case were

$$|u_{ymax}| = 0{,}001997 \text{ m}; \ \sigma_{zmax}^{+} = 1{,}854 \text{ MPa}.$$

In order to verify the used finite element model, considering the gravitational effect, a similar section calculation was performed by means of the ANSYS complex. Visualization of the fields u_y and σ_z obtained based on eight-node SOLID185 type CE is shown in Fig. 6 and 7.

Comparing the results shown in Fig. 4 and 6, a fairly good quantitative coincidence of the maximum deflections along the edges of the ½ part of the span section in section Z = 5 m is found. are qualitatively different, although the values in both calculations are quite close: The distribution patterns σ_z are qualitatively different, although the values σ^+_{zmax} in both calculations are quite close::

$$\sigma^+_{zmax} = 1{,}82 \text{ MPa (ANSYS)}; \ \sigma^+_{zmax} = 1{,}854 \text{ MPa (Polygon)}.$$

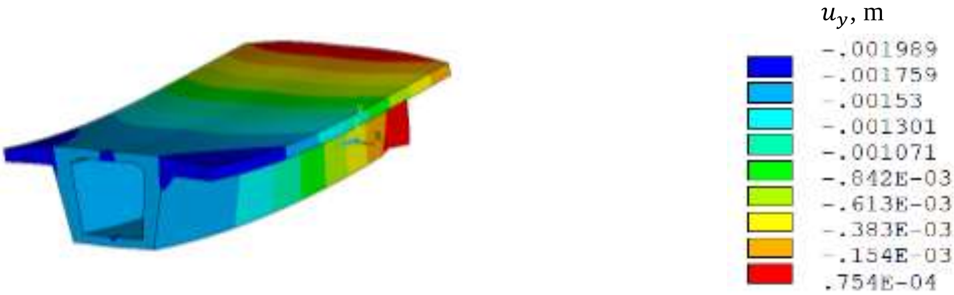

Fig. 6. Picture of the distribution of movements u_y in ½ of a section from the action of its own weight (ANSYS)

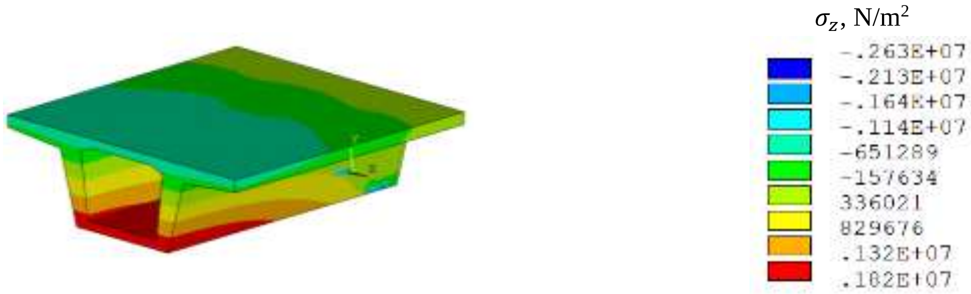

Fig. 7. The pattern of displacement distributionσ_z in ½ of a section from the action of its own weight

In order to demonstrate the capacities of the Polygon complex, Fig. 8 shows a model with the introduced background reinforcement of the carriageway of the twenty-meter section under consideration. The cross-sectional area of the reinforcing bars is 0.001 m^2, the modulus of elasticity is $1.96 \cdot 10^5$ MPa. The locations of the reinforcing rods modeled by spatial two-node girders in the section section are marked with dots in Fig. 8.

The results obtained using the Polygon complex were compared with the data from the ANSYS complex. In both cases, the load was only the dead weight of a section. The values of the maximum deflections were:

$$|u_{ymax}| = 0.001977 \text{ m (ANSYS)}; \ |u_{ymax}| = 0.002009 \text{ m (Polygon)}.$$

The result of calculating the deflection of a section with no reinforcement is $|u_{ymax}|$ = 0.001997 m. Hence it can be concluded that the inclusion of 14 rod FEs with a diameter of 35.6 mm in the design model has barely any effect on the bending stiffness of the span section.

Fig. 8. Finite-element model with background reinforcement of the roadway

It should be noted that the "direct" solver of the ANSYS complex calls for introduction of additional connections to the nodes of the core elements that prohibit rotations relative to the Y and Z axes. The solver of the Polygon complex is free from such a restriction. Visualizations of the fields $\sigma_z$obtained using the Polygon and ANSYS complexes are shown in Fig. 9 and 10.

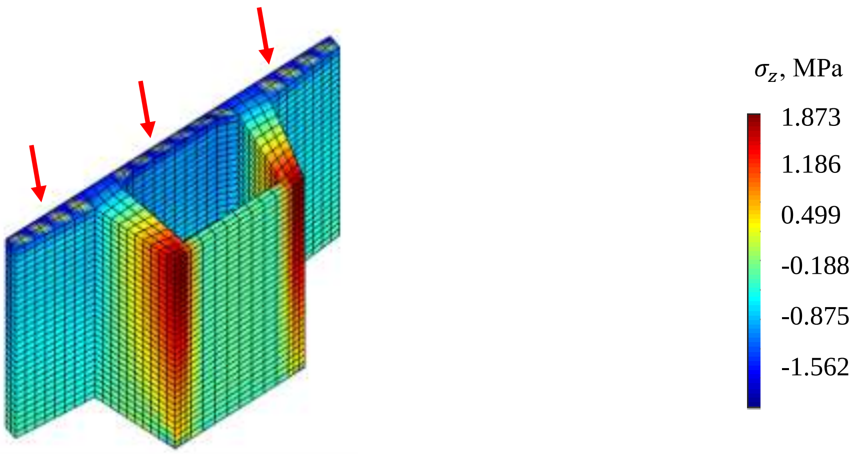

Fig. 9. Pattern of displacement distribution σ_z in ½ of a section with reinforcement from the action of its own weight (Polygon)

Fig. 10. Pattern of displacement distribution σ_z in ½ of a section with reinforcement from the action of its own weight (Polygon)

While comparing the data in the figures, it is found that the maximum tensile stresses calculated using different complexes practically coincide: 1.873 MPa (Polygon); 1.81 MPa (ANSYS).

It is also noteworthy that the stress concentration zones σ_z at the locations of the reinforcing bars (marked with arrows). The postprocessor of the ANSYS complex does not provide such information.

In conclusion, a calculation of the span section during prolonged deformation will be performed. As a loading scenario, the following chronology of events is used:

a) at a moment in time t = 28 days. self-loading and pre-tensioning of cables with a force of 245.6 kN;

b) at a moment in time t = 72 days. A uniformly distributed load q = 20.000 N/m^2 (2 t/m^2) is applied to the roadway surface.

The parameters of the elastic-creeping body model are taken from the monograph [2].

The graphs of the displacements $u_y(t)$ at points 1 and 2 of a section Z = 10 m of the span section are shown in Fig. 11.

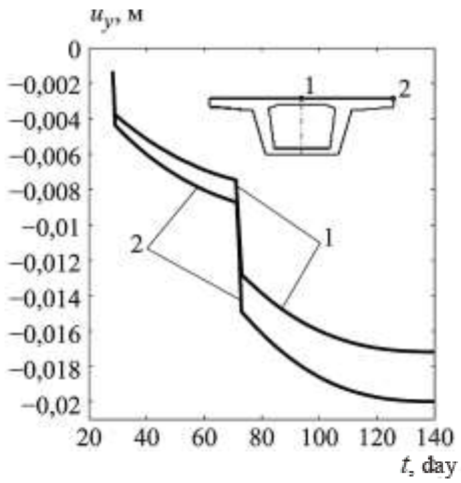

Fig. 11. Creep curves at points 1 and 2 (Polygon)

The value of the maximum tensile stress at a specified loading law at the time of observation t = 140 days was 16.7 MPa, which is significantly beyond the strength limit of concrete.

Discussion and Conclusion. Based on the obtained results of finite element modeling, it can be noted that the adopted trapezoidal layout scheme of the prestressed rope reinforcement has barely any effect on the load-bearing capacity of the bridge section as it does not provide the necessary bending. Also, the introduction of reinforcing rods with a diameter of 35.6 mm into the finite element model in the area of the roadway has barely any effect on the bending stiffness of the bridge section. In order to increase the operational reliability of prestressed concrete bridge sections, it is recommended that a modern viscoelastic concrete model implemented in the authorized Polygon software package at the design stage apart from the existing computing technologies is employed.

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LIFE CYCLE MANAGEMENT OF CONSTRUCTION FACILITIES

УПРАВЛЕНИЕ ЖИЗНЕННЫМ ЦИКЛОМ ОБЪЕКТОВ СТРОИТЕЛЬСТВА

УДК 621.928:728.1

Original Empirical Research

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Sustainable Lifecycle Management: From Wind Turbine Blades to Modern Playground Complexes

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Abstract

Introduction. In modern conditions of the construction industry development, the concept of the life cycle of construction facilities encompasses not only the design and operation stages, but also the disposal of structural elements, particularly the components of wind turbines (hereinafter referred to as WT) with a service life of 20–25 years. Conventional methods of disposal of used elements of wind turbines display a low environmental efficiency. In the context of the circular economy, innovative solutions for integrating used WT components into construction practice are to be developed.

The aim of the study was to scientifically substantiate a possibility of using recycled wind turbine blades in the construction of children's playground complexes.

Materials and Methods. The study is based on a methodology for analyzing the life cycle of wind turbine blades, taking the specific features of their processing in the construction industry into consideration. The following methods were employed: a systematic analysis of the characteristics of wind turbine blades, a statistical assessment of their life cycle, a comparative analysis of recycling technologies and an assessment of the safety of structures made from recycled materials.

Research Results. As a result of the analysis and research conducted, a technology for processing blades into structural elements of a children's playground complex has been developed.

Discussion and Conclusion. The results confirm the fundamental possibility and practical expediency of employing recycled wind turbine blades as construction facilities in order to design safe and durable children's playground complexes. The developed technological solutions that take into consideration the full life cycle of materials – from operation as part of wind turbines to secondary use in building structures – enable transformation of the environmental problem of recycling into a resource for urban infrastructure development.

Keywords: life cycle, utilization of wind power plants, construction of playgrounds, secondary use, construction site

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Оригинальное эмпирическое исследование

Устойчивое управление жизненным циклом: от лопастей ветрогенераторов до современных игровых комплексов

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Аннотация

Введение. В современных условиях развития строительной отрасли концепция жизненного цикла строительных объектов охватывает не только этапы проектирования и эксплуатации, но и вопросы утилизации конструктивных элементов, в частности компонентов ветроэнергетических установок (далее — ВЭУ) со сроком службы

20–25 лет. Традиционные методы утилизации отработавших элементов ветрогенераторов демонстрируют низкую экологическую эффективность. В контексте циркулярной экономики возникает необходимость разработки инновационных решений для интеграции отработанных компонентов ВЭУ в строительную практику.
Целью исследования являлось научное обоснование возможности применения утилизированных лопастей ВЭУ в конструкциях детских игровых комплексов.
Материалы и методы. Исследование основано на методологии анализа жизненного цикла лопастей ветряных турбин с учетом специфики их обработки в строительной отрасли. В работе использовались следующие методы: систематический анализ характеристик лопастей ветряных турбин, статистическая оценка их жизненного цикла, сравнительный анализ технологий вторичной переработки и оценка безопасности конструкций, изготовленных из переработанных материалов.
Результаты исследования. В результате проведенного анализа и исследования разработана технология переработки лопастей в конструктивные элементы детского игрового комплекса.
Обсуждение и заключение. Полученные результаты подтверждают принципиальную возможность и практическую целесообразность использования утилизированных лопастей ВЭУ в качестве объектов строительства для создания безопасных и долговечных детских игровых комплексов. Разработанные технологические решения, учитывающие полный жизненный цикл материалов — от эксплуатации в составе ветроэнергетических установок до вторичного применения в строительных конструкциях, — позволяют трансформировать экологическую проблему утилизации в ресурс для развития городской инфраструктуры.

Ключевые слова: жизненный цикл, утилизация ветроэнергетических установок, строительство детских площадок, вторичное использование, строительный объект

Для цитирования. Самарская Н.С. Устойчивое управление жизненным циклом: от лопастей ветрогенераторов до современных игровых комплексов. *Современные тенденции в строительстве, градостроительстве и планировке территорий*. 2026;5(1):96–103. https://doi.org/10.23947/2949-1835-2026-5-1-96-103

Introduction. In modern conditions of the construction industry development, there is a tendency towards the integrated implementation of the concept of the life cycle of construction facilities including not only the design and operation stages, but also the disposal of structural elements. The issue of life cycle management of the components of renewable energy sources, particularly wind turbines (hereinafter referred to as WT), with a physical service life of up to 20–25 years, is gaining particular relevance.

Conventional approaches to disposal of used elements of wind turbines, such as landfill disposal or heat treatment, display a low environmental efficiency and contradict the principles of sustainable development of the construction industry. In the context of the circular economy, innovative solutions that integrate used WT components into construction practice are to be developed.

The concept of a project life cycle (PLC) serves as a methodological foundation for forming effective strategies for construction waste management. Special attention is paid to searching for sustainable ways of recycling composite materials of wind turbine blades with high strength characteristics and resistance to external impacts.

The aim of the study is a comprehensive scientific substantiation of a possibility of using recycled WT elements, particularly blades, as structural elements of children's playground complexes. This research area corresponds to the current trends in the construction industry development and contributes to implementing the principles of cyclical economics in urban development. The foundation for developing the scientific idea was the architectural design of a playground in Rotterdam (Fig. 1).

Unlike individual architectural projects (Fig. 1), the work integrates a comprehensive management system for the entire life cycle – from the design and construction of a wind turbine to the criteria for selecting blades at the stage of dismantling WT and assessing the economic feasibility of a project for av secondary use of WT structural elements.

Attaining this goal entails addressing the following: analyzing the physical and mechanical characteristics of WT blade materials, examining the existing methods of their disposal, developing technological solutions for processing and modifying composite materials, as well as substantiating a possibility of using them in playground structures in view of the safety and durability requirements.

Fig. 1. Architectural solutions for a playground in Rotterdam designed by means of WT blades

Materials and Methods. The study is based on an integrated approach to analyzing the life cycle of WT blades. The methodological foundation of the research is the concept of life cycle management of construction facilities adjusted to specific features of WT processing components. To this end, the following research methods were applied:

– a comprehensive analysis of design features and physical and mechanical characteristics of WT blades;
– a statistical analysis of data on the service life and utilization volumes of WT blades;
– a comparative analysis of the existing technologies for processing composite materials;
– expert evaluations of the safety and durability of structures made from recycled materials.

Research Results. The analysis of data from numerous studies of composite materials employed in the production of WT blades (E-glass fiberglass and EPIKOTE™ epoxy resins) has enabled us to conclude that they have stellar performance characteristics [1, 2]. According to a study conducted by a team of authors [3], the bending strength of these materials is 350–500 MPa, which is actually comparable with the characteristics of individual grades of structural steel.

The data on the resistance of materials to atmospheric influences are of particular interest. As shown by the climatic tests described in a study by Ermakov B.S., et al. [4], following 1000 cycles of accelerated testing, a decrease in strength characteristics was not over 5–7%. These results have been confirmed by more recent studies [5].

The conclusions drawn by a group of researchers [6] regarding the preservation of up to 80% of the original mechanical properties of blades following 25 years of operation are critical. This lays a science-based foundation for their secondary use as load-bearing structures.

As regards the processing parameters of materials, the optimal cutting temperature mode to prevents delamination of a composite structure was 60–80°C, as found through the course of the experiments described in [7]. The requirements for edge grinding (rounding radius ≥ 3 mm), ensuring safety for children, are justified in by a team of authors in [8] based on a series of mechanical tests.

The data on the physical and mechanical characteristics of composite materials of wind turbine blades are paramount for choosing the optimal methods of their disposal. [3, 5] show that preserving 80% of the original properties following 25 years of operation considerably expands capacities of recycling structural elements. However, the efficiency of different processing methods varies greatly depending on the safety of a material and the required quality of an end product.

Based on the systematization of the latest research [6, 7, 9], a classification of wind turbine blade processing methods has been developed including three major technological directions (Fig. 2).

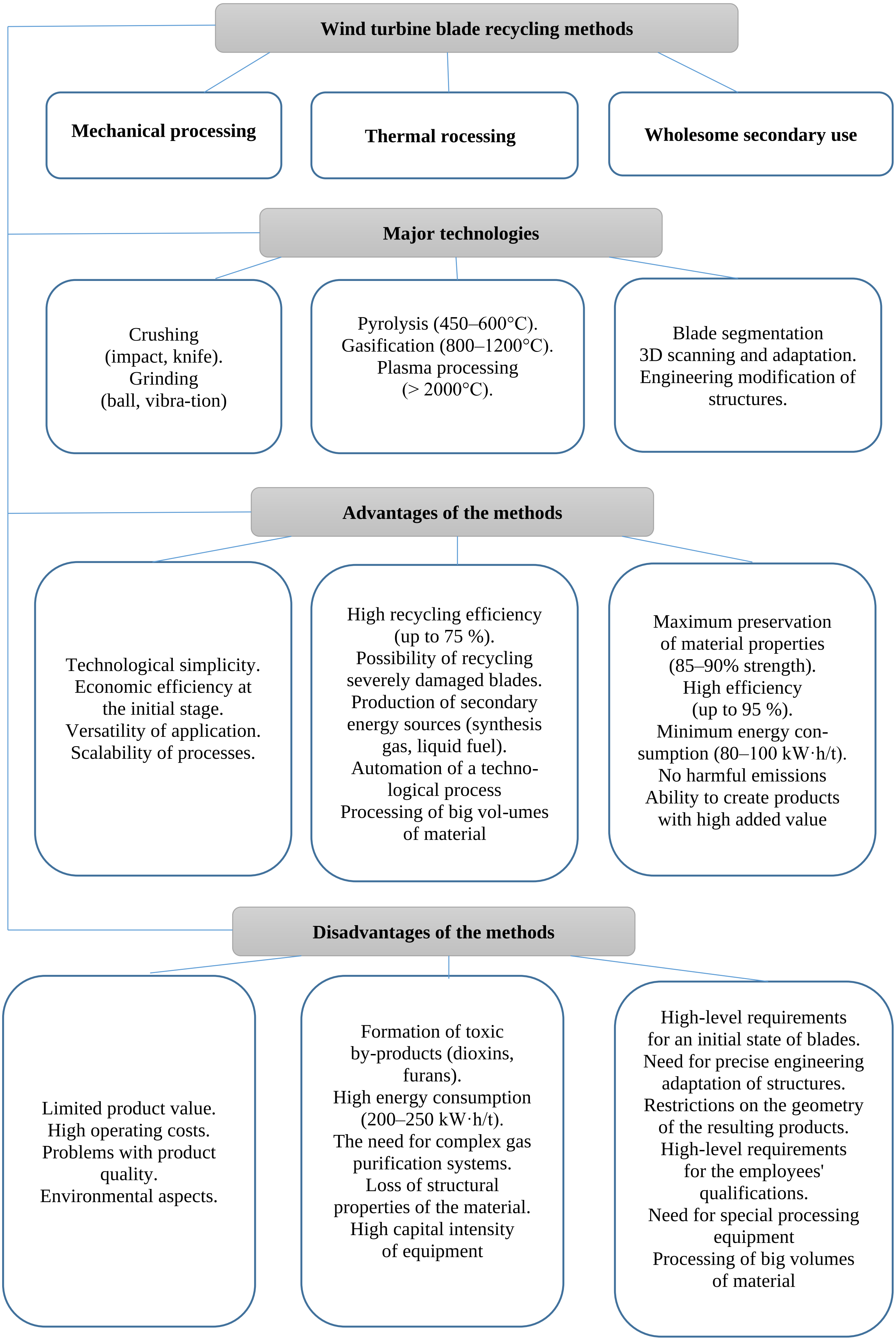

Fig. 2. Classification of the recycling methods of wind turbine blades

The economic and environmental analysis performed (Table 1) has identified significant benefits of wholesome processing.

Table 1

Comparative indicators of the disposal methods

Criterion	Mechanical crushing	Pyrolysis	Wholesome processing
Unit costs, roubles/t	9600–12000	16000–20000	6400–8000
Carbon footprint, kg CO_2-e/t	180–220	350–400	70–90
Payback period, years	5–7	8–10	3–4
Toxicity level	Low	High	No

Calculations show that wholesome processing provides:

– a 60% reduction in carbon footprint compared to pyrolysis;

– a reduction of energy consumption by 35–40% compared to mechanical crushing;

– a possibility of obtaining value-added products.

The major technological advantages of integral reuse have been confirmed by [10, 11]:

– preserving the anisotropic properties of a material;

– minimum technological waste (< 5%);

– a possibility of creating a closed cycle of composite use.

The prospects of the area are made particularly obvious while analyzing the life cycle of processed products where solid structures display a 2.5–3 times better environmental performance compared to the alternative recycling methods [12].

Based on a comprehensive analysis of the existing recycling methods, an innovative three-stage technological chain for preparing blades for reuse in children's playground complexes has been developed including 4 main stages: dismantling, preparation, processing and quality control.

The specialized technology for dismantling wind turbine blades involves the following processes:

– laser scanning of the structure to identify the optimal cutting points;

– use of water-cooled diamond rope saws;

– use of vacuum lifts with a lifting capacity of up to 20 tons;

– transportation in shock-absorbing containers fitted with a climate control system, vibration sensors, GPS monitoring.

At the same time, it is critical to minimize mechanical damage to dismantled structures and comply with the safety standards while working with large-sized objects. In order to assess a possibility of reuse of wind turbine blades in playgrounds and other facilities, a system of 12 major criteria grouped into 4 categories (Table 2) is set forth.

The next preparation stage is a multi-stage system of technological processes and includes:

– mechanical cleansing: abrasive blasting (pressure 2–3 atm);

– chemical treatment: biodegradable detergents (*pH* 7.5–8.5), neutral solvents;

– antiseptic treatment.

Cleansing quality control can be performed by means of electron microscopy, infrared spectroscopy, and contact angulometry.

The stage of mechanical preparation of the elements of the dismantled blades is a critical stage of the suggested technology to be used in construction of a children's playground complex. The geometry is adapted to new tasks by means of precision cutting that removes damaged areas, optimizes dimensions for target structures, and creates mounting joints.

Also, the stage of mechanical treatment makes it possible to ensure the safety of a future structure. This typically involves treatment of the edges (rounding, polishing, applying protective chamfers) and creating technological holes for fastening systems. E.g., for a children's slide, a central segment (6 m long) must be cut out of a blade, there must be side stops up to 30 cm high as well as a controlled sliding surface with a roughness of 3.2–6.3 microns for a safe descent, and attachment zones to the foundation must be treated.

Table 2

Criteria for the suitability of wind turbine blades for secondary use

Category	Parameter	Requirements	Assessment method
1. Structural and mechanical characteristics	Residual flexural strength	≥ 60 % of the original one (~ 300 MPa)	Three-point bending tests by ASTM D790
	No critical damage	Cracks > 5 mm, delaminations > 10 % of the area	Ultrasonic flaw detection
	State of a composite matrix	Humidity < 1 %, no resin degradation	Thermogravimetric analysis (TGA)
2.Geometric and operational parameters	Preserved segment length	≥ 3 m for load- bearing elements	Visual inspection
	Curvature radius	1–5 m for arch structures	Geometric measurements
	Wall thickness	Homogeneity ± 15 % of the original	Measurement tools
3.Environmental and chemical safety	No toxic emissions	Volatile organic compounds < 0.1 mg/m³	Gas chromatography
	UV resistance	Strength loss < 5 % after 1000 h of a test	UV-testing
	Corrosion resistance of metal inclusions	No corrosion is allowed	Visual inspection, corrosion tests
4. Economic and technological criteria	Recycling energy costs	< 100 kW·h/t	Energy analysis
	Compatibility with the joining methods	Adhesive / mechanical fasteners	Technological assessment
	Damage localization	≤ 30 % of the area is in need of repair	Visual inspection, measurements

Hence the developed technological chain allows one:

– to ensure a full cycle of wind turbine blades recycling;
– to preserve up to 95% of the original material properties;
– to create safe and durable gaming complexes;
– to reduce recycling costs by 40–45%.

Discussion and Conclusion. The study has confirmed that the recycled blades of wind turbines can be successfully employed in construction of children's playground complexes providing an appropriate degree of safety and durability. The resulting technological solutions take a full life cycle of materials into consideration: from their operation as part of wind turbines to the secondary use in building structures. This approach allows not only to solve the environmental problem of composite waste disposal, but also to create an additional resource for urban infrastructure development.

The major aspect is analyzing a life cycle of such structures, which displays their economic and environmental efficiency at all of the stages: from design and installation to operation and possible recycling. Introducing this technology into playground construction contributes to a sustainable urban environment that adheres to the principles of the circular economy.

For further improvement of the method of recycling wind turbine blades, the following areas should be addressed:

– automation of dismantling processes — introduction of modern technologies for disassembling and sorting blades in order to reduce labour costs and increase efficiency;

– robotic processing complexes — use of automated systems for cutting, grinding and preparing composite segments in order to increase the operating accuracy and safety;

– development of "smart" coatings — designing multi-functional materials with a capacity to self-diagnose damage, which will increase the service life of structures and reduce maintenance costs.

Implementing these measures would not only expand the application scope of recycled wind turbine blades, but also boost the economic attractiveness of the technology contributing to its large-scale adoption in the construction industry.

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Load-Bearing Capacity of Bamboo Joints and their Use in Truss Structures

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Abstract

Introduction. Bamboo structures have become widespread in Asia, Africa, and Latin America. Bamboo is a gradient material with unequal cross-sectional properties and characteristic anisotropy: good properties in the longitudinal and weak transverse directions. The connection of bamboo rods thereby represents a weak point in the design, which is a scientific issue. In modern literature, the lack of the efficiency of various types of bamboo rod joints has been shown leading to a progressive collapse of a structure. The identified gaps in the existing research has enabled us to formulate the aim of the article, which is to develop new types of bamboo rod connections to ensure safe and reliable operation of the truss structure.

Materials and Methods. The object of the study is a bamboo truss with a wall thickness of at least 10 mm. The trusses were calculated by means of the advanced methods of cutting nodes, selecting cross-sections, and designing influence lines.

Research Results. A new design for connecting bamboo rods in the spatial case has been set forth. The advanced spatial hinge is a one-piece hot-forged steel sphere with 18 threaded holes and a machined support surface at angles of 45°, 60° and 90° in relation to each other. A conical steel section is attached at each end of the spatial structure element to transfer force from the bamboo joints to the nodal ones. Due to this tapering cone-shaped section, the nodal joints can be connected to lots of elements at once. The pedestrian bridge truss has been calculated for various load application options. It is shown that the suggested type of connection ensures efficient operation of the spatial structure. The actual reliability factor of 2.33 is 29% over the traditional value.

Discussion and Conclusions. The suggested options for ensuring a reliable connection of bamboo rods are of primary importance in the design and construction of bamboo truss structures of a spatial type. A spherical hinge and a conical attachment with a metal cable create a reliable connection, which is critical for bridge-type structures or residential buildings. The prospects of the work are focused on investigating the efficiency of the suggested compounds in dynamic tasks under a moving load and creep.

Keywords: bamboo, truss, connecting bamboo rods, pedestrian bridge

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Оригинальное эмпирическое исследование

Несущая способность соединений из бамбука и их применение в ферменных конструкциях

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Аннотация

Введение. Конструкции из бамбука получили широкое распространение в странах Азии, Африки, Латинской Америки. Бамбук представляет собой градиентный материал с неодинаковыми свойствами по поперечному сечению и характерной анизотропией: хорошими свойствами в продольном и слабыми поперечном направлениях. В связи с этим соединения бамбуковых стержней представляет слабое место в конструкции, что является научной проблемой. В современной литературе показана недостаточная эффективность различных типов соединений бамбуковых стержней, что приводит к прогрессирующему обрушению конструкции. Выявленные пробелы в существующих исследованиях позволили сформулировать цель настоящей статьи: разработка новых типов соединений стержней бамбука для обеспечения безопасной и надежной работы ферменной конструкции.

Материалы и методы. Объектом исследования является ферменная конструкция из бамбука с толщиной стенки не менее 10 мм. Расчет ферм проводился по усовершенствованным методам вырезания узлов, подбора сечений и построения линий влияния.

Результаты исследования. Предложена новая конструкция соединения бамбуковых стержней в пространственном случае. Усовершенствованный пространственный шарнир представляет собой цельную стальную сферу горячей ковки с 18 резьбовыми отверстиями и обработанной опорной поверхностью под углами 45°, 60° и 90° относительно друг друга. На каждом конце элемента пространственной конструкции прикрепляется коническая стальная секция для передачи усилия от бамбуковых соединений к узловым соединениям. Благодаря такому сужающемуся конусообразному сечению узловые соединения могут быть соединены со многими элементами одновременно. Проведен расчет фермы пешеходного моста при различных вариантах приложения нагрузки. Показано, что предложенный тип соединения обеспечивает эффективную работу пространственной конструкции. Реальный коэффициент надежности 2,33 превосходит традиционное значение на 29 %.

Обсуждение и заключения. Предложенные варианты обеспечения надежного соединения бамбуковых стержней имеют большое значение при проектировании и строительстве бамбуковых ферменных конструкций пространственного типа. Сферический шарнир и конусное крепление с металлическим тросом создают надежное соединение, что имеет решающее значение для конструкций мостового типа или жилых помещений. Перспективы настоящей работы сосредоточены на исследовании эффективности предложенных соединений в динамических задачах в условиях движущейся нагрузки и ползучести.

Ключевые слова: бамбук, ферма, соединение бамбуковых стержней, пешеходный мост

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Для цитирования. Сюаньчжэнь Сун, Муселемов Х.М., Бескопыльный А.Н. Несущая способность соединений из бамбука и их применение в ферменных конструкциях. *Современные тенденции в строительстве, градостроительстве и планировке территорий*. 2026;5(1):104–114. https://doi.org/10.23947/2949-1835-2026-5-1-104-114

Introduction. Bamboo structures have become widespread in Asia, Africa, and Latin America, i.e., in the regions where bamboo has been actively cultivated and in abundance. In Latin American countries, bamboo has traditionally been used for building residential buildings, bridges, galleries, and lots of other structures. The unique properties of bamboo make it inexpensive to use and environmentally friendly, which aligns with the "green" construction agenda. Bamboo structures offer aesthetic advantages creating authenticity and harmony with nature. In the northern regions of Ecuador, such a trend has existed for lots of centuries and has resulted in a reduction in traditional building materials in favor of wooden and bamboo frames covered with clay plaster traditionally combined with the local architecture.

As a building material bamboo offers unique advantages, including rapid growth (up to 1 m per day), high carbon sequestration capacity — up to 17 tons of CO_2 per hectare annually, which is four times more than in tropical forests [2, 3]. There are abundant bamboo resources in tropical countries such as China, India, etc. (almost 3.5 billion bamboos per year in China), create significant biodiversity and provide a vital human habitat. Research of bamboo structures, houses and residential buildings using bamboo materials serves to preserve cultural and historical identity. Bamboo houses have witnessed considerable changes in materials and technologies used at various stages of evolution — from houses with bamboo construction to those with wooden structures, hybrid ones made of wood and brick, brick-concrete ones and, finally, with a concrete frame, while maintaining their traditional visual appearance [4]. It is also noteworthy that in some countries [5] the preservation of the traditional appearance is included in the national program and is on the UNESCO Cultural Heritage list.

Even though bamboo-based materials have played a role throughout human civilization, it was rather late that a comprehensive understanding of their relationship between structure and properties has emerged. Bamboo is a gradient material with unequal cross-sectional properties and characteristic anisotropy in the longitudinal and transverse directions [6]. Bamboo has excellent properties in the longitudinal direction, which enables it to be used as fiber additives [7,8] or reinforcing structural elements.

In order to eliminate this major disadvantage of bamboo, i.e., fragility in the transverse direction, bamboo-epoxy composites are used that are capable of significantly strengthening the overall structure, while maintaining the advantages and appearance [9]. In order to get a better understanding of the effective operation of bamboo rods in construction, it is necessary to know the features of bamboo destruction [10]. Bamboo is more susceptible to rotting than ordinary wood due to the absence of natural toxins and thin fiber walls. This causes a decrease in its dynamic properties [11, 12] and the need to develop laminated bamboo [13]. Bamboo is sensitive to changes in temperature and humidity, which leads to cracking in the transverse direction. Changes in these temperature and humidity conditions result in the expansion and compression of the material, which causes microscopic cracks or fractures [14]. The weak point of bamboo structures is the connection of bamboo rods [15]. While using nagels and bolts, there might be gaps in the joints of bamboo elements leading to wea kening of the structure.

a) *b)*

Fig. 1. Examples of connecting bamboo rods: *a* — through; *b* — strapping

Thus, it can be seen that the unreliable connection of bamboo rods is one of the causes of structural failure, which, due to the fragile behavior of bamboo, leads to progressive structural failure. At the moment, the search for reliable and inexpensive fasteners and joints of bamboo segments is a scientific issue. An effective joint is a major part of the structure determining its strength and reliability. The identified gaps in the existing research has enabled us to formulate the aim of this article, which is to develop new types of bamboo rod connections to ensure safe and reliable operation of the truss structure.

Materials and Methods. Bamboo has been considered as the material of the rod. The tensile strength of a joint with a few nodes is determined by the weakest point in the rod. Bamboo with a wall thickness of at least 10 mm is used to identify the strength of the joint. This minimum wall thickness is taken from the smallest wall thickness of the various types of samples. This is to ensure that the load-bearing capacity of each node reaches at least 24 kN. This assumption is based on the 80% ultimate load capacity of the sample, which was the lowest one among the three bamboo compound samples.

After the bamboo has been selected, the permissible load on the joint and the safety factor are to be identified. Unless there are some other preferences rather than strength while choosing a rod, the next step is to identify the appropriate rod with a maximum load capacity slightly exceeding the permissible load multiplied by the safety factor. After that, the bamboo joint will be designed based on the maximum load capacity of this rod, as the rod must be designed as the weakest part of the joint to ensure the strength of the joint at a certain level.

The next step is to calculate the number of nodes to be used by dividing the maximum load capacity of the rod $(F_{u,b,c,\parallel})$ by 24 kN. The last step is to identify suitable connecting wires so that a minimum tension force is created in the rod until all of the wires have been stretched.

The breakage $(F_{m,w})$ occurs if the permissible load on the rod has been exceeded. As it is more challenging to work with larger steel wires, smaller ones can be used, but then the number of nodes is to be increased.

While using rings and bamboo with approximately the same properties, the composition of the connectors is identified in the following way. If it is planned that the permissible load on the joint will be 15 kN with a safety factor of 2, the M12 rod of class 4.6 should be selected as its characteristic strength is 33.70 kN. If the tensile strength of each node is 24 kN, two nodes are needed to ensure stronger nodes (48 kN in total) than on the rod. The minimum force applied to the rod until each wire breakages is $(F_{m,w} = 2.41 \cdot f_{u,w})$ while using two wires with a diameter of $4mm$ and limiting strength $9.41kN$

$$F_{m,w} = 2.41 \cdot f_{u,w} = 4.82 \cdot 9.41 = 45.36kN$$

$F_{m,w} = 45.36 -$ still exceeds the permissible load of the used rod $(F_{u,r})$.

The wire can be replaced with a smaller diameter (3mm) with a characteristic strength $(f_{u,w} = 5.59kN)$, e.g., depending on availability, but the number of nodes is to be increased to three. The maximum load on the nodes reaches $72kN$, and the minimum force applied to the rod until the breakage of all of the wires is:

$$F_{m,w} = 3 \cdot 2.41 \cdot f_{u,w} = 3 \cdot 2.41 \cdot 5.59 = 40.42kN$$

$$F_{m,w} = 40.42 > F_{u,r}$$

The composition of the other used bars and steel wire, as well as the number of nodes, can be obtained in the same manner. Examples of the results are shown in Table 1. To this end, the lowest and highest class bars available on the market are used. While using bamboo with similar properties, but with a tensile strength limit obtained as a result of various studies and components $82.62\ N/mm^2$, the ultimate strength of the entire cross-section over the smallest cross-sectional area is $224.23kN$.

Table 1

Dimensions of the rod, number of nodes and diameter of the steel wire

No.	Rod				Node		Steel wire	
	Class	Diameter	Permissible load	Maximum load-bearing capacity $(F_{u,r})$	Number of nodes	Maximum load-bearing capacity $(F_{u,k,t,\parallel})$	Diameter	Maximum load-bearing capacity $(F_{u,r})$
		mm	kN	kN		kN	mm	kN
1	4.6	12	**16.86**	33.72	2	48.00	4	45.19
2					3	72.00	3	40.25
3		16	**31.40**	62.80	3	72.00	4	67.78
4		20	**49.00**	98.00	5	120.00	4	112.97
5	8.8	14	**46.00**	92.00	4	96.00	5	138.39
6					5	120.00	4	112.97
7		18	**76.80**	153.60	8	192.00	4	180.76

Research Results. In order to create an effective structure with a large span, the elements of the spatial structure must be loaded only by means of stretching or compressing. The suggested bamboo fastener-based joints also comply with this requirement. Due to the connection with no mortar or resin, the suggested compounds provide a relatively easier connection than injection ones. This is consistent with the principle of spatial structure as a lightweight structure.

Fig. 2. Application of the joint for a spatial structure

Among some nodal joints, the Mero nodal joint is one of the most popular ones (Fig. 2). The original Mero ball joint is a single hot-forged steel sphere with 18 threaded holes and a machined support surface at angles of 45°, 60° and 90° in relation to each other. In order to make use of these nodal connections in a spatial bamboo structure, some improvements are to be made as shown in Fig. 3. A conical steel section should be attached at each end of the spatial structure element in order to transfer force from the bamboo joints to the nodal ones. Due to this tapering cone-shaped section, the nodal joints can be connected to lots of elements at once; otherwise, there is not enough space around the nodal joint, e.g., in extreme conditions, to use all of the threaded holes, except for extending the rods of bamboo joints. However, elongating the rods of bamboo joints before joining with the nodal ones is not possible due to bending, albeit secondary, effects.

Another improvement is to separate the rod as part of the bamboo joint and the bolt to connect the nodal one. This can be done by using a conical steel cone section, so that the the section to the bamboo joint or to the nodal one can be attached separately.

Let us consider the application of the suggested connection using the example of a pedestrian bridge (Fig. 4). As a very simple technology is used in manufacturing bridges, and bamboo is easy to assemble, after a while bridges can be replaced with new ones with no problems encountered. However, use of traditional technology is limited to bridge spans. Bamboo bridges have recently been marketed as environmentally friendly, which is challenging for lots of architects and engineers, particularly after the problem of bamboo durability has been solved using modern conservation technologies.

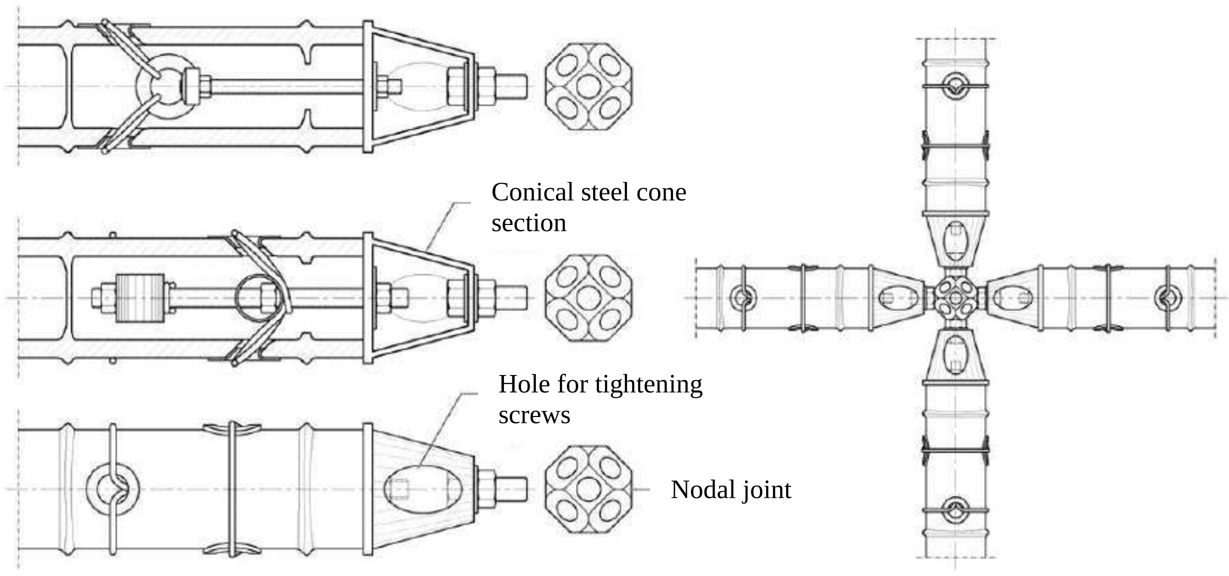


Fig. 3. Conceptual application of the bamboo joint based fastening for a spatial structure

Fig. 4. Example of a simple bridge design with a 10-meter span

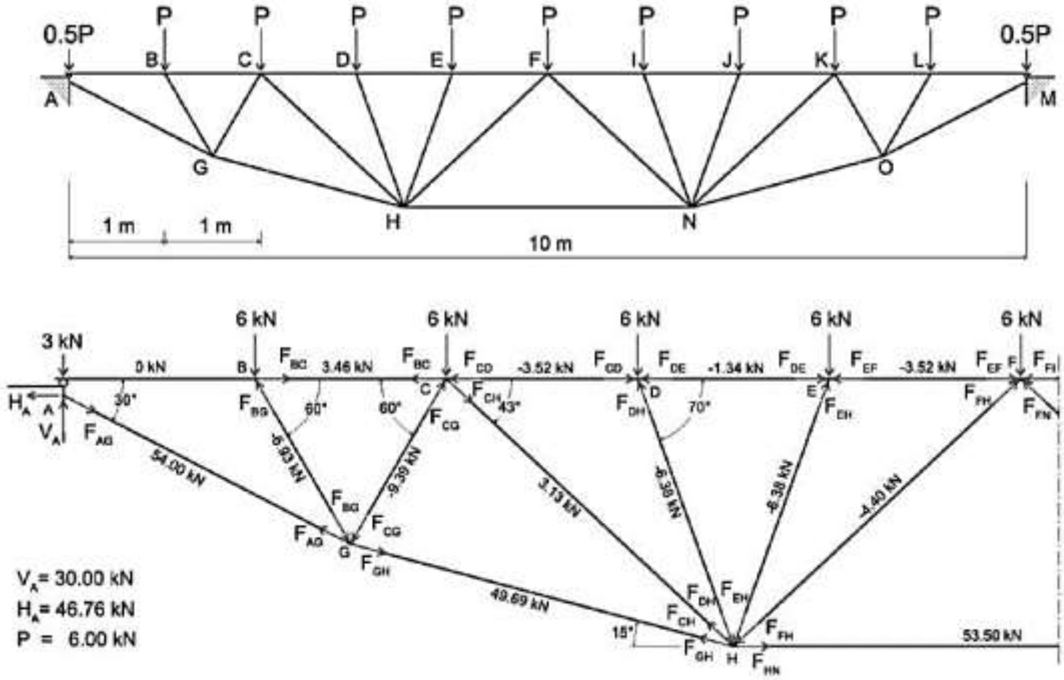


Fig. 5. Distribution of forces in the rods of the bridge truss

Prior to starting designing the joints, it is necessary to calculate the distribution of forces throughout the structure. It is assumed that the bridge is a pedestrian one with a distributed load $5\,\mathrm{kN/m^2}$ including the operating and non-operating load (Fig. 5).

The span of the bridge is $10m$ with the width 1.20m. The total load on the bridge can be obtained:

$$\mathrm{W} = 5\frac{\mathrm{kN}}{\mathrm{m}^2}\cdot 10\mathrm{m}\cdot 1.2\mathrm{m} = 60\mathrm{kN}$$

The modulus of elasticity of bamboo along the fibers is 12222.3 MPa, the modulus of elasticity across the fibers is 1489 MPa. The Poisson's ratio along the fibers is 0.325, the Poisson's ratio across the fibers is 0.039.

According to the calculations, the high tension forces act on all of the elements in the lower part of the structure: $FAG = FOM = 54.00kN; FGH = FNO = 49.69kN; FHN = 53.50kN.$ The other elements created with the low tension forces are important $FBC = FKL = 3.46kN \wedge 148FCH = FKN = 3.13kN.$

The remaining elements, exception those with a neutral axis AB or LM, are exposed to the relatively low compression forces in the range from 1.34 to 9.39 kN. Based on the calculations, tensioning joint designs are divided into two types: to ensure a high load-bearing capacity, a bamboo joint with a few nodes is used, and to ensure a low load-bearing capacity, a bamboo joint with an eye bolt is used.

The high tension forces in the bridge structure such as $FAG, FGH, FHN,$ are transmitted with two parallel racks. Thus, half of the force falls on each rack. Using the highest tension force as the foundation of the calculation, the joints with multiple nodes must withstand a load of 27.00 kN. This is considered as an acceptable load.

While using a safety factor of 2, the maximum load on the hinge should be $54.00kN$. According to Table 1, it is possible to make use of M16 class 4.6 rods with a maximum load of 62.80 kN. On top of that, the number of the nodes and the wire diameter can be defined as 3 nodes and 4 mm, respectively. It is with this combination that the rod has the lowest limiting load capacity.

Thus, an actual reliability factor is achieved:

$$\mathrm{F.S.} = \frac{62.8\mathrm{kN}}{27.00\mathrm{kN}} = 2.33$$

In order to simplify the calculations, they were performed for a single-layer frame in two dimensions as shown in Fig. 5–11.

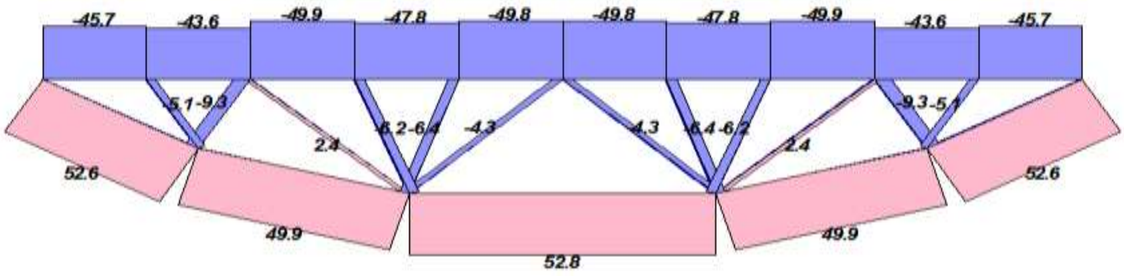
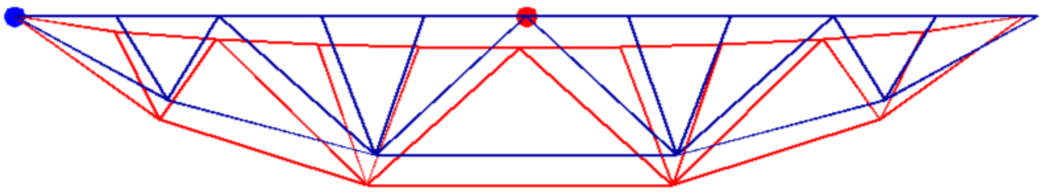


Fig. 6. Forces in the rods, kN



Max: Node 1, Uz = 0.000 mm Min: Node 6, Uz = –32.538 mm

Fig. 7. Node movements, mm (the scale of movements has been increased by 10 times)

Loading the left half of the span

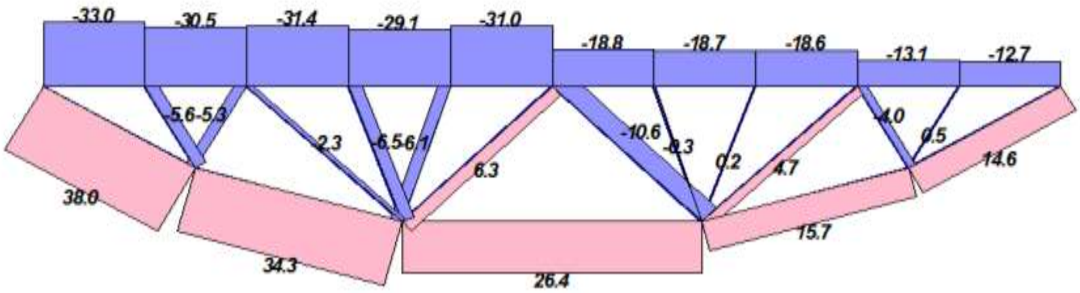
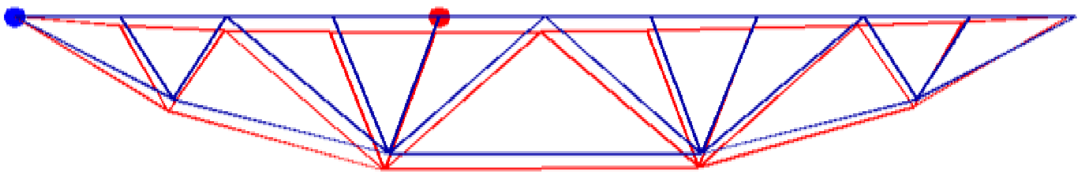


Fig. 8. Forces in the rods, kN



Max: Node 1, Uz = 0.000 mm Min: Node 5, Uz = –17.250 mm

Fig. 9. Node movements, mm (the scale of movements has been increased by 10 times)

Influence line

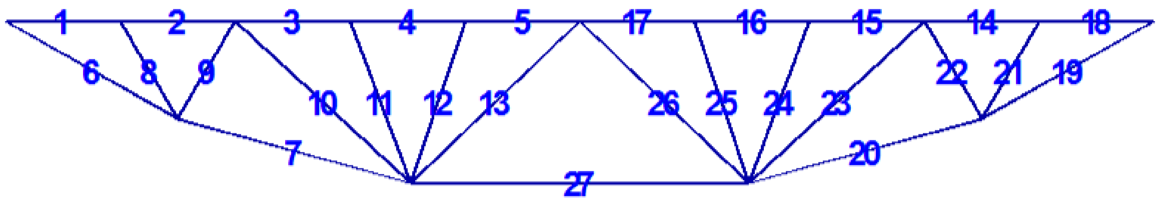


Fig. 10. Numbering of the rods

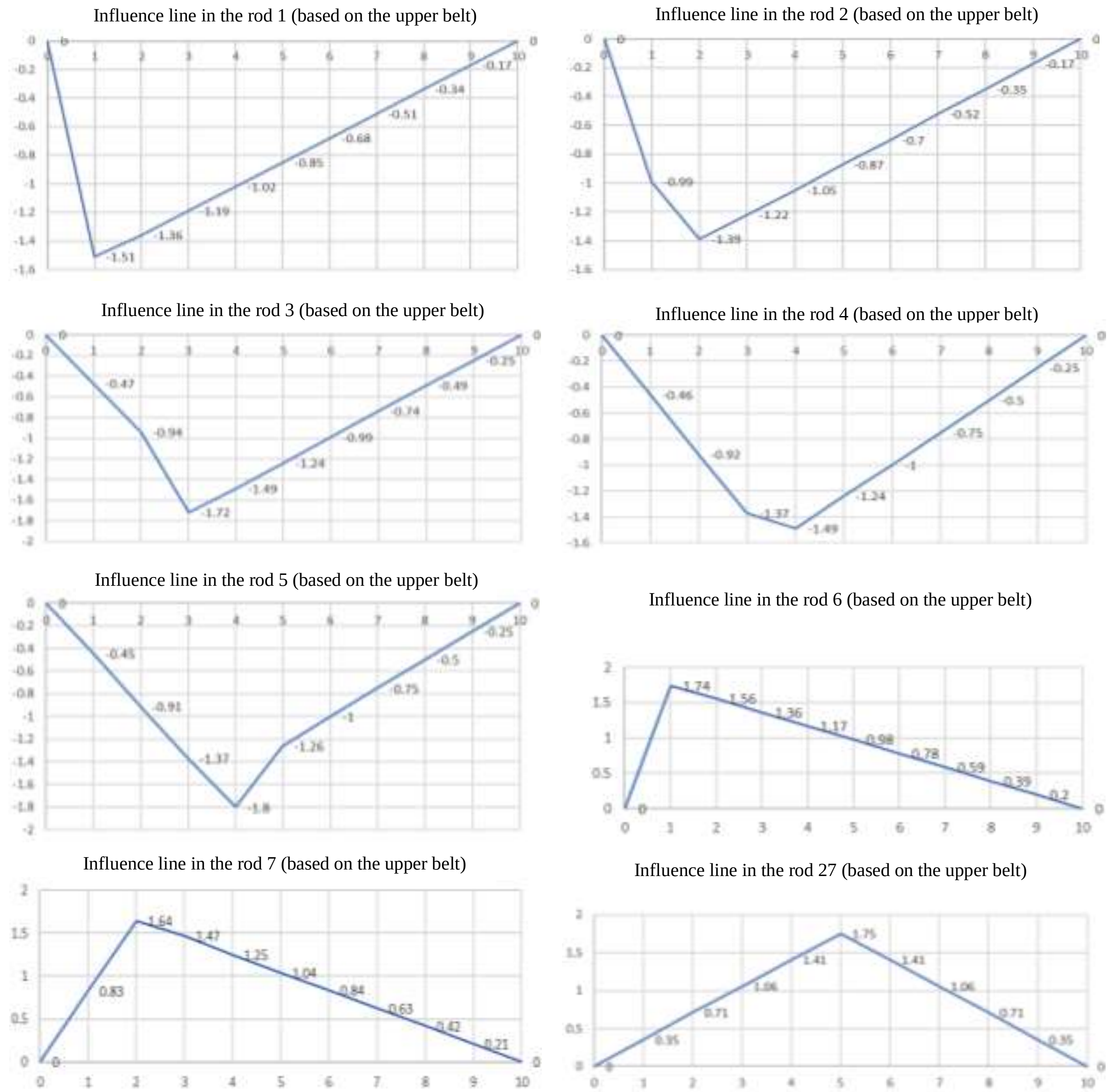


Fig. 11. Influence lines in the truss belts (single force in the upper belt)

The analysis of the lines of influence of longitudinal forces has caused the following conclusions to be made:

— the maximum forces in the belts of the truss occur under a load acting on all of the nodes of the upper belt. At the same time, there are negative forces in the rods of the upper belt, and tensile forces in the rods of the lower one.

— the maximum efforts in the braces of the farm will occur:

a) for rod 8 – the force at point X = 1; for rod 11, the force at point X=3; for rod 12, the force at point X=4.

b) in rods 9, 10, 13, the influence line is alternating. For rod 9, the maximum tensile force is at the force at the node X =1, the maximum compressive force is at the simultaneous action of forces at the nodes $X \in [2\ 10]$. For rod 10, the maximum tensile force is $X \in [3\ 10]$, the maximum compressive force is $X \in [1\ 2]$. For rod 13, the maximum tensile force is $X \in [0\ 4]$, the maximum compressive force is $X \in [5\ 10]$.

Discussion and Conclusions. The suggested options for a reliable connection of bamboo rods are of primary importance in designing and constructing bamboo truss structures of a spatial type. A spherical hinge and a conical attachment with a metal cable create a reliable connection, which is critical for bridge-type structures or residential buildings. The prospects of the work are focused on investigating the efficiency of the suggested compounds in dynamic tasks under a moving load and creep.

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