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Building Constructions, Buildings and Engineering Structures
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Construction Materials and Products
Technology and Organization of Construction
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Современные тенденции в строительстве, градостроительстве и планировке территорий

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2.1.5 – Строительные материалы и изделия (технические науки)
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Evaluating Building Operational Safety under Variations of the Technical Condition of Foundations

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Abstract

Introduction. Modern buildings operate under changing climate and loading conditions, where foundation degradation leads to cracks, uneven precipitation and load-bearing capacity loss. Regular monitoring and inspections enable timely risk detection, accident prevention and repair cost reduction. The aim of the study is to develop methods for assessing operational safety across various foundation conditions, including calculating precipitation and performance limits.

Materials and Methods. The study included visual inspection, structural scheme identification, instrumental diagnostics, building inspection with preparation of the architectural drawings, defect detection and documentation, installation of pits with preparation of measuring drawings of epy foundations for open pits and identification of the strength characteristics of the foundation material, opening structures and testing building materials to identify their strength characteristics with subsequent verification calculations of the load-bearing elements.

Research Results. Based on the results of the verification calculations and identified structural defects, the condition of the building elements and the building as a whole has been established. Recommendations have been developed for ensuring the normal operation of a building as well as for strengthening or repairing structures.

Discussion and Conclusion. Comprehensive technical inspection of the building using advanced non-destructive testing methods and verification calculations revealed the current technical condition of the foundation elements and foundations. Based on the results of the technical inspection, taking into account the results of the verification calculations, it can be concluded that for further normal operation of a building in some areas, it is necessary to take measures in order to strengthen the foundations or strengthen foundations soils. The scientific significance lies in the developed and testing of the methodology for comprehensive assessment of the load-bearing capacity of foundations taking into account real-world operating conditions, which makes it possible to increase the accuracy of forecasting the durability of buildings on weak soils. The practical value of the results lies in the formation of specific recommendations for strengthening foundations. The findings can be used in inspecting similar civil and industrial facilities.

Keywords: foundations, engineering and technical inspection, verification calculations, technical condition, deformation and strength characteristics

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Оригинальное эмпирическое исследование

Анализ эксплуатационной безопасности здания при вариациях технического состояния фундаментов

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Аннотация

Введение. Современные здания эксплуатируются в условиях изменяющегося климата и нагрузок, где изменения технического состояния фундаментов могут привести к трещинам, неравномерным осадкам и потере несущей способности. Регулярный мониторинг и обследование позволяют вовремя выявлять риски и предотвращать аварии, снижая затраты на ремонт. Целью исследования в данной статье является разработка методов оценки эксплуатационной безопасности при различных технических состояниях фундаментов здания, включая расчет осадок и пределов работоспособности.

Материалы и методы. Исследование включает в себя визуальный осмотр, определение конструктивной схемы здания, инструментальную диагностику, выполнение обмерных работ по зданию с составлением архитектурных чертежей, выявление и фиксацию дефектов, устройство шурфов с составлением обмерных чертежей фундаментов по отрытым шурфам и определение прочностных характеристик материала фундаментов, выполнение вскрытий конструкций и испытаний строительных материалов с определением их прочностных характеристик с последующим выполнением поверочных расчетов несущих элементов.

Результаты исследования. На основании результатов поверочных расчетов и выявленных дефектов по конструкциям устанавливалось состояние строительных элементов и здания в целом. Разработаны рекомендации для обеспечения нормальной эксплуатации здания и рекомендации по усилению или ремонту конструкций.

Обсуждение и заключение. Проведенное комплексное техническое обследование здания с применением современных методов неразрушающего контроля и поверочных расчетов позволило выявить текущее техническое состояние конструктивных элементов фундаментов и оснований. По результатам проведенного технического обследования с учетом результатов поверочных расчётов можно сделать вывод, что для дальнейшей нормальной эксплуатации здания на отдельных участках требуется выполнить мероприятия по усилению фундаментов или укреплению грунтов оснований. Научная значимость исследования заключается в разработке и апробации методики комплексной оценки несущей способности оснований и фундаментов с учетом реальных условий эксплуатации, что позволяет повысить точность прогноза долговечности зданий на слабых грунтах. Практическая ценность результатов состоит в формировании конкретных рекомендаций по усилению фундаментов. Полученные выводы могут быть использованы при обследовании аналогичных объектов гражданского и промышленного назначения.

Ключевые слова: фундаменты, инженерно-техническое обследование, поверочные расчёты, техническое состояние, деформационные и прочностные характеристики

Для цитирования: Римшин В.И., Евдокименко А.С., Кецко Е.С., Воробьев А.Е., Савельев Е.С. Анализ эксплуатационной безопасности здания при вариациях технического состояния фундаментов. *Современные тенденции в строительстве, градостроительстве и планировке территорий*. 2026;5(2):7–21. https://doi.org/10.23947/2949-1835-2026-5-2-7-21

Introduction. Modern buildings are being operated in changing climatic conditions, dynamic loads and heterogeneous weak soils (bulk sands, water-saturated clays) leading to deformations of foundations, cracks, uneven precipitation and further to emergencies. The methods currently employed tend to focus on new structures, with no consideration of the operational characteristics of structures. The aim of the study is to develop a methodology for assessing the operational reliability of building foundations on various types of soils, calculating precipitation and identifying the technical condition of structures. To this end, the following tasks were set: to conduct an instrumental examination, perform verification calculations for Group I and Group II of limiting conditions, and to develop recommendations for strengthening foundations [1–4].

Materials and Methods. The work on the technical inspection of foundations and bases of a building included installing pits with preparing dimensional drawings of the foundations for the excavated pits and identifying the strength characteristics of the foundation material. The depth of the pits was identified taking into account the establishment of the actual depth of the existing foundations, their structures, strength of materials, etc. Soil samples were mostly taken in the range of 10–60 cm below the soles of the foundations of the undisturbed structure (3 samples for each pit), laboratory tests of the samples were performed based on the standard methods in compliance with the current state standards. The deformation and strength characteristics of the soils were identified according to Appendix B of SP 22.13330.2011 "Foundations of Buildings and Structures".

Based on the results obtained from the excavated pits, verification calculations of the bases of the foundations for Group II of limiting states (deformations) were performed. The calculations were conducted taking into account Clause 5.3.18 of SP 22.13330.2011.

While performing the verification calculations of the foundations, the following parameters were set: the calculated resistance of the foundations, permissible load on the foundations, pressure on the soles of the foundations considering the existing loads.

The inspection of vertical and horizontal load-bearing structures, staircases and roofs included a study of the condition of the load-bearing elements, including measurements with preparing architectural drawings of structural components at the excavation sites.

Defects were identified and photofixed in the interior of the building (in the areas of actual access) with preparing diagrams of the location of defects on the floor plans and fixing defects on the facades of the building with the application of defective areas on the sketch drawings of the facades.

Through the course of the technical inspection, the strength characteristics of building materials of load-bearing structures were identified by means of non-destructive testing methods. The concrete strength class was identified based on the direct measurements of concrete strength using non-destructive methods according to GOST 22690 "Concretes. Identifying the Strength by Means of Mechanical Methods of Non-Destructive Testing" (by means of the shock pulse method) and taking into account the requirements of Clause 8.3.5 of SP 13-102-2003 "Rules for Inspection of Load-Bearing Building Structures of Buildings" (the actual concrete strength class was set as $B = 0{,}8 \cdot R_{sc}$)[1,2] [1–3].

The strength characteristics of the reinforcement structures were identified based on an expert assessment considering the profile of the reinforcement, the year of construction of the building as well as the data of SNiP 2.03.01-84* "Concrete and Reinforced Cconcrete Structures". The calculated resistance of the rod reinforcement is assumed to be equal to R_s = 510 MPa, R_{sc} = 450 MPa — for fittings of the A600 periodic profile.

Tests of metal structures were not performed. The strength characteristics of rolled steel elements were identified according to SP 13-102-2003 and available reference data. While calculating the steel elements, the value of the calculated tensile strength of steel is assumed to be equal for columns made of rolled channels R_y = 230 MPa (for steel C235), for beams made of rolled I-beams, corner support tables and steel strips R_y = 240 MPa (for steel C245). For the calculation of welded joints, it is assumed that manual electric arc welding with E42 electrodes has been performed.

Based on the results of the excavations, measurements, and calculation of the strength characteristics of the materials, piecemeal verification calculations of horizontal and vertical load-bearing structures were performed.

Verification calculations of structures were performed according to Group I of limiting states (in terms of the load-bearing capacity) according to Section 10 of SP 13-102-2003. Verification calculations of steel structures were performed in compliance with SP 16.13330.2011 "Steel Structures", of reinforced concrete in compliance with SP 52-101-2003 "Concrete and Reinforced Concrete Structures without a Prestressing Reinforcement".

The verification calculations were performed considering the useful load for the ceiling above the 1st floor identified using SP 20.13330.2011 "Loads and Impacts" and amounting to 200 kg/m^2 (standard value). The useful snow load on the surfacing is assumed to be 180 kg/m^2 (snow region III). The standard load from the partitions is assumed to be 50 kg/m^2. The weight of the floors is 150 kg/m^2 (standard value). Since the roof was not opened, the load from the pie was assumed based on the expert assessment.

[1] Bondarenko VM, Rimshin VI *Dissipative Theory of Force Resistance of Reinforced Concrete*. Moscow: Student; 2015. 111 p.
[2] Kurbatov VL, Rimshin VI, Dironas MV *Control and Supervision in Construction and Housing and Communal Services*. Mineralnye Vody; 2024.

While performing the verification calculations, the loads from the own weight of the structures were identified based on the excavations and measurements. The specific weight of the materials was identified based on the expert assessment: concrete γ_f = 24 kN/m^3; reinforced concrete γ_f = 25 kN/m^3; steel γ_f = 78,5 kN/m^3; bricks γ_f = 18 kN/m^3; claydite-concrete γ_f = 12 kN/m^3; cement-sand screed γ_f = 18 kN/m^3; backfilling soil γ_f = 16 kN/m^3.

Given the fact that selective excavations were performed, the load on the structures in some areas was identified based on the expert assessment. The forces in the structures were identified by means of the methods of structural mechanics. Based on the results of the verification calculations of the vertical and horizontal load-bearing structures, the following parameters were established: values of the actual load-bearing capacity of vertical and horizontal load-bearing structures, coefficient of utilization of the load-bearing capacity of the existing elements of vertical and horizontal load-bearing structures. The condition of the structures and the structure as a whole was identified based on the technical inspection and verification calculations. The inspection of the load-bearing structures, identification of the strength of the materials, and verification calculations were performed in compliance with the current regulatory documents [4–10].

The technical inspection of the building was performed in stages. Preliminary examination involved familiarization with the available documentation, general inspection of the facility, continuous visual inspection of the building structures, identification of defects and damage based on the external signs with a necessary measurements and fixation. A detailed continuous inspection involved performing measurements, measuring the strength characteristics of the materials of the load-bearing structures, performing openings of the load-bearing structures, setting up pits, performing measurement drawings of foundations and structures at the opening sites, performing engineering and geological inspections of the bases of foundations, designing measurement drawings of the floor plans, facades, sections, roof plan as well as the load-bearing layout schemes floor and coating structures, measuring the necessary geometric parameters of building structures, elements, nodes and foundations, instrumental identification of defect and damage parameters, performing verification calculations, analyzing the inspection results, writing a technical report with the conclusions based on the inspection results, preparing recommendations to ensure further normal operation of the building. The preliminary insoection included a preparatory period when the initial data was being collected and analyzed. Based on the preliminary inspection, the locations of the pits, openings and measurements of the strength characteristics of structures were established. The Onyx 2.6 device was used to identify the strength characteristics of the building materials by non-destructive methods. A total station inspection of the structures was performed by means of a Trimble M3 total station (5”)[3,4] [11–14].

Technical inspection of the building. A brief description of the inspection object

The administrative building under study was built in 1995 (two-storeyed, with no basement). The height of the rooms on the first floor from the floor level to the bottom of the floor slabs is about 3900 mm, the height of the second floor is about 2570 mm. The structural scheme of the building is a frame one. The structure is made in the form of two rectangular volumes with the overall dimensions of about 25.0×9.4 m and 37.2×9.8 m. The spatial rigidity and stability of the building is provided by the frame frames with rigid coupling of the columns with the foundations and hinged coupling of crossbars with the columns. The joint work of the frame elements is provided by rigid horizontal disks - prefabricated slabs of floor-to-ceiling and surfacing. The area around the building is planned, the courtyard is paved. An asphalt concrete blind area is made along the contour of the building. The drainage of meltwater and rainwater in an asphalt-paved area is provided by the vertical layout. The layout of the building is shown in Fig. 1.

The foundations for the frame columns are made of columnar monolithic reinforced concrete. The basement parts of the exterior walls are supported by monolithic reinforced concrete foundation beams laid on the column foundations. The wall panels of the aboveground part of the building above the basement are supported by solid concrete blocks via rolled steel elements (at the ground floor level), on the underlying panels (at the second floor level) and on steel support tables welded to the columns (at the first and second floor levels). The walls of the staircase block are supported by the concrete floors laid on the ground.

The main columns of the frame are made of composite cross-section of rolled steel channels No. 30, connected by strips of 200 ×7 mm with a pitch of about 700 mm. The main columns take the loads from the floors above the ground

[3] Rimshin VI, Bondarenko VM, Bakirov RO, Nazarenko VG *Reinforced Concrete and Stone Structures.* Moscow; Stroyizdat: 2007.
[4] *Inspection and Testing of Buildings and Structures.* Moscow; 2012.

floor and the coating, as well as from prefabricated expanded clay wall panels. Half-timbered steel columns made of paired channels No. 30 are arranged between the main columns of the frame. Half-timbered columns serve to support the exterior wall panels and withstand wind loads. There are no steel vertical connections along the frame columns.

The enclosing walls are made of prefabricated expanded clay wall panels 340 mm thick. The panels are made mainly of non-load-bearing (hinged) panels supported by steel support tables welded to the main columns of the frame and the half-timbered columns. The individual wall panels are self-supporting, supported by solid concrete blocks through steel linings made of rolled elements (at the ground floor level) and on the underlying panels (at the second floor level). The basement of the exterior walls is made of precast solid concrete blocks supported by monolithic reinforced concrete foundation beams. The facades are lined with brick about 120 mm thick. The facade cladding along the axes A, D and 9 is made of ceramic tiles in the form of factory wall panel cladding. The brickwork of the cladding is supported by prefabricated solid concrete blocks. The interior walls of the staircase block at the ground floor level are made of solid clay brick masonry on a cement-sand mortar. The wall thickness, including cladding, is about 310–460 mm. The partitions with a thickness of 90–410 mm are made of solid clay brick masonry, foam concrete blocks on cement-sand mortar and frame-sheathing of plasterboard sheets.

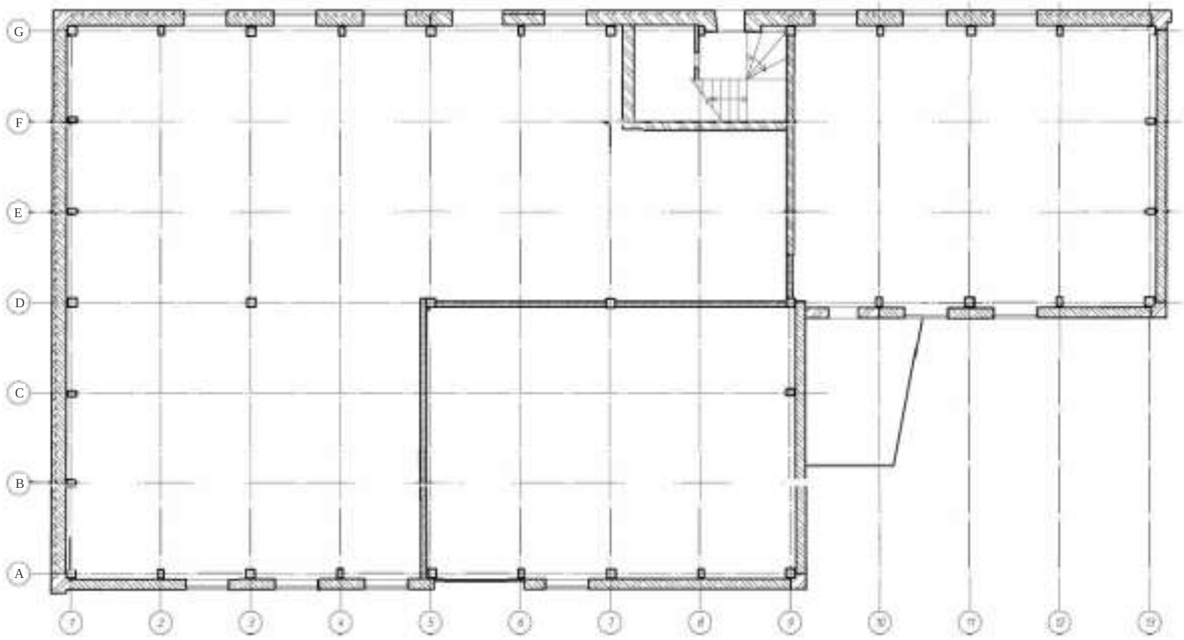

Fig. 1. Scheme of the building under study in the axes 1–13/A–G

The ceiling of the building is made according to a beam scheme of prefabricated reinforced concrete multi-hollow slabs measuring 6.0×1.2×0.22*(h)* m and 6.0×1.8×0.22*(h)* m supported by I-beam steel crossbars No. 50 and No. 50B2, as well as monolithic sections supported by reinforced concrete girders along axes A, G and 9–13.

Vertical communication throughout the building is provided by a two-marched prefabricated reinforced concrete staircase located in axes 7–9/F–G. The staircase from 0.000 m to +0.680 m is made of monolithic reinforced concrete. Safe movement along the staircase is ensured by fences made of steel elements.

The coating is made flat. Reinforced concrete multi-cavity slabs with a span of 6.0×1.2×0.22(*h*) m and 6.0×1.8×0.22(*h*) m are supported by steel crossbars of the I-beam frame No. 50 and No. 50B2, as well as monolithic sections supported by reinforced concrete girders with cross-sectional dimensions of 120×220(*h*) mm along axes A and G. Cross-section crossbars consisting of two rolling elements are united by steel bars.

Precipitation drainage is provided by sloping the roof surface in the direction of the funnels of the external downpipes. The roof is made of soft rolled material arranged on a layer of cement-sand screed and insulation. Along the contour of the roof there is a parapet made of solid clay bricks laid on a cement-sand mortar.

The exterior condition of the building is deemed satisfactory.

Results of the survey of bases and foundation soils. The column foundation along pit No. 1 in axes 9/A for the column is made in the form of a slab of monolithic reinforced concrete with overall dimensions of about 2.1×1.8×0.36(*h*) m. The coupling of the frame column with the foundation is provided by means of a column made of monolithic reinforced concrete measuring approximately 1×1×1.45(h) m. Reinforced concrete foundation beams with the overall dimensions of about 500×570 mm are arranged under the exterior walls. At the level of the top of the foundation beam, horizontal waterproofing is made from a layer of roofing material. The depth of the foundation from the level of the planned ground level is about d = 1.55 m. The layout of pit No. 1 and sections 1–1 and A–A are shown in Fig. 2.

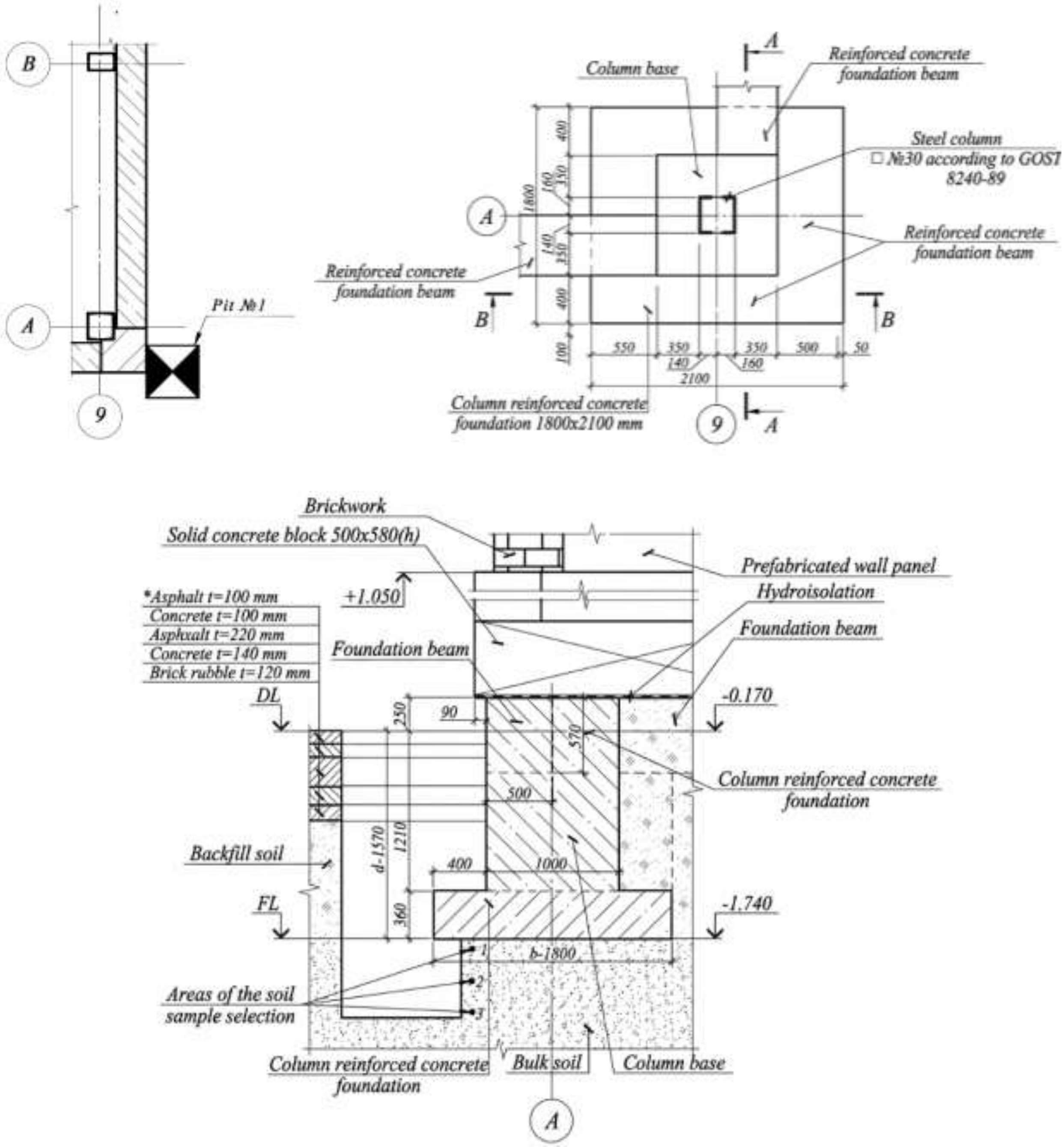

Fig. 2. Scheme of the layout of the pit No. 1 and section 1–1 and A–A

The column foundation along pit No. 2 in the 5/G axes for the column is made in the form of a slab of monolithic reinforced concrete with overall dimensions of about 2.4×2.4×0.6(h) m. The coupling of the frame column with the foundation is provided by means of a column made of monolithic reinforced concrete of 1.2×1.2×1.05(h) m. The depth of the foundation from the level of the planned ground level is about d = 2.65 m. Fig. 3 shows the layout of pit No. 2 and section A–A.

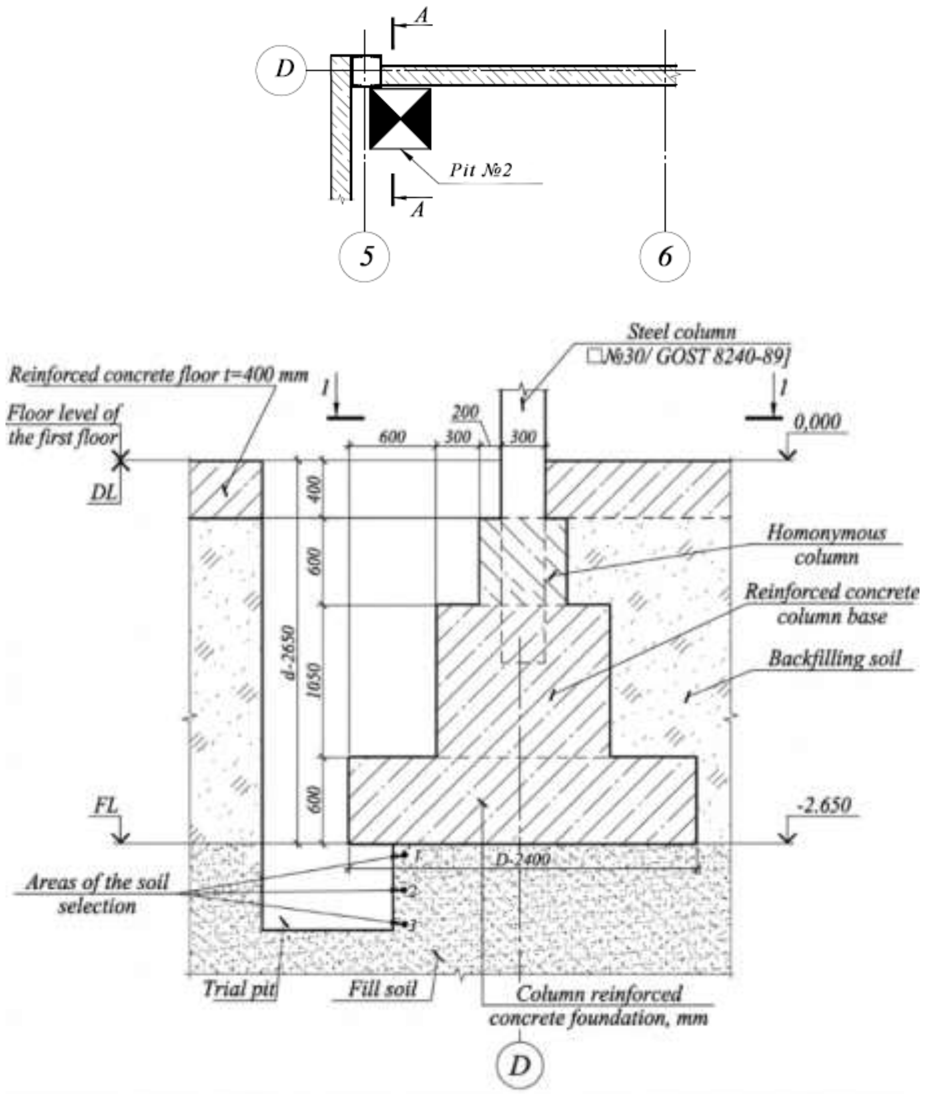

Fig. 3. Scheme of the layout of the pit No. 2 and section A–A

The columnar foundation along pit No. 3 in axes 1/H for the column is made in the form of a slab of monolithic reinforced concrete with overall dimensions of about 1.6×1.6×0.7(h) m. The conjugation of the frame column with the foundation is provided by means of a column made of monolithic reinforced concrete measuring 1.2×1.2×0.9(h) m. Reinforced concrete foundation beams with overall dimensions of about 500×570 mm are arranged under the exterior walls. At the level of the top of the foundation beam, horizontal waterproofing is made from a layer of roofing material. The depth of the foundation from the level of the planned ground level is about d = 1.6 m. In Fig. 4 shows the layout of pit No. 3 and section A–A.

The column foundation along pit No. 4 in axes 1/A for the column is made in the form of a slab of monolithic reinforced concrete with the overall dimensions of about 1.8×1.8×0.6(h) m. The coupling of the frame column with the foundation is provided by means of a column made of monolithic reinforced concrete measuring 1.0×1.0×1.0(h) m. Reinforced concrete foundation beams with the overall dimensions of about 500×570 mm are arranged under the exterior walls. At the level of the top of the foundation beam, horizontal waterproofing is made from a layer of roofing material. The depth of the foundation from the level of the planned ground level is about d = 2.0 m. Fig. 5 shows the layout of pit No. 4 and section A–A.

The foundation of the pit No. 5 in axes 7/F in sections A–A and B–B for the walls of the staircase block is made in the form of concrete floors on loose soil. The thickness of the floors is about 150 mm. There is no waterproofing of the floors. The depth of the foundation from the level of the planned ground level is about d = 1.05 m. Fig. 6 shows the layout of the pit No. 5 and section A–A.

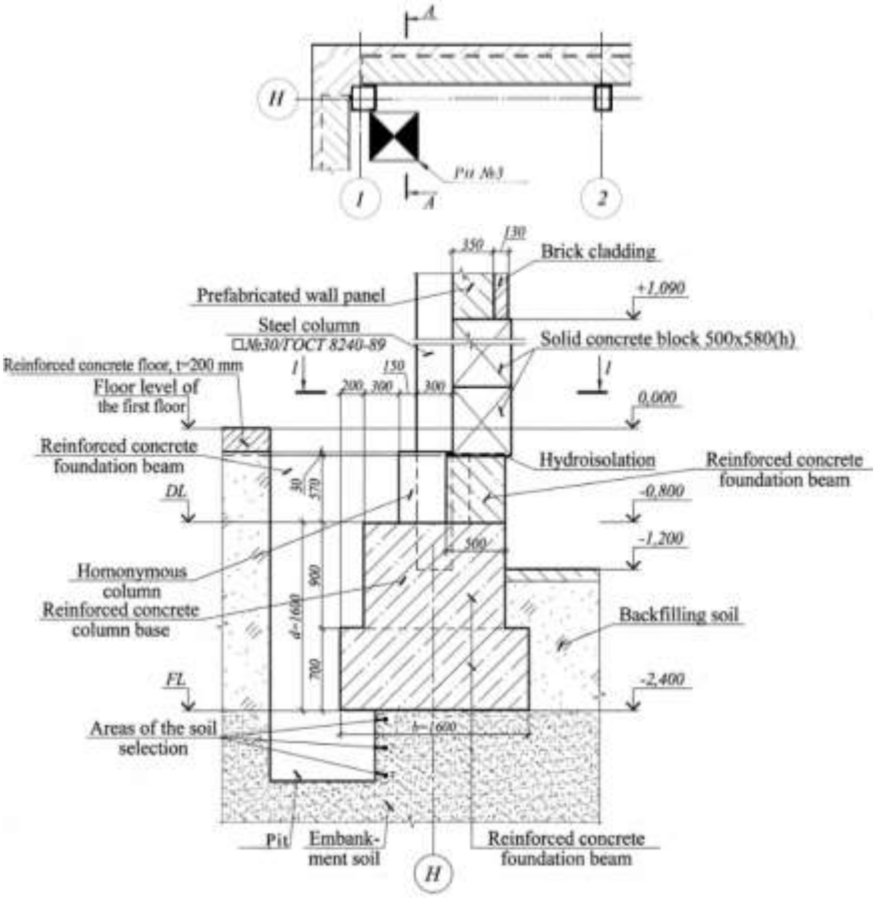

Fig. 4. Scheme of the layout of the pit No. 3 and section A–A

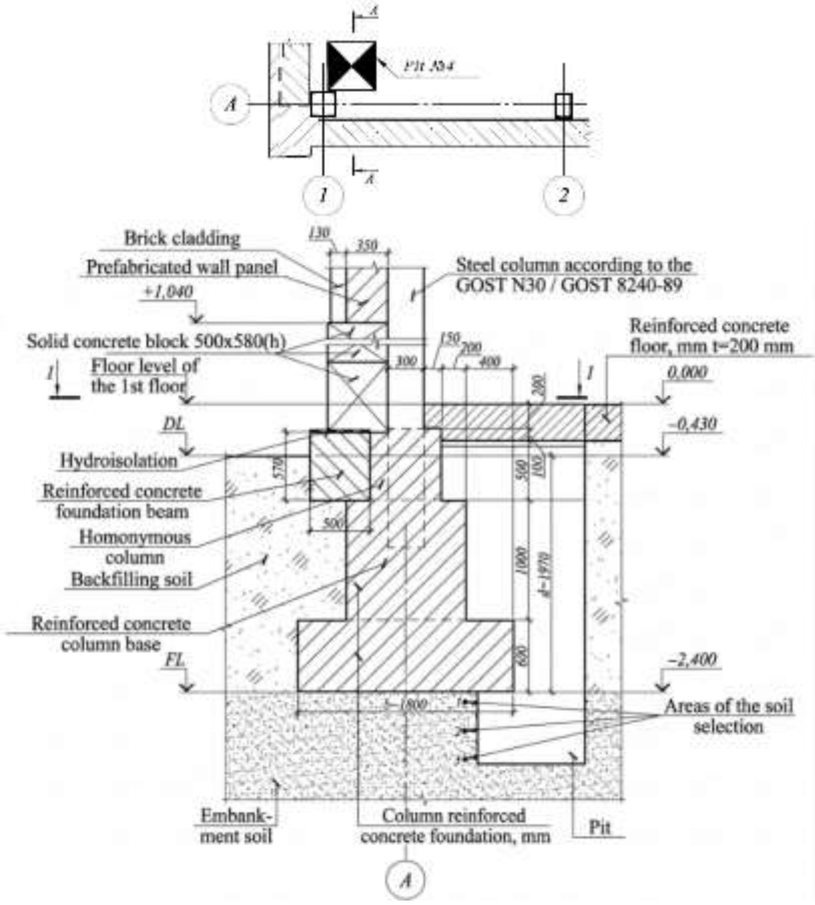

Fig. 5. Scheme of the layout of the pit No. 4 and section A–A

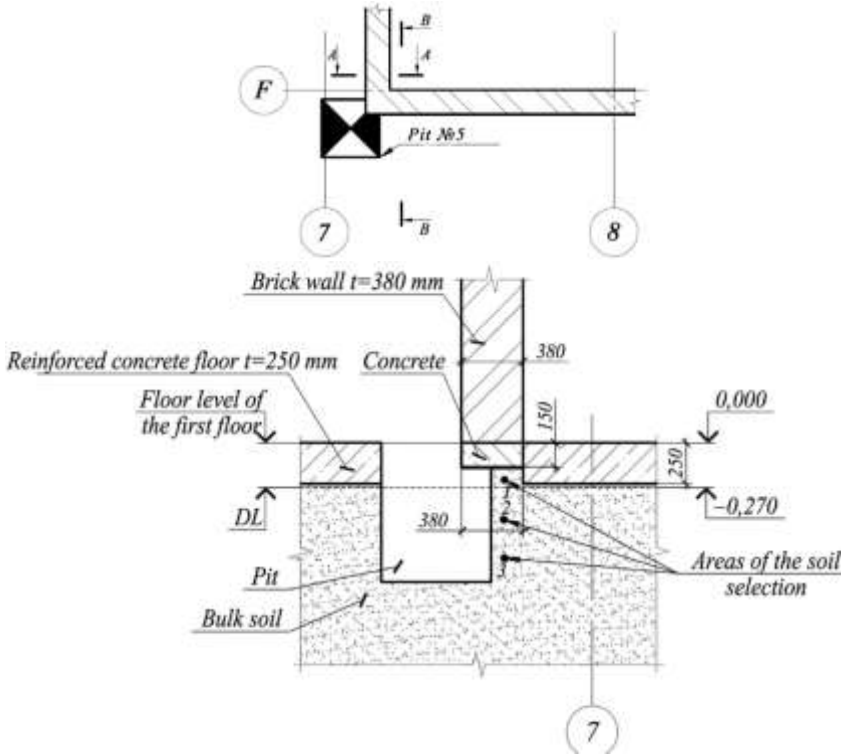

Fig. 6. Scheme of the layout of the pit No. 5 and section A–A

The column foundations for the columns of the frame and the foundation beams for the self-supporting exterior walls are made of monolithic reinforced concrete of a strength class of at least B20. The foundation for the walls of the staircase block is made in the form of concrete floors on the ground. The depth of the soles of the columnar foundations from the level of the planning mark is 1.05–2.65 m.

According to the opened pits No. 1, 3, 4, horizontal waterproofing is performed on top of the foundation beams from a layer of the roofing material. There is no waterproofing of the foundations and floors on the ground. The floors of the ground floor rooms are made of monolithic concrete 150–400 mm thick. No foundation defects have been identified. The concrete strength of columnar foundations and foundation beams, measured by means of non-destructive testing methods, corresponds to class B20 with a calculated compressive strength of R_b = 11.5 MPa.

In the pit No. 1 sandy bulk soils at a depth of 0.1–0.6 m from the base of the foundation have the following average values of the physical characteristics: soil density ρ = 18.8 kN/m^3; humidity w = 0.184; dry soil density ρ_d = 15.9 kN/m^3; soil particle density ρ_s = 26.7 kN/m^3; porosity coefficient e = 0.67; degree of water saturation S_r = 0.73. The granulometric composition is > 2 mm — 4.8%; > 0.5 mm — 8.2 %; > 0.25 mm — 24.7 %; > 0.1 mm — 46.7 %; < 0.1 mm — 15.6%. The underlying bulk soils are represented with high-quality dark gray-yellow fine sands of medium density, moist, clayey with inclusions of red brick crumbs, lime mortar and limestone rubble stone.

The minimum normative values of the strength and deformation characteristics of the underlying soils (according to Appendix B of SP 22.13330.2011) are as follows: specific adhesion a_s = 1 kPa; angle of internal friction φ_n = 31°; modulus of deformation E = 26 MPa.

While calculating the bases of the foundations for Group II of limiting conditions, the standard values of long-term loads were used. At the same time, for useful short-term loads on floors and from snow, long-term parts were taken by multiplying by reduction coefficients 0.35 and 0.7, respectively. While performing verification calculations of foundations, the calculated resistance of the bases of the foundations, the permissible load on the foundations, operating or design pressure on the soles of the foundations taking into account the accepted useful load were established [5,6,7] [15–21].

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[6] Rimshin VI, Ketsko ES, Truntov PS *A Large Construction Dictionary. Volume 2 P–Ya.* Moscow: ASV; 2022. 626 p.
[7] Rimshin VI, Ketsko ES, Truntov PS *A Large Construction Dictionary. Volume 1 A–O.* Moscow: ASV; 2022. 572 p.

In the pit No. 1 in the 9/A axes with the overall dimensions of the columnar foundation about 2.1×1.8×0.36(h) m and a depth of about d = 1.55 m, the calculated resistance according to the minimum normative values of the strength characteristics of fine sands at a_s = 1 kPa and $\phi_n = 31^o$ is R = 0.023 MPa. The collection of the loads on the foundation in the pit No. 1 is shown in Table 1.

Table 1

Collection of the loads. Pit No. 1

Loads	$A_{гр}$, m²	$q_н$, MPa	$q_р$, MPa	$N_н$, kN	$N_р$, kN
Loads from the surfacing					
Constant					
Weight of the surfacing and roofing	13.5	0.0056	0.0066	75.6	89.1
Temporary					
Surfacing	13.5	0.0009	0.0013	11.9	17.0
Loads from the ceiling above the 1st floor					
Constant					
Weight of the ceiling, floors	13.5	0.0052	0.0061	70.2	82.4
Temporary					
Temporary useful loads and partitions	13.5	0.0012	0.0015	16.2	20.8
Total loads on the foundation					
Weight of the ceiling				173.9	209.3
Weight of the walls and columns				256.6	294.9
Weight of the foundation				117.4	131.8
Total loads				547.9	636.0
Area of the sole of the column foundation, m²					3.78
Base pressure from the normative loads, MPa					0.15

Research Results. Based on the results of the verification calculations, the calculated resistance of the base soils is 0.23 MPa. The maximum permissible load on the foundation, taking into account a certain calculated resistance of the soil of the base and the dimensions of the foundation, should not be over 0.23 MPa. The average pressure at the base of the foundation, taking into account the accepted regulatory ceiling load of 2 kN/m^2, is about 0.145 MPa. The requirement of clause 5.6.7 JV 22.13330.2011 is met.

Similar verification calculations were performed for the pits No. 2–4. The calculation results are shown in Table 2.

Table 2

Results of the verification calculations for the pits

No. of a pit	R, MPa	N_{max}, MPa	p, MPa	Normative load, kN/m²	Compliance with Clause 5.6.7 SP 22.13330.2011
1	0.23	≤ 0.23	0.145	2	Is met
2	0.13	≤ 0.13	0.175	2	Is not met
3	0.18	≤ 0.18	0.188	2	Is not met
4	0.28	≤ 0.28	0.195	4	Is met

R is the calculated ground resistance of the foundation, Mpa; N_{max} is the maximum permissible load on the foundation taking into account the identified calculated ground resistance of the foundation and the dimensions of the foundation; p is the average pressure along the base of the foundation taking into account the accepted regulatory load on the floors, MPa.

In the pit No. 5 in axes 7/F (Section A–A), with a base width of about 0.38 m and a foundation depth of about d = 0.15 m (the wall rests on a concrete floor about 150 mm thick), the calculated resistance is equal to (in according to Table B.9 of SP 22.13330.2011) R_0 = 180 kPa (soil dumps with no compaction at a humidity level of no more than 0.5), is R = 0.09 MPa. In accordance with Table B.9 of SP 22.13330.2011, the calculated resistance of the underlying bulk soils for a conditional foundation with a sole width of 1.0 m and a depth of 2.0 m is R_0 = 180 kPa. The collection of the loads on the foundation in the pit No. 5 (section A–A) is shown in Table 3.

Table 3

Collection of the loads. Pit No. 5 (Section A–A)

Loads	$A_{гр}$, m²	$q_н$, MPa	$q_р$, MPa	$N_н$, kN	$N_р$, kN
Loads from the staircase structure					
Constant loads					
Weight of the staircase structure	2.8	0.0035	0.0039	9.8	10.8
Temporary loads					
Temporary useful	2.8	0.0011	0.0013	2.9	3.5
Total loads on the foundation, MPa					
Staircase structures				9.8	10.8
Weight of the wall				36.3	39.9
Weight of the foundation				1.4	1.6
Temporary useful				2.9	3.5
Total load				50.5	55.8
Width of the sole of the column foundation, m					0.38
Normative load pressure on the foundation, MPa					0.13

In the pit No. 5 in axes 7/F (Section B–B), with a base width of about 0.25 m and a foundation depth of about d = 0.15 m (the wall rests on a concrete floor about 150 mm thick), the calculated resistance is equal to (in according to Table B.9) R_0 = 180 kPa (soil dumps with no compaction at a humidity level of no more than 0.5), is R = 0.09 MPa. The collection of loads on the foundation in the pit No. 5 (section B–B) is shown in Table 4.

Based on the results of the verification calculations, the following has been established. In section A–A, the calculated resistance of the base soils is 0.09 MPa. The maximum permissible load on the foundation, taking into account a certain calculated resistance of the soil of the base and the dimensions of the foundation, should not be over 0.09 MPa. The average pressure at the base of the foundation, taking into account the accepted standard load on the staircase of 3 kN/m^2, is about 0.13 MPa. The requirement of Clause 5.6.7 SP 22.13330.2011 is not met.

Table 4

Collection of the loads. Pit No. 5 (Section B–B)

Loads	$N_н$, kN	$N_р$, kN
Weight of the wall	23.4	25.8
Weight of the foundation	0.9	1.0
Total load	24.3	26.8
Width of the sole of the column foundation, m		0.25
Normative load pressure on the foundation, MPa		0.0975

In section B–B, the calculated resistance of the base soils is 0.09 MPa. The maximum permissible load on the foundation, taking into account a certain calculated resistance of the soil of the base and the dimensions of the foundation, should not be over 0.09 MPa. The average pressure at the base of the foundation is about 0.1 MPa. The requirement of Clause 5.6.7 SP 22.13330.2011 is not met.

As there are loose soils with a low design resistance (below the design pressure) in certain sections of the building at the base of the foundations, the condition of the foundations for the columns of the frame and the walls of the staircase block is characterized as unsatisfactory — Category III (limited operability). According to the results of the technical inspection, given the results of the verification calculations (Table 5) it can be concluded that for the further normal operation of the building in some areas, it is necessary to take measures in order to strengthen the foundations or the bases of the foundations.

Table 5

Results of the verification calculations of the soils of the foundations and structures of the foundations

Location of the foundation	Load-bearing capacity of the foundation structures based on the material, MPa	Design resistance of the foundation soils, MPa	Permissible load on the foundation soils, MPa	Existing load on the foundation, MPa	Coefficient of the use for the soil	Coefficient of the use for the foundation structures
Column foundation in the pit No. 1 in the axes 9/A	Determined by the reinforcement of the structures	0.23	0.23	0.145	0.63	Determined by the foundation soil
Column foundation in the pit No. 2 in the axes 5/D	Determined by the reinforcement of the structures	0.13	0.13	0.175	1.35	Determined by the foundation soil
Column foundation in the pit No. 3 in the axes 1/Zh	Determined by the reinforcement of the structures	0.18	0.18	0.188	1.04	Determined by the foundation soil
Column foundation in the pit No. 4 in the axes 1/A	Determined by the reinforcement of the structures	0.28	0.28	0.195	0.7	Determined by the foundation soil
Ribbon foundation in the pit No. 5 in the axes 7/E (Section A–A)	1.5	0.09	0.09	0.133	1.48	0.1
Ribbon foundation in hte pit No. 5 in the axes 7/E (Section B–B)	1.5	0.09	0.09	0.1	1.09	0.07

Discussion and Conclusion. A comprehensive technical inspection of the building by means of modern methods of non-destructive testing and verification calculations has revealed the current technical condition of the structural elements and foundations.

Based on the results of the technical inspection, it can be concluded that the condition of the building is deemed satisfactory — Category II (operational), except the foundations under the columns of the frame and the walls of the staircase block, with its condition characterized as unsatisfactory — Category III (limited operational).

It has been found that in order to ensure the normal operation of the building in areas with bulk soils (pits No. 2, No. 3, No. 5), it is necessary to take measures to strengthen the foundations or bases. Ribbon foundations for the walls of a staircase block can be reinforced by means of widening the sole or strengthening the foundations of the bases, column foundations for columns — by means of installing a column foundation or strengthening the foundations of the bases. It is recommended that the base of the foundations of the walls of the staircase block is widened by means of installing a monolithic reinforced concrete cage (at least 300 mm wide) or a liner. The final overall dimensions of reinforcement structures should be identified by means of calculations. The pile base for the frame columns can be made of vertical drilling piles. A joint operation of piles and foundation structures should be ensured by tying the reinforcing frames of piles with foundation structures and their subsequent concreting. Base soils can be strengthened by injecting cement or special mortars into the soil.

The above method of comprehensive assessment of operational safety in various technical conditions of building foundations increases the forecast of operability, load-bearing capacity and durability and is applicable to similar structures.

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A Combined Finite Element Analysis and Artificial Neural Network Approach for Diagnostics of Building Cross-Sections Weakened with Stress Concentrators

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Abstract

Introduction. The study is dedicated to developing a new method for identifying defects in building structures with stress concentrators. The method is based on the integration of shadow ultrasonic testing with deep learning algorithms, which would enable accurate diagnosis with reliably identifying the geometric characteristics of defects.

Materials and Methods. A finite element model of an area with an angular point and damping layers made of metal with a flexible coating was used. An ultrasonic actuator and receiver were placed on the opposite edges. Numerical experiments with changes in the geometry and materials of the area and defect parameters were conducted on a distributed computing system. The resulting signals were converted into spectrograms which were used in order to train a convolutional neural network that establishes a connection between the spectrogram and the defect parameters.

Results. An extensive dataset of spectrograms has been formed. The trained neural network has displayed the ability to accurately identify the key defect parameters based on a spectrogram such as size, position, and orientation. Verification of the method has shown that it outperforms the traditional methods of ultrasonic signal analysis in terms of its accuracy and speed.

Discussion and Conclusion. The hybrid approach for non-destructive testing in complex geometric conditions has been proven to be effective. The major advantage is automated and intelligent data analysis reducing a degree of subjectivity. The practical significance is the creation of a prototype adaptive diagnostic system. Prospects are related to further training on experimental data and integration into portable systems for monitoring structures.

Keywords: building structures, non-destructive testing, ultrasonic diagnostics, stress concentrator, shadow method, deep learning, convolutional neural network, finite element modeling, defect identification, spectrogram

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Оригинальное эмпирическое исследование

Комбинированный подход конечно-элементного анализа и искусственных нейронных сетей для диагностики сечений строительных конструкций, ослабленных концентраторами напряжений

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Аннотация

Введение. Исследование посвящено разработке нового метода идентификации дефектов в строительных конструкциях с концентраторами напряжений. Метод основан на интеграции теневого ультразвукового контроля с алгоритмами глубокого обучения, что позволит достичь точной диагностики с достоверным определением геометрических характеристик дефектов.

Материалы и методы. Использовалась конечно-элементная модель области с угловой точкой и демпфирующими слоями из металла с гибким покрытием. Ультразвуковой актуатор и приемник размещались на противоположных гранях. На распределенной вычислительной системе проведены численные эксперименты с варьированием геометрии и материалов области и параметров дефектов. Полученные сигналы преобразованы в спектрограммы, которые использовались для обучения сверточной нейронной сети, устанавливающей связь между спектрограммой и параметрами дефекта.

Результаты исследования. Сформирован обширный датасет спектрограмм. Обученная нейронная сеть продемонстрировала способность с высокой точностью определять по спектрограмме ключевые параметры дефекта: размер, положение и ориентацию. Верификация метода показала, что он превосходит по точности и скорости традиционные методы анализа ультразвуковых сигналов.

Обсуждение и заключение. Подтверждена эффективность гибридного подхода для неразрушающего контроля в сложных геометрических условиях. Основное преимущество — автоматизированный и интеллектуальный анализ данных, снижающий субъективность. Практическая значимость — создание прототипа адаптивной диагностической системы. Перспективы связаны с дообучением на экспериментальных данных и интеграцией в портативные комплексы для мониторинга конструкций.

Ключевые слова: строительные конструкции, неразрушающий контроль, ультразвуковая диагностика, концентратор напряжений, теневой метод, глубокое обучение, сверточная нейронная сеть, конечно-элементное моделирование, идентификация дефектов, спектрогра мма.

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. The fundamental scientific task this study is aimed at addressing is to investigate the problems of strength and reliability of critical elements of building structures taking into account internal stress concentrators (defects). In order to tackle this research problem, an integrated approach has been implemented consisting in the following. At the first stage, the sections of structural elements are diagnosed in order to identify technological or operational defects. At the second stage, the problems of deformable solid mechanics are solved for these elements taking into account the identified defects, and conclusions are made as regards their operability [1, 2].

The initial formulation of the problem at the first stage is analyzing forced oscillations of a region with an angular point within the framework of elasticity theory described by the corresponding system of equations:

$$\sigma_{ij,j} = \rho \ddot{u}_i,$$
$$\sigma_{ij} = c_{ijkl} u_{k,l}.$$

The desired functions are the components of the displacement vector u_i (i = 1, 2, 3), the system of equations relates them to the stress tensor through elastic constants c_{ijkl} and material density ρ, complimented by the boundary conditions

$$u_i|_{S_u} = u_i^{(0)}$$
$$\sigma_{ij} n_j|_{S_t} = p_i$$
$$\sigma_{ij} n_j|_{S_d} = q_i$$

The boundary conditions are identified on surfaces S_u (specified by the displacements $u_i^{(0)}$), S_t (specified by the loads p_i, q_i, taking into account the normal n_i) and S_d representing the surface of the crack rupture itself in a direction parallel to the axis of the application.

The detection of defects, which include cracks and impurities, is an inverse geometric problem in the mechanics of a deformable solid [3], where it is required to identify the contours of defects, including their shape, location and size, which directly determines the boundaries of the shores S_d.

In the initial formulation, it is assumed that the crack banks are not in contact with each other and are stress-free, which corresponds to the condition

$$q_i = 0.$$

The inverse problem of identifying the boundaries of a defect cannot be solved solely on the basis of boundary conditions due to its incorrectness. In order to ensure the stability of the solution, additional experimental data is being used – the amplitude-time characteristics (ATCs) of the displacement field $\bar{U}$ recorded on a free surface.

$$X: u_i = U_i(\bar{x}_j, t).$$

Here t is the duration of the time interval [0, T] of the signal recording, N is the total number of the control nodes $\bar{x}_j$ on the surface S_t where ATCs were being measured.

Materials and Methods. The paper analyzes a model (Fig. 1) of an area with an angular point with a coating that contains an internal defect. The location areas of the actuator and sensors are indicated.

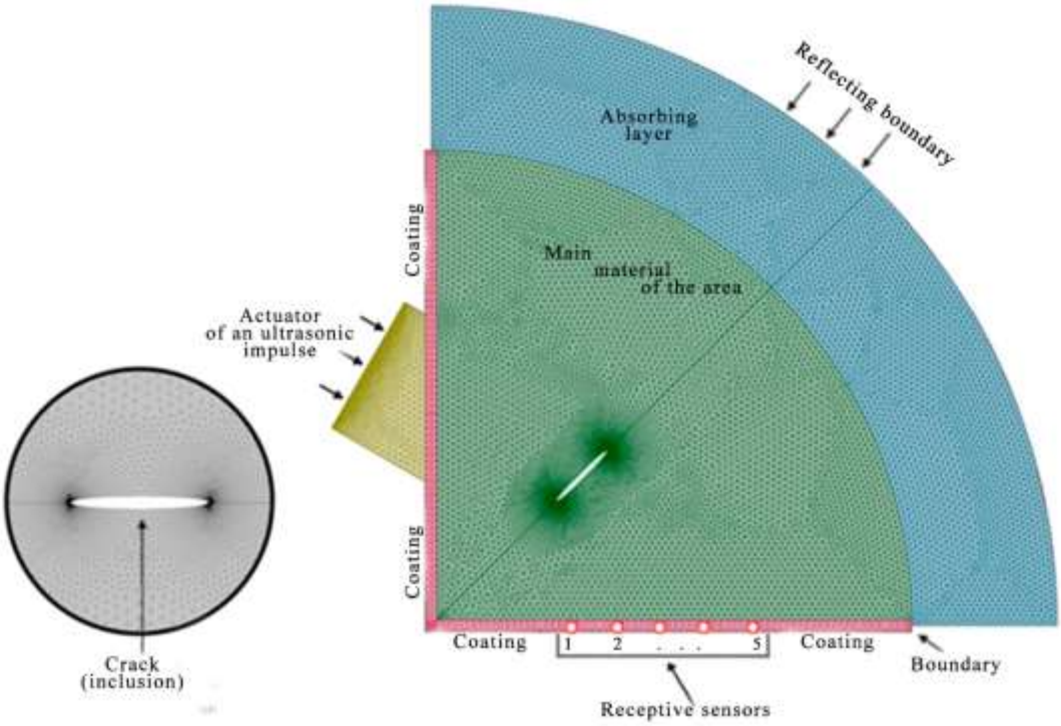


Fig. 1. Investigated model of a structure with an angular point

The ANSYS Mechanical application software package version 2023 R1 was used for numerical modeling, which allows simulations of complex physical phenomena including structural mechanics [4].

The Explicit Dynamics package module serves as a tool for modeling elastic wave propagation, demonstrating a high level of accuracy in complex and heterogeneous materials. An important element of such calculations are special absorbing layers that are used in finite element analysis to implement non-reflective boundary conditions and minimize parasitic reflections of waves. They are designed in order to approximate an infinite region by simulating the behavior of waves when they interact with an infinite medium. The main functional aim of the absorbing layers is to suppress reflections of wave energy from artificial boundaries of the computational domain, which can lead to non-physical results and disruption of the stability of the solution.

From a methodological standpoint, the approach used is based on the approximation of the desired functions using piecewise polynomial basis functions. The algorithmic implementation of the method is focused on performing calculations for models with a significant number of degrees of freedom [5]. The basic dependent variables defining the mathematical model of the module are velocity and deformation. The Rayleigh damping model is used in order to mathematically describe dissipative processes.

The method is based on solving a system of equations describing the behavior of a linear elastic medium.

$$\rho \frac{\partial v}{\partial t} - \operatorname{div} S = F_v$$

$$2\frac{\partial E}{\partial t} - [\nabla v + (\nabla v)^T] = 0$$

$$S = C{:}\,\epsilon$$

The system describes the law of conservation of momentum (equation of motion), where v is the vector of the velocity; ρ is the density of the material; S is the Cauchy stress tensor; S is the resulting superficial force; F_v is the vector of external volumetric forces; a kinematic ratio relating the rate of change of deformations (ϵ is the deformation tensor) with the velocity gradient and generalized Hooke's law (C is the elasticity tensor).

The propagation velocities of elastic waves in a continuous medium are directly dependent on its mechanical properties, i.e., the Young's modulus (E) and the Poisson's ratio (v). This relation for the longitudinal velocities (c_p) and transverse (c_s) waves is mathematically expressed with the following ratios:

$$c_p = \sqrt{\frac{E(1-v)}{\rho(1+v)(1-2v)}}, c_s = \sqrt{\frac{E}{2\rho(1+v)}}.$$

The use of an explicit integration scheme calls for a special setting of internal boundary conditions in the area of the connection between the coating and the base material, particularly in the presence of angular points. These conditions are formulated taking into account a surge in the physical and mechanical characteristics at the contact of dissimilar materials. They ensure the continuity of the velocity field and the equality of the normal components of the voltage vector.:

$$v_1 - v_2 = 0,$$

$$(S_1 - S_2)n = 0,$$

where indices «1» and «2» denote the values of the parameters on the opposite sides of the interface and n is a unit vector of the normal to it.

An absorbing layer is used in the model to suppress unwanted reflections from the boundaries of the calculated area. Its algorithm combines use of the three methods: scaling coordinates, applying filters, and applying simple non-reflective conditions. In the model the cylindrical geometry of the absorbing layer was chosen due to its symmetrical location relative to the angular point of the area under study. The configuration of the absorbing layer in a cylindrical coordinate system ensures isotropic absorption of wave energy regardless of the direction of its propagation. This approach minimizes the effect of reflection of waves from the artificial boundaries of the computational domain returning their energy back to the domain.

In spite of the high efficiency of the absorbing layer technique for suppressing spurious boundary reflections, its application is faced with a few limitations. Some properties of the media being modelled might cause premature reflection of waves on internal inhomogeneities prior to their interaction with the damping layer. An additional factor is the increased computational costs, particularly expressed in tasks with a large number of degrees of freedom. Thus the decision on the implementation of absorbing layers calls for a preliminary analysis and is to be justified by the specific features of the problem at hand.

In order to ensure minimal reflection of wave disturbances from the geometry feature in the form of an angular point on the right boundary of the structure (Fig. 1), a special boundary condition is implemented. It minimizes wave reflection by matching the properties of the boundary with the medium enabling longitudinal and transverse waves to leave the modeling area unhindered. The combined application of this condition with the technique of absorbing layers forms a highly efficient absorption mechanism for most possible angles of incidence of waves.

Correct modeling of the propagation of ultrasonic waves in elastic media calls for adequate discretization of the computational domain [6]. The accuracy and stability of the solution are mostly determined by the parameters of the finite element grid which are to be in compliance with a number of criteria. The key requirement is to provide a spatial resolution sufficient to approximate the shortest wavelength component of the wave field. To this end, the maximum characteristic size of the grid element (h) must be less than the minimum wavelength of the wave being modelled (λ_{min}). This condition can be formalized with the following ratio:

$$h \le \lambda_{min} / n,$$

where n is the number of elements per wavelength determined with the order of the finite elements used and the required accuracy (as a rule, $n \ge 6$–10 for linear elements, the value $n = 5$ is normally used).

Special requirements are regarding the discretization of areas adjacent to the boundaries of the materials. In order to describe the processes of reflection and wave passage in a correct way, a local increase in the density of the grid is essential. A mandatory stage of the model preparation is verification of the grid quality. The critical parameters are the distortion level of the elements and their aspect ratio. Exceeding the permissible limits for these parameters might lead to a deterioration in the conditionality of the system of equations as well as numerical variance. The spatial sampling parameters are determined with the wave properties of the elastic medium, while the key factor is the mode with the highest phase velocity. The minimum wavelength is a critical parameter in the construction of the grid λ_p associated with this mode that sets the required spatial resolution. For the explicit integration scheme used in the Explicit Dynamics module, the use of higher-order approximation elements requires that the maximum size of the element is in compliance with the following criteria:

$$h_{max} = \frac{\lambda_{max}}{1,5} = \frac{c_{max}}{1,5 \cdot f_{max}},$$

where f_{max} is the upper limit of the frequency range to be resolved during modelling of wave processes.

The geometric parameterization of the mesh should adequately approximate the features of the structure, including the curved boundaries of an elliptical defect localized in the vicinity of an angular point. The details of the grid in the vicinity of the vertices of this defect are shown in Fig. 1 (the lower left fragment). The curvature coefficient settings used in the model ensure high-quality geometry approximation that is in compliance with the requirements of the applied numerical method.

The stability of the explicit integration scheme used in this work is identified using the Courant-Friedrichs-Levy criterion that sets a strict limit on the size of the calculated time step. The global time step is limited by the minimum value in the computational domain of the ratio of the characteristic size of the final element to the propagation velocity of the fastest volume wave c_p . Thus, the permissible time interval is determined by the smallest size of the grid element.

The spatial discretization criteria for each component of the composite structure are established based on the dispersion characteristics of the material, particularly the propagation velocity of the dominant elastic mode. This makes it necessary to use an adaptive grid with an increased density of elements in areas of wave field concentration, i.e., the defect localization and acoustic input area.

Based on the calculated dependencies, the carrier frequency of the probing pulse was set at 0.5 MHz. The exciting signal is formed in the form of a sinusoidal fill modulated by the Henning window function and is set by the following boundary conditions:

$$Sn = F_A(t), F_A = -p(t)n.$$

The optimization of the computational process and modelling parameters made it possible to conduct the necessary number of numerical experiments within the limited time. Fig. 1 shows the geometric configuration of the computational domain whose parameters are provided in Table 1. The model includes a two-component structure consisting of an aluminum alloy coating and a steel substrate forming an area of angular singularity.

Research Results. In order to increase the stability of the solution, additional experimental data is used in the work — the displacement fields of the ATCs recorded in a free surface. The set of specified data allows a defect to be identified. The suggested approach, which integrates the shadow ultrasound method and deep learning algorithms, allows not only defects, but also the geometric parameters of areas with an angular point to be identified. This combined solution demonstrates a significant superiority in velocity, accuracy and reliability over the traditional methods of non-destructive testing [6].

In order to verify the accuracy and increase the reliability of the technique, a finite element model describing the propagation of ultrasonic waves has been developed. In order to minimize the distorting effect of parasitic reflections, special damping layers have been introduced into the model to absorb these signals and to prevent them from propagating in the area with an angular point. Based on this model, a shadow ultrasound scanning method is implemented, where the transducer and receiver are located on the opposite sides of an object being monitored.

In order to form a training sample in a distributed computing environment, a series of numerical experiments has been performed, suggesting a multivariate solution to problems with changing geometric parameters: location of a signal source and receiver, as well as configuration of an internal defect. The training sample for the neural network model is based on parametric finite element analysis. In compliance with the methodology in [7, 8], a balanced data set has been formed, including modelling results with variations in the geometric parameters of the system. The obtained data set was used in order to train a neural network model whose task was the binary classification of a defect and regression of its spatial characteristics.

The ranges of parameter variation included: the defect length — 5–25 mm (20 discrete values); defect width — 0.5–2.5 mm (20 values); defect position — 5–50 mm (20 values); actuator position — 0–80 mm (20 values).

Table 1

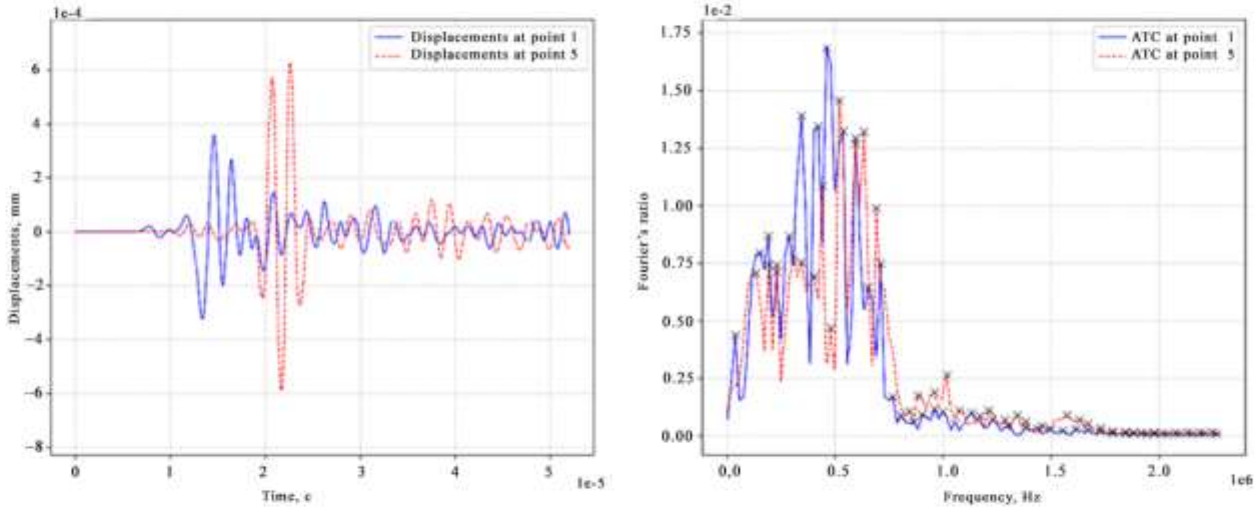
Main parameters of the finite element model made of steel with aluminum alloy coating

Parameter	Value
Opening angle of the area with an angular point	90 °
Coating thickness	1 mm
Size of the area with an angular point	150 mm
Crack length	20 mm
Crack thickness	2 mm
Crack distance from an angular point	30 мм
Actuator and receiver distance from an angular point	25 mm
Velocity of propagation of a longitudinal wave in the coating	6 197 m/sec
Velocity of propagation of a longitudinal wave in the body of the area with an angular point	5 778 m/sec
Coating density	2.7 g/cm^3
Density of the body of the area with an angular point	7.85 g/cm^3
Young's module of the coating	70 GPa
Young's module of the body of the area with an angular point	200 GPa
Poisson's ratio of the coating	0.33
Poisson's ratio of the body of the area with an angular point	0.29
Frequency of an ultrasonic signal	0.5 MHz

In compliance with the accepted methodology, intact models that did not contain defects were analyzed in 5% of cases. The elements of the training sample were represented by means of spectrograms generated using the fast Fourier transform (FFT) algorithm. The obtained spectrograms represent the frequency content of the signals and are used as input data for training a neural network model.

The joint registration of acoustic responses in five measuring positions (Fig. 1) allowed us to form a representative database, including 15,500 implementations for samples with defects and 780 implementations for reference samples with no defects. The physical nature of the phenomenon under study excludes the possibility of making use of the input data augmentation algorithms. The adequacy of the model was checked on a control subsample that made up 20% of the total experimental data.

The temporal realizations of acoustic signals received at the monitoring points are shown in Fig. 2*a*, and their spectral characteristics are shown in Fig. 2*b*. The target variables of the model are geometric descriptors of the defect and a binary classifier for its detection. This set of features is used in order to design the training vectors fed to the input of a neural network architecture.



a) *b)*

Fig. 2. Acoustic signal analysis: *a* — amplitude-time characteristic (time realization); *b* — amplitude-frequency characteristic (spectrum)

Fig. 3 shows the spectrograms of the signals recorded at various control points on the surface of the area with an angular point. The recording was carried out with synchronous movement of the source and receiver of ultrasonic vibrations along the edges of the area under study. Visual analysis of spectrograms demonstrates their significant informative value for solving the problem of defect identification.

The analysis of the spectrograms at control points 1–5 reveals a pronounced acoustic shadow area resulting from the presence of an internal defect. The spatial and temporal features of the propagation of wave fields recorded during the experiment are the basis for creating a training dataset of a neural network architecture.

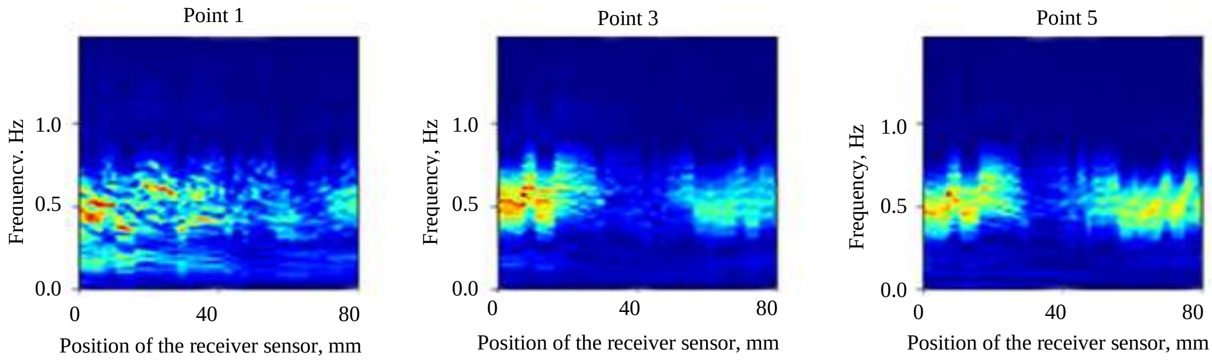


Fig. 3. Spectrograms of a signal received at 3 different points on the surface of the area with an angular point

The paper presents the architecture of a convolutional neural network (CNN) designed for image classification [5]. The network model accepts two-dimensional pixel intensity matrices with a size of 60×120 as the input. The input data is transformed by means of a three-stage hierarchical structure. At each stage of this hierarchy, a convolutional operation is applied followed by a nonlinear transformation using the ReLU function, and then a subsampling operation to reduce the spatial dimension of the generated features. The number of filters in the convolutional layers is consistently reduced from 64 to 32 and 16, respectively, which provides a hierarchical allocation of features of different levels of abstraction. After the convolutional layers, the spatial data structure is transformed in order to ensure compatibility with fully connected layers. The final stage of processing is a fully connected hidden layer containing 512 neurons with ReLU nonlinearity that is trained in order to identify complex dependencies between high-level features. The fully connected layer, also known as the dense layer, is an essential component of the CNN. These layers follow the convolutional and unifying layers in the CNN architecture and serve to combine the information extracted by the convolutional layers into a single output signal.

The output layer of the network is formed by five neurons and a Softmax activation function that provides a probabilistic distribution across target classes. Each neuron corresponds to a separate characteristic of the defect: two of them implement a binary classification of the fact of the defect in the control area, whereas the other three are regression neurons encoding, respectively, the length of the defect, the width of the crack and the coordinate of its position relative to the vertex of an angular point. The total number of trainable parameters of the suggested architecture is 559,285.

The mean square error (MSE) was chosen as the optimization objective function. The experiments have shown that satisfactory model quality is achieved following 30 training epochs. The stabilization of the learning process is ensured by a balanced composition of the training sample and use of batch normalization layers [9] that accelerate convergence and act as a regularizer reducing the risk of overfitting.

An experimental assessment of the operability of the trained neural network model was carried out on a control sample with a capacity of 2,500 samples. The input space of the model consisted of the frequency characteristics generated during finite element modeling of the ultrasonic flaw detection process. The output interface of the model combined a defect detector (binary classification) and predictors of its spatial characteristics (regression problem).

Discussion and Conclusion. The accuracy of the regression estimation of the crack width was analyzed by means of the suggested model (Fig. 4). The analysis of dependencies showed that as the crack width increases, so does the prediction accuracy subsequently stabilizing in the upper part of the range under study. The calculation of the mean square error for this parameter demonstrated that its value is not over 5% for the opening angle of the controlled area $\alpha = 60°$ and 10 % — for $\alpha = 120°$.

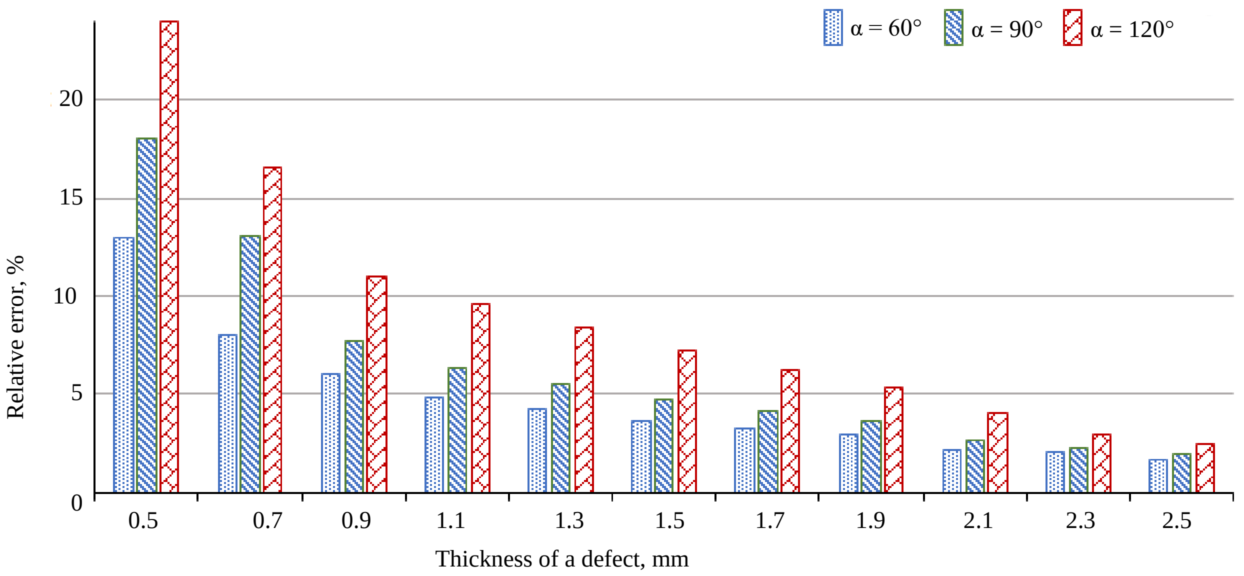


Fig. 4. Error in identifying the thickness of the defect at α = 60°, α = 90° and α = 120°

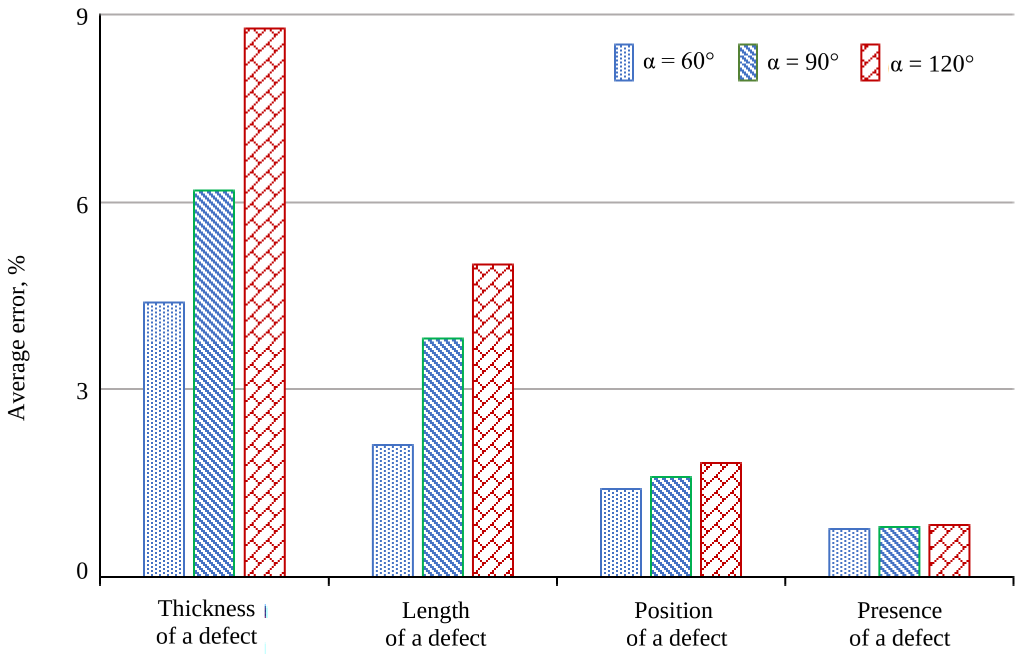


Fig. 5. Average errors of the CNN for identifying the defect characteristics for different angles of the model area opening (60°, 90° and 120°)

The obtained data confirm the prospects of using ultrasonic monitoring models using high-frequency probing signals, as well as with alternative schemes of acoustic transducers, which opens up avenues for further improvement of the accuracy of identifying geometric parameters of defects.

The analysis demonstrates the limited effect of the opening angle of the controlled area (α) on the accuracy of the neural network model (Fig. 5).

The statistical analysis has enabled us to find out that the change in the angle α is not the dominant source of error. Its contribution to the total error in identifying the coordinates of the defect relative to the vertex of the angle is significantly lower compared to the other factors considered. The highest accuracy of the technique is attained for the small values, which confirms its effectiveness and high reliability in the control of structural elements with an angular point.

An increase in the accuracy of the neuromodel has been attained by processing the input data and further training it on the synthesized numerical experiments [10–12]. The developed method demonstrates a high degree of reliability in detecting defects in areas with an angular point with coatings with a capacity to continuously improve diagnostic characteristics. Combining the shadow ultrasound scanning method with deep machine learning technologies, the technique provides a faster and more reliable tools of detecting defects and has prospects to be applied in a broad range of areas [7].

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Applicability of Non-Destructive Methods for Assessing the Strength of Masonry of Existing Structures

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Abstract

Introduction. The compressive strength of masonry is the most important mechanical characteristic assessed during inspections of buildings and is determined based on direct testing of bricks and mortar selected from the walls. However, current regulatory documents also recommend the use of non-destructive testing methods, particularly while examining cultural heritage buildings. However, non-destructive methods fail to take into account differences in the strength of bricks and mortar joints in the surface layers and their main volume, interaction of bricks and mortar, as well as anisotropy of the mechanical characteristics of masonry. The article presents the results of a study of the anisotropy of the compressive strength of ceramic bricks and an analysis of its influence on the results of assessing the strength of masonry using indirect methods.

Materials and Methods. The object of the research are two types of bricks: historical bricks from the walls of the barracks of the Brest Fortress built in 1933, as well as modern bricks produced in the Republic of Belarus. The strength of the brick was identified under a compressive load in the direction of the front, support and end surfaces on cubes with an edge size equal to the height of the brick.

Research Results. Graphs of the ratio between the obtained strength values of experimental cube samples under compression perpendicular to the front and end surfaces of the brick to the compressive strength perpendicular to its supporting surface are presented. Similar studies by other authors, including on ceramic cylinder samples, are analyzed. It was found that the compressive strength of the historical bricks perpendicular to its front and end surfaces was higher than their compressive strength perpendicular to the supporting surface. The opposite pattern was observed for modern bricks. However, due to the high variation of the results, it is not possible to establish a correlation between the compressive strength and the direction of the compressive force.

Discussion and Conclusion. The results of some studies that have shown that ceramic bricks are an anisotropic material are presented. A possibility of using non-destructive testing methods for brick strength has been evaluated, as well as that of designing a calibration ratio linking the compressive strength of a brick with the results of indirect testing of the front surface.

Keywords: masonry, brick, compressive strength, non-destructive testing methods, calibration ratio

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Оригинальное эмпирическое исследование

Применимость неразрушающих методов для оценки прочности каменной кладки существующих конструкций

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Аннотация
Введение. Прочность каменной кладки на сжатие является важнейшей механической характеристикой, оцениваемой при обследовании зданий, которая определяется на основании прямых испытаний отобранного из стен кирпича и раствора. Однако действующие нормативные документы также рекомендуют использовать неразрушающие методы контроля, особенно при обследовании объектов культурного наследия. Но неразрушающие методы не учитывают различия в прочности кирпича и растворных швов в поверхностных слоях и их основном объеме, взаимодействие кирпича и кладочного раствора, а также анизотропию механических характеристик кладки. В статье приведены результаты исследования анизотропии прочности на сжатие керамического кирпича и анализ ее влияния на результаты оценки прочности каменной кладки косвенными методами.
Материалы и методы. Объектом исследований является два типа кирпича: исторический кирпич из стен здания казармы Брестской крепости, построенного в 1933 г., и современный кирпич, произведенный в Республике Беларусь. Прочность кирпича определялась при действии сжимающей нагрузки в направлении лицевой, опорной и торцевой поверхностей на кубах с размером ребра, равным высоте кирпича.
Результаты исследования. Приведены графики отношения полученных значений прочности опытных образцов-кубов при сжатии перпендикулярно лицевой и торцевой поверхностям кирпича к прочности на сжатие перпендикулярно его опорной поверхности. Проведен анализ подобных исследований других авторов, в том числе на керамических образцах-цилиндрах. Установлено, что прочность исторического кирпича при сжатии перпендикулярно его лицевой и торцевой поверхностям оказалась выше прочности при сжатии перпендикулярно опорной поверхности. Для современного кирпича наблюдалась обратная закономерность. Однако из-за высокого разброса полученных результатов невозможно установить корреляционную зависимость прочности на сжатие от направления сжимающего усилия.
Обсуждение и заключение. Приведены результаты исследований, которые показали, что керамический кирпич является анизотропным материалом. Выполнена оценка возможности применения неразрушающих методов контроля прочности кирпича, а также возможность построения градуировочной зависимости, связывающей прочность на сжатие кирпича с результатами косвенного испытания лицевой поверхности.

Ключевые слова: каменная кладка, кирпич, прочность на сжатие, неразрушающие методы контроля, градуировочная зависимость

Для цитирования. Деркач В.Н., Демчук И.Е., Матяс П.И. Применимость неразрушающих методов для оценки прочности каменной кладки существующих конструкций. *Современные тенденции в строительстве, градостроительстве и планировке территорий*. 2026;5(2):32–40. https://doi.org/10.23947/2949-1835-2026-5-2-32-40

Introduction. The most important mechanical characteristic of masonry evaluated during inspection of stone buildings is its compressive strength. The compressive strength of masonry is commonly identified by means of calculating the test results of bricks and mortar taken from the walls of a building carried out in compliance with GOST 8462 "Wall Materials. Methods for Identifying Compressive and Flexural Strength Limits" and GOST 5802 "Building Mortars. Test Methods" [1–5]. This method is rather labor-intensive, thus while examining stone structures in order to identify the strength of bricks and mortar, lots of specialists tend to resort to non-destructive (indirect) methods that include sclerometric methods of elastic rebound, shock pulse, or the method of measuring the propagation velocity of an ultrasonic pulse. While making use of sclerometric methods, the compressive strength of bricks and mortar is identified using IPS MG-4 type sclerometers or Schmidt hammers where the calibration ratio "ceramic brick" is specified by the manufacturer. It is to be noted that in order to assess the strength characteristics of the brickwork of cultural heritage sites, GOST R 55567 "Procedure for Organizing and Conducting Engineering and Technical Research at Cultural Heritage Sites. Historical and Cultural Monuments. General Requirements" directly recommends prioritizing non-destructive testing using devices based on the elastic rebound method in compliance with GOST 24332 "Silicate Bricks and Stones. Ultrasonic Method for Identifying Compressive Strength" or other specialized and calibrated devices for identifying the strength characteristics of bricks and mortar. A correlation between the indicators of non-destructive testing devices and strength characteristics of masonry materials can be specified by means of comparing the average values of these characteristics obtained by non-destructive testing and laboratory tests of at least three samples for each type of masonry. Laboratory tests are conducted in compliance with the requirements of GOST 8462, GOST 5802.

It is known that non-destructive methods for assessing the compressive strength of masonry which has been in use for a long time have some drawbacks that are associated with uncertainties caused by the differences in the strength of bricks and mortar joints in the surface layers and their bulk, interaction of bricks and masonry mortar, as well as anisotropy of the mechanical characteristics of masonry [1, 3, 7, 9]. Overlooking these factors might cause significant errors in assessment of the compressive strength of masonry, and thereby underestimation or overestimation of the load-bearing capacity of stone structures. This article looks at the effect of anisotropy of the compressive strength of ceramic bricks on the strength of masonry identified by means of indirect methods.

Materials and Methods. The anisotropy of the compressive strength of ceramic solid bricks was investigated. Two types of bricks were tested: historical bricks taken from the walls of the Brest Fortress barracks built in 1933 as well as modern bricks produced in the Republic of Belarus. The historical brick had the following dimensions: length — 265 mm, width — 130 mm, height — 60 mm. Bricks produced in Poland in the second half of the 19th century correspond to these sizes [5]. The dimensions of the modern brick are as follows: length — 250 mm, width — 120 mm, height — 65 mm. Initially, the average values of the compressive and bending strength of the bricks were identified in compliance with GOST 8462 (Fig. 1).

a) *b)*

Fig. 1. Tests of the bricks: *a* — compression; *b* — bending

Based on the tests, it was found that the average value of the tensile strength of the historical bricks under compression is 16.9 MPa, in bending — 5.1 MPa, of the modern bricks — 17.3 MPa and 5.3 MPa, respectively.

In order to identify the strength of a ceramic brick under a compressive load in the direction of its front, support and end surfaces (Fig. 2), cubes with an edge size equal to the height of a brick were cut out of the bricks.

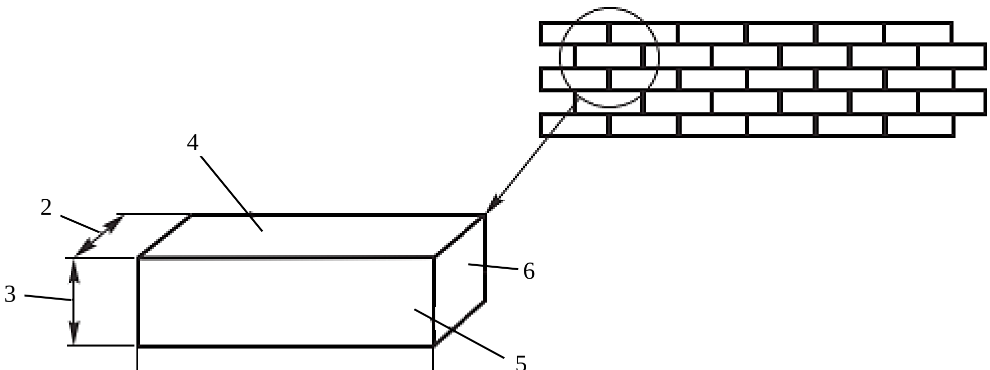


Fig. 2. Brick dimensions and surfaces: 1 — length; 2 — width; 3 — height; 4 — support surface; 5 — front surface; 6 — end/header surface

For each direction of the compressive load, 5-6 ceramic cube samples were made from different types of bricks. The surface of the cubes in contact with the press plates was leveled with a thin layer of gypsum mortar. The general view of ceramic cube samples prepared for the tests is shown in Fig. 3.

Fig. 3. Ceramic cube samples prepared for the tests

The ceramic cubes were loaded using a TP-1-500 testing machine. The cube samples were mounted with one of the selected faces on the lower base plate of the testing machine centrally relative to its longitudinal axis.

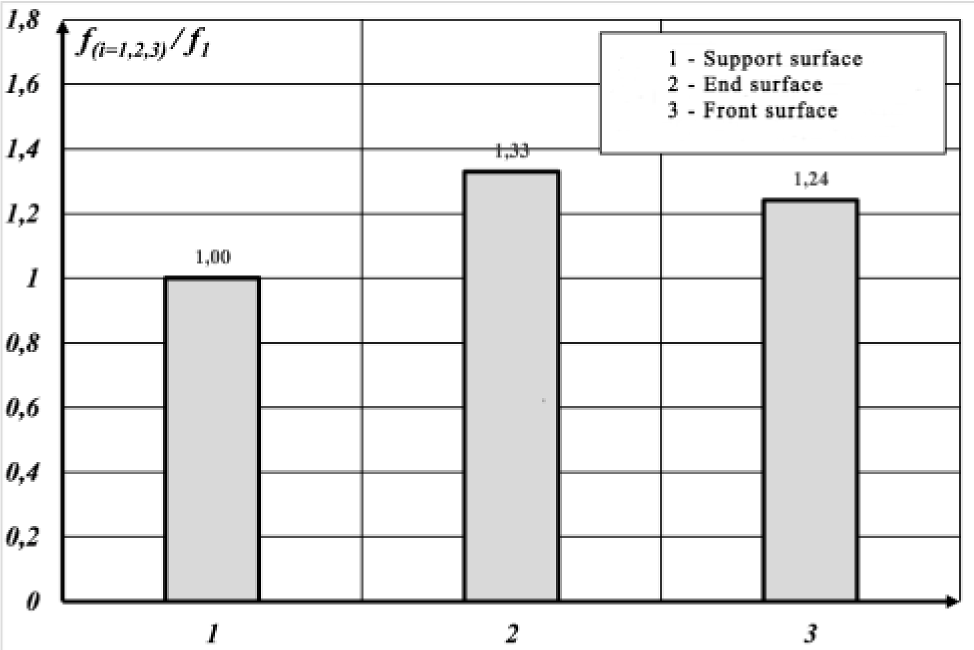
After the sample has been installed on the base plate, the upper plate of the machine was aligned with the upper support face of the sample so that their planes completely adjoined each other. The sample was loaded continuously at a rate that ensured its destruction within 30 seconds. The general appearance of the ceramic cube sample in the test facility and the nature of its destruction are shown in Fig. 4.

a) *b)*

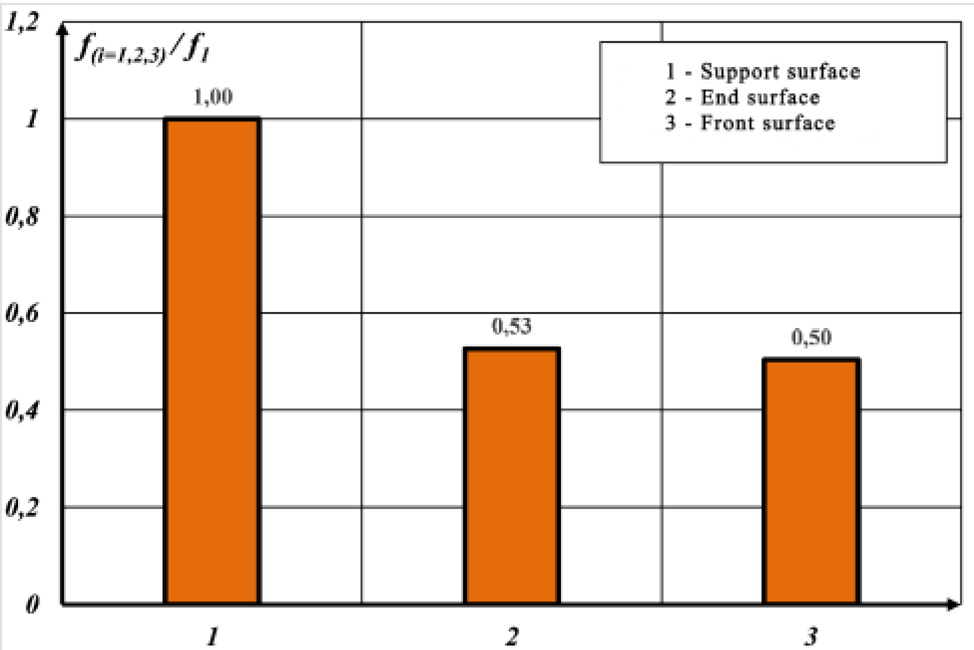
Fig 4. Testing of ceramic cubes: *a* — the general appearance of the sample; *b* — the nature of destruction

The maximum force attained during the tests was assumed to be a destructive load. The compressive strength of the sample was identified as the quotient of the destructive load divided by the working area of its cross-section. Based on the test results of the cube samples, the average strength values of the samples were identified under compressive forces in the direction of the front, support and end surfaces of the brick.

Research Results. Fig. 5, 6 show the graphs of the ratio of the resulting strength values of the experimental cube samples when compressed perpendicular to the front (f_3) and end (f_2) surfaces of the brick to the compressive strength perpendicular to its supporting surface (f_1).



a)



b)

Fig. 5. Results of identifying the strength of ceramic cube samples: *a* — historical brick; *b* — modern brick

According to Fig. 5, the strength of ceramic cube samples sawn from historical bricks when compressed perpendicular to its front and end surfaces turned out to be 24% and 33% higher, respectively, than that obtained by applying a compressive force perpendicular to the supporting surface.

The reverse pattern was observed for the modern bricks. The strength of ceramic cubes subjected to compressive loads perpendicular to the front and end surfaces of the brick was 50% and 47% lower than the compressive strength perpendicular to the support surface, respectively.

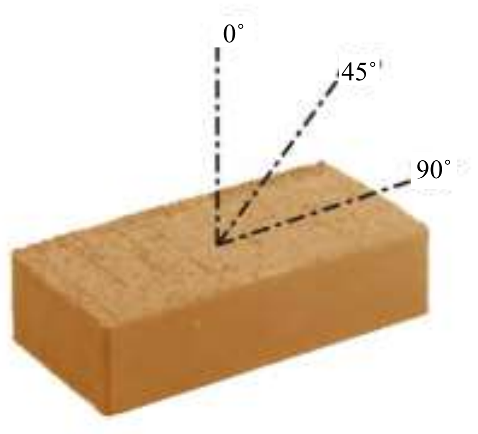
While testing the ceramic cube samples, the coefficient of variation in compressive strength was 25–40% for the historical bricks and 10–15% for the modern bricks.

The results of the studies of the anisotropy of compressive strength of the modern bricks are in a fairly good agreement with the test results of the ceramic cylinder samples provided in [7]. Cylinder samples with a diameter of 56 mm were selected from solid ceramic bricks of the grades M150 (Novgorod), M200 (Vitebsk) and M250 (St. Petersburg) perpendicular to its front and support surfaces. It was found that the ratio of the compressive strength of cylindrical samples taken orthogonally to the front and support surfaces of the brick was approximately 0.6.

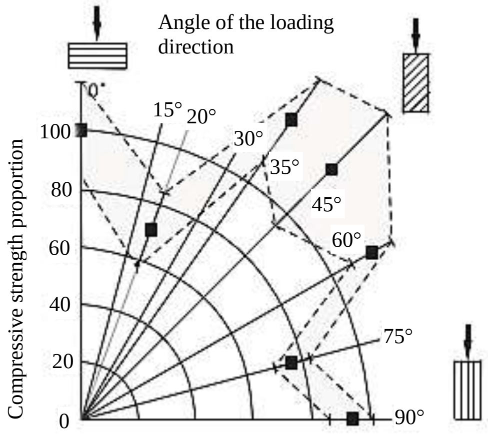
The test results of the cylindrical samples taken from the historical bricks were characterized by a significant range of values (the coefficient of variation of 30–45%) [8]. It was not possible to establish a ratio for them between the compressive strength orthogonally to the front and support surfaces of the brick. This is accounted for by the high heterogeneity of the material within a single brick, as well as the use of different types of bricks in the masonry.

[9] presents the results of studies of the anisotropy of compressive strength of historical ceramic bricks of the Austrian standard with a length of 290 mm, width of 150 mm and height of 65 mm, with a normalized compressive strength of 19.28 MPa and modern German NF standard bricks (length — 250 mm, width — 120 mm, height — 65 mm) with a normalized compressive strength of 28 MPa. The anisotropy of the compressive strength of the brick was identified based on tests of cores with a diameter of 45 mm selected at different angles to the supporting surface of the brick (Fig. 6*a*).

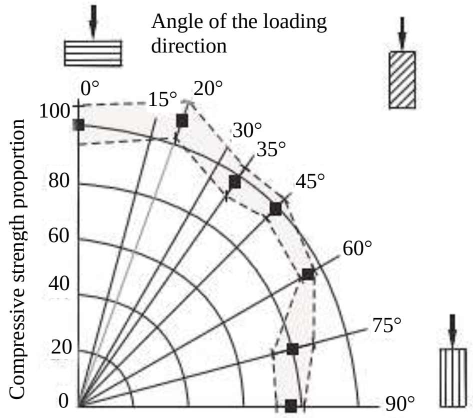
Studies have shown that for modern bricks when the compressive force is directed at an angle to the support plane of 0 ° < φ ≤ 60 °, the compressive strength is close to that perpendicular to the support plane, φ = 0° (Fig. 6*c*). The minimum values of compressive strength occurred at φ = 90 ° (f_0/f_{90} = 1.3).



a)



b)



c)

Fig. 6. Results of identifying the compressive strength of the cylinder samples: *a* — sampling directions; *b* — historical brick; *c* — modern brick [6]

For the historical bricks, under a compressive force at the angles of 35° ≤ φ ≤ 60°, the compressive strength turned out to be 20–25% higher than that at an angle φ = 0° (Fig. 6*b*). The minimum compressive strength was obtained at the angles φ = 20° and 75°. When a compressive force was directed perpendicular to the end surface of the brick, φ = 90°, the compressive strength was 23% lower than that at an angle φ = 0°.

Based on the test results, it was concluded that due to the high variation of the results for historical bricks, it is not possible to identify a ratio between the compressive strength and the direction of a compressive force.

Studies [6] have shown that while testing bricks in masonry, the amount of elastic rebound is affected not only by the direction of impact, but also by the quality of brick sealing in masonry, which is largely due to the type and condition of the mortar joints. This is accounted for by the influence of these factors on the degree of absorption of the impact energy of the test hammer on the brick surface.

Discussion and Conclusion. The results of this study as well as [7–9] have shown that ceramic bricks are an anisotropic material. The degree of anisotropy of the strength characteristics of a brick is due to a host of factors, with the technology of its manufacture and the raw materials used as the primary ones. At the same time, for historical bricks it is difficult to identify any patterns of compressive strength from the direction of a compressive force due to the high values of the coefficients of variation of the test results. It is not possible to design a calibration ratio connecting the compressive strength of such a brick in the direction of the support surface with the results of indirect testing of the front surface of the brick by means of the elastic rebound method or measuring the propagation velocity of an ultrasonic pulse.

While examining stone structures, tests to identify the elastic rebound of a hammer enable measurements of the surface hardness of the front surface of the brick, but not to evaluate its quality over the entire section and, which is more, to obtain compressive strength values in the direction of a compressive force acting in the section of a stone structure.

The velocity of propagation of an ultrasonic pulse in masonry is impacted a broad range of major factors: heterogeneity of the structure of bricks and mortar joints, thickness of the joints and quality of their execution, humidity of masonry, as well as degree of degradation of bricks and mortar [10].

The results of non-destructive testing methods are challenging for interpretation as the material in the surface areas of bricks and mortar might be different from their deeper layers. As an example, Fig. 7 shows a section of a ceramic modern brick where the different color of its outer and inner layers is clearly distinct, which indicates the difference in their strength characteristics. This is of particular relevance for historical bricks whose degree of firing and cross-sectional strength can vary considerably.

Fig. 7. Ceramic bricks in the section: different color of the outer and inner layers

It is recommended that indirect methods for assessing the compressive strength of bricks of existing structures are used in order to assess the uniformity of masonry as well as to identify the locations of brick and mortar selection. These methods can also be employed in order to control the strength of factory-made bricks where it is possible to design particular calibration ratios for each set of parameters (a type of raw material, molding method, temperature and duration of firing, a type of brick, etc.) [7]. At the same time, as noted in [6], use of sclerometric methods for assessing the strength of ceramic products is possible only if the volume of their voids is not over 10%. While testing masonry products with a large volume of voids, there is significant absorption of impact energy making it not possible to assess the strength of products in a reliable manner.

It is to be noted that the results of assessing the strength of the masonry obtained based on the tests of individual bricks and hardened mortar taken from its body have a low degree of reliability, as they fail to take into account the interaction of the brick and mortar in the masonry. The only methods that allow reliable data to be obtained on the compressive strength of masonry of existing structures are its structural tests or laboratory tests of masonry samples selected from a structure under study[1,2] [2, 3, 5, 9–14].

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СТРОИТЕЛЬНЫЕ КОНСТРУКЦИИ, ЗДАНИЯ И СООРУЖЕНИЯ

Original Empirical Research

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On the Issue of Determining the Design Lengths of Elements in Structural Constructions of the Kislovodsk Type

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Abstract.
Introduction. The article looks at the issue of determining the design lengths of rods of the structural construction of the Kislovodsk type. While examining the existing structure of the coating of the shopping center, in the design documentation for the object, the authors found that the design length coefficient equal to 0.8 was used in the calculations of the rod structure. The value of 0.8 is due to the results of the tests performed at Kucherenko Central Research Institute in 1987. However, according to the current regulations, a coefficient of design length equal to one is to be used. The aim of the article is to justify the value of the design length coefficient in calculations of such structures.
Materials and Methods. A standard unit of attaching rods to a nodal connector is considered. It was decided to study the work of both a separate structural unit and rods together by modeling in the Femap with Nastran software package. The paper considers the effect of a nodal joint on the stability of an individual rod. To this end, a nodal structural element with a support at the contact connection of a compressed rod and a calculation of a separate rod for stability was modelled. A system of rods is then considered in order to determine the critical force, taking into account the influence of adjoining elements. The scheme consists of four rods of the lower belt and four rods of the braces connected in one node and loaded with a compressive or tensile load.
Research Results. The considered separate joint of the rod with the connector, where compressive forces are transmitted through a tight contact, is very close to rigid fastening by the nature of the work when checking the stability of the rod. The design length coefficient for compressed elements ranges from 0.77 to 0.88, depending on the force values in the elements and their cross sections. Based on this, it can be unambiguously concluded that the application of a coefficient of 0.8 for all compressed rods cannot be theoretically justified.
Discussion and Conclusion. The coating studies show that the requirements of SP 16.13330.2011 "Steel Structures" are correctly applied in terms of determining the design length of a compressed rod equal to 1, and the use of a coefficient equal to 0.8 cannot be theoretically justified. Reducing the metal consumption while using reduced design length coefficients in calculations does not lead to significant savings for the overall structure and is not feasible.

Keywords: spatial lattice system, structural design, design length, stability

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Оригинальное эмпирическое исследование

К вопросу определения расчетных длин элементов в структурных конструкциях типа «Кисловодск»

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Аннотация

Введение. В статье рассмотрен вопрос определения расчетных длин стержней структурной конструкции типа «Кисловодск». При обследовании существующей конструкции покрытия торгового центра в проектной документации на объект авторами статьи выявлено, что в расчетах конструкции для стержней применяется коэффициент расчетной длины, равный 0,8. Значение 0,8 обусловлено результатами проведенных испытаний в ЦНИИСК им. Кучеренко в 1987 г. Однако действующие нормативные документы предписывают использовать коэффициент расчетной длины, равный 1. Целью данной статьи является обоснование значения коэффициента расчетной длины в расчетах подобных конструкций.

Материалы и методы. Рассмотрен стандартный узел крепления стержней к узловому коннектору. Принято решение об изучении работы как отдельного узла структурной конструкции, так и стержней совместно при помощи моделирования в программном комплексе Femap with Nastran. В работе рассматривается влияние узлового соединения на устойчивость отдельного стержня. Для этого выполнено моделирование узлового элемента конструкции с опиранием при контактном соединении сжатого стержня и расчет отдельного стержня на устойчивость. Затем рассматривается система стержней для определения критической силы с учетом влияния соседних элементов. Схема представляет собой четыре стержня нижнего пояса и четыре стержня раскосов, соединенных в одном узле и загруженных сжимающей или растягивающей нагрузкой.

Результаты исследования. Рассмотренный отдельный узел стыка стержня с коннектором, в котором сжимающие усилия передаются через плотный контакт, по характеру работы при проверке устойчивости стержня весьма близок к жесткому закреплению.

Коэффициент расчетной длины для сжатых элементов находится в пределах от 0,77 до 0,88 в зависимости от значений усилий в элементах и их сечений. Из чего можно сделать однозначный вывод о том, что применение коэффициента 0,8 для всех сжатых стержней не может быть теоретически обоснованно.

Обсуждение и заключение. Проведенные исследования покрытия показывают правильность применения требований СП 16.13330.2011 «Стальные конструкции» в части определения расчетной длины сжатого стержня равной 1, а использование коэффициента равного 0,8 не может быть теоретически обосновано. Снижение металлоемкости при использовании в расчетах пониженных коэффициентов расчетной длины не приводит к значимой экономии для конструкции в целом и не является целесообразной.

Ключевые слова: пространственная решетчатая система, структурная конструкция, расчетная длина, устойчивость

Для цитирования. Щуцкий С.В., Лиманцев А.А. К вопросу определения расчетных длин элементов в структурных конструкциях типа «Кисловодск». *Современные тенденции в строительстве, градостроительстве и планировке территорий*. 2026;5(2):41–48. https://doi.org/10.23947/2949-1835-2026-5-2-41-48

Introduction. Spatial lattice systems have been actively developed since the second half of the 20th century, and it was during this time that lots of different buildings and structures were designed that were covered with structural constructions[1] [1–3]. One of the most common lattice structures of pipe coating in Russia is the Kislovodsk type structure [4–5] developed as part of a state program and designed to solve the problems of organizing production and complete supply of light metal structures of industrial buildings.

The main property of both planar and spatial rod systems is that all elements are subject to a uniaxial stress-strain state (compression or stretching). Stretched elements are calculated for strength, and compressed elements are calculated for stability. In turn, the stability of an element directly depends on its calculated or reduced length. According to the regulatory documents, both in the current version and those in force in the past, while calculating the elements of spatial structural structures, the full geometric length of the rod is to be taken as the design length, or, in other words, a coefficient of

[1] Dykhovichny YuA, Zhukovsky EZ *Modern Spatial Structures (Reinforced Concrete, Metal, Wood, Plastics).* Moscow: Higher School; 1991. 543 p.

the design length equal to 1 should be applied (except in cases when the elements are welded in the nodes adjacent to spherical or cylindrical nodal elements; in this case the coefficient is assumed to be 0.85).

Throughout the course of the study of the mall coating designed in 2004, while analyzing the initial documentation and collecting initial data for a verification calculation of the structural construction, it was found that while designing it for rods with pipe cross-sections of 48×2.8, 57×3.2 and 76×3, the design length coefficient of 0.8 was used. This coefficient value was adopted by the developers of the design documentation as a result of tests conducted at the Kucherenko Central Research Institute in 1987. The aim of the article is to model the operation of a nodal element of a Kislovodsk-type structural construction and to substantiate the value of the design length coefficient in calculations of similar structures.

Materials and Methods. Let us consider the design of the attachment point of the elements to the nodal connector. The connecting element is a nodal connector that is a steel polyhedron with threaded holes (Fig. 1). All rods of the system have the same nominal length and consist of electro-welded or hot-rolled pipes with washers welded to the end. The rods of high-strength bolts with hexagon couplings attached to them are passed through the holes of the washers (Fig. 2). The threaded holes are directed towards the belts and braces converging at the node. The faces of the nodal element are normal to the axes of the holes and are equally spaced from the center of the node. Thus, in the case of stretched rods, the force is transmitted through high-strength bolts, and with a compressed rod, the force is transmitted through the contact of the coupling with the connector face[2].

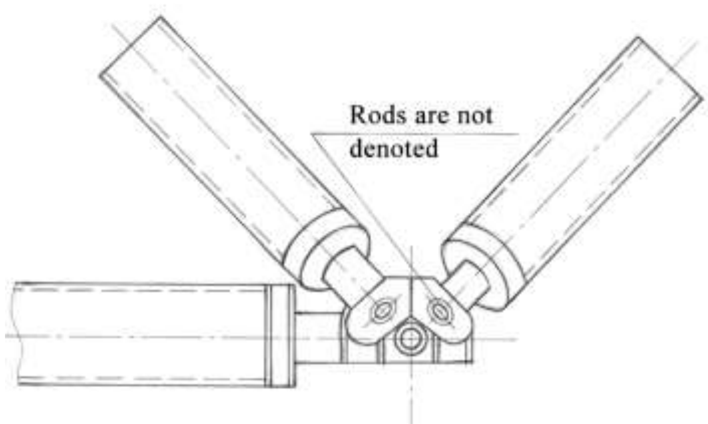


Fig. 1. Rod attachment unit of the Kislovodsk system.
URL: https://meganorm.ru/Index2/1/4293830/4293830102.htm (accessed: 17.04.2026)

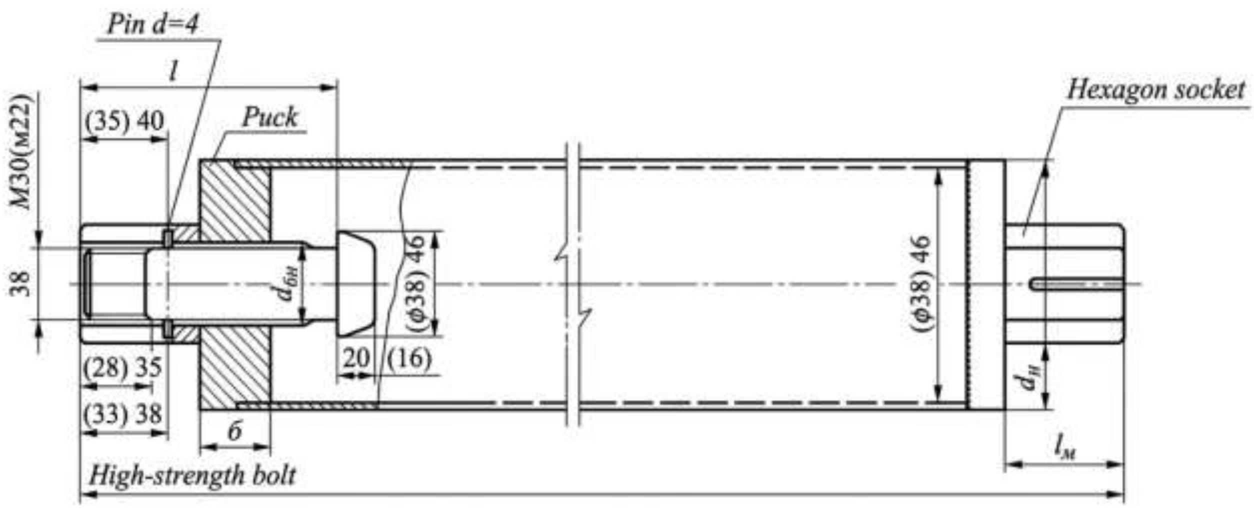


Fig. 2. Rod element of the Kislovodsk system.
URL: https://meganorm.ru/Index2/1/4293830/4293830102.htm (accessed: 17.04.2026)

[2] *Spatial Lattice Structures Made of Pipes of the Kislovodsk Type. Working Drawings.* 1.466-2 Series. Gipromontazhindustry; 1973. URL: https://meganorm.ru/Index2/1/4293830/4293830102.htm (accessed: 16.03.2026).

At a quick glance, such a joint seems to be more "hinged" than "rigid". However, a coefficient less than 1 can be obtained only with a "rigid" connection of the rods. Therefore, in order to verify the validity of applying a coefficient equal to 0.8, two questions need to be answered: what is the coefficient of stability of the rods of the lattice elements with a rigid connection and to what extent the accepted junction node can be considered close to rigid in terms of the stability of the rods.

As part of the study, it was decided to investigate the operation of structural construction elements using computer modeling. Modern engineering analysis systems (or automation systems for engineering calculations) — CAE (computer-aided engineering) — provide solutions to a large number of diverse problems of linear and nonlinear static analysis with a high degree of accuracy [6–10].

In order to perform calculations within the framework of this work, the Femap with Nastran software package was chosen as a powerful and flexible tool for solving a wide range of task types.

Research Results. The research part is divided into two separate tasks. First, the influence of the nodal connection described above on the stability of one rod is considered. Secondly, a system of rods is considered in order to determine the critical force, taking into account the influence of neighboring elements.

Task 1. Femap with Nastran provides the ability to connect two bodies of three-dimensional finite elements that touch each other, but do not have common nodes, using a "contact" and "glued" type connection. A "glued" joint restricts the movement of nodes in all directions over the entire joint area, and in fact the two bodies become one. The "contact" type connection prohibits the mutual penetration of bodies and allows one to model the free support of one body on another with no connection.

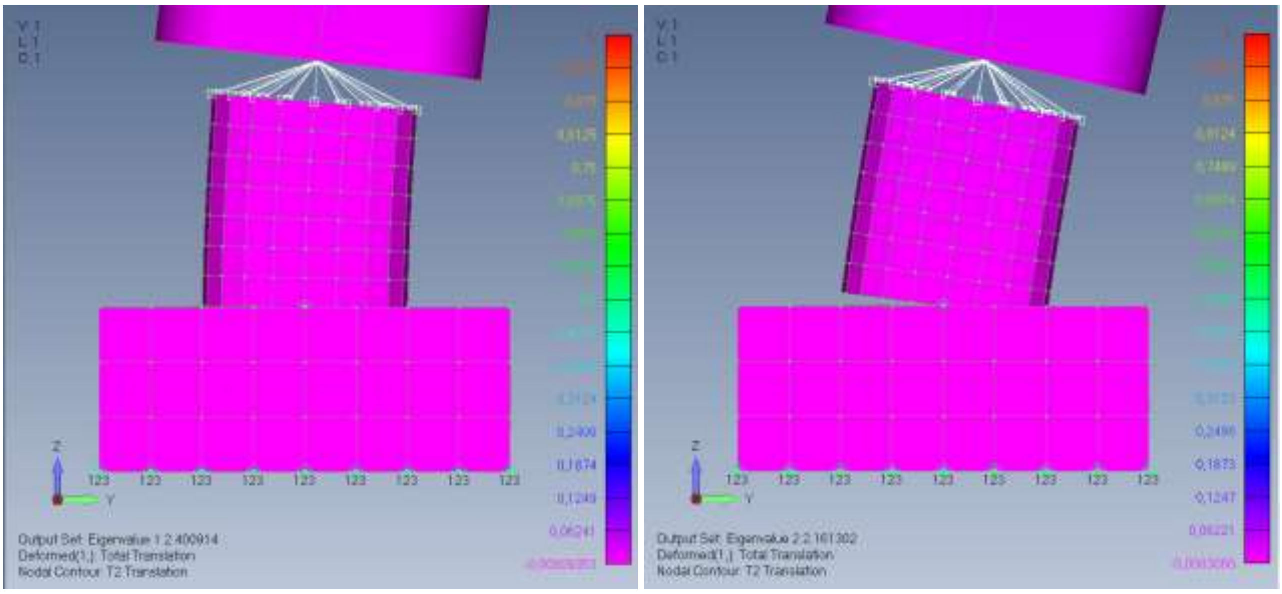
Let us model a node connection in the Femap with Nastran system. The connector is conventionally represented as a parallelepiped, the hexagonal coupling is replaced by a cylinder. The main rod is represented as a rod element and is connected to the coupling by means of a rigid coupling. The coupling and connector in the model is made as a "contact", which is closest to the actual design of the node. To compare, a stability calculation was also performed for the "glued" joint, which corresponds to a rigid joint (Fig. 3).

The main rod in the structural design has a cross-section in the form of a round tube of 80×3 and a total length of 300 cm. The Euler force for such a rod is

$$P_{\mathrm{E}} = \frac{\pi^2 E I_x}{l^2} = \frac{\pi^2 \cdot 20600 \cdot 53{,}87}{300^2} = 121{,}7 \text{ kN}.$$

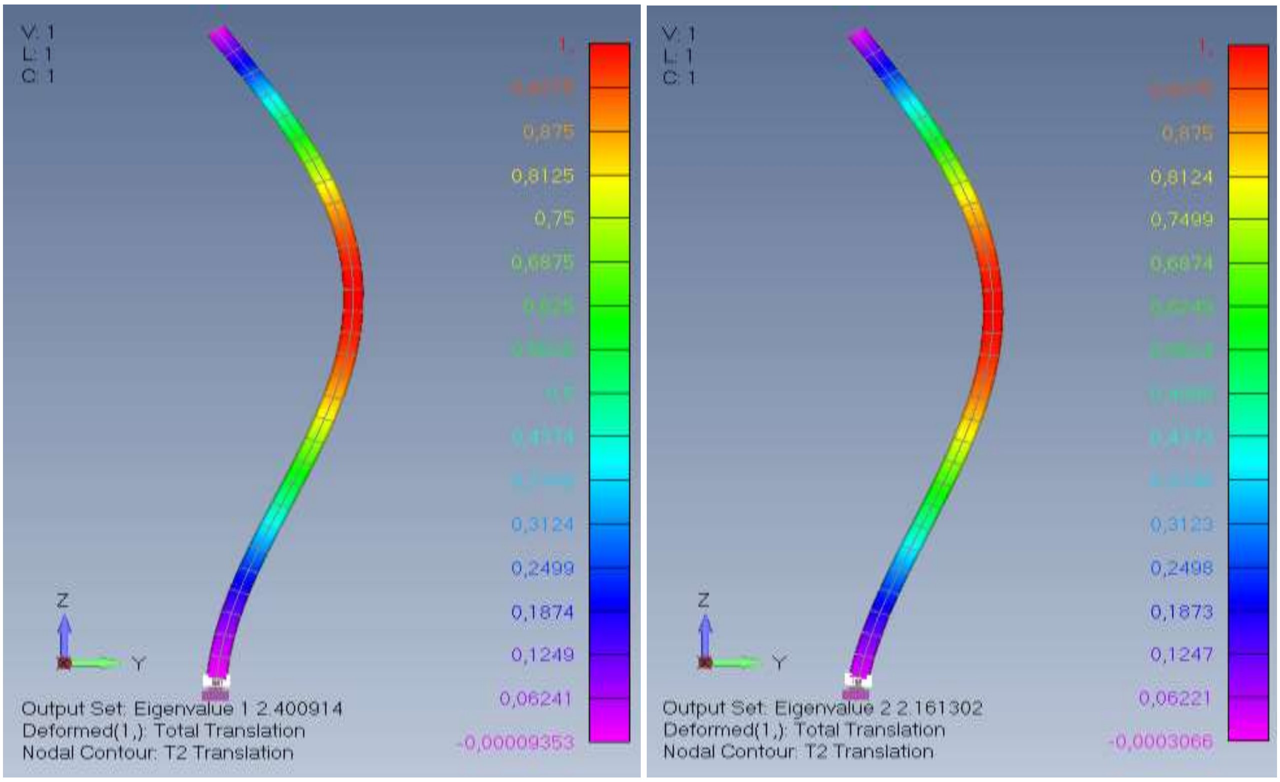
The values of the forces where the stability of such a rod was lost were 240.1 kN at the "glued" type connection, 216.1 kN at the "contact" type connection (Fig. 4). In this case, the design length coefficients are 0.712 and 0.75, respectively.

Comparing the results clearly shows that the actual operation of the node in terms of stability is very close to the rigid connection of the rod to the connector. The value of the longitudinal bending coefficient increases by 5%.



a) *b)*

Fig. 3. Options for modeling a nodal connection: *a* — "glued" type connection; b — "contact" type connection



a) *b)*

Fig. 4. Loss of stability while connecting: *a* — "glued"; *b* — "contact" type

Task 2. In order to identify the critical force for compressed elements, the following calculation scheme was chosen: four elements of the lower belt and four lattice elements connected in one node are considered [11–14]. The opposite end of each of the circuit elements has a hinge attachment. The length of all of the rods is 3 meters. It should be noted that while adding elements of the upper belt to the calculation model, the bevel rod will have two ends with varying degrees of pinching from loss of stability. However, it is assumed that the elements of the upper belt are predominantly in a compressed state, therefore they have a lower fixing effect, which is commonly neglected.

For starters, let us provide as an example an idealized loading variant of the design scheme, where all the elements of the lower belt are stretched by a single force, and the elements of the braces are stretched and compressed in pairs by a single force (Fig. 5). The cross-section of all the elements is a 60×3 tube.

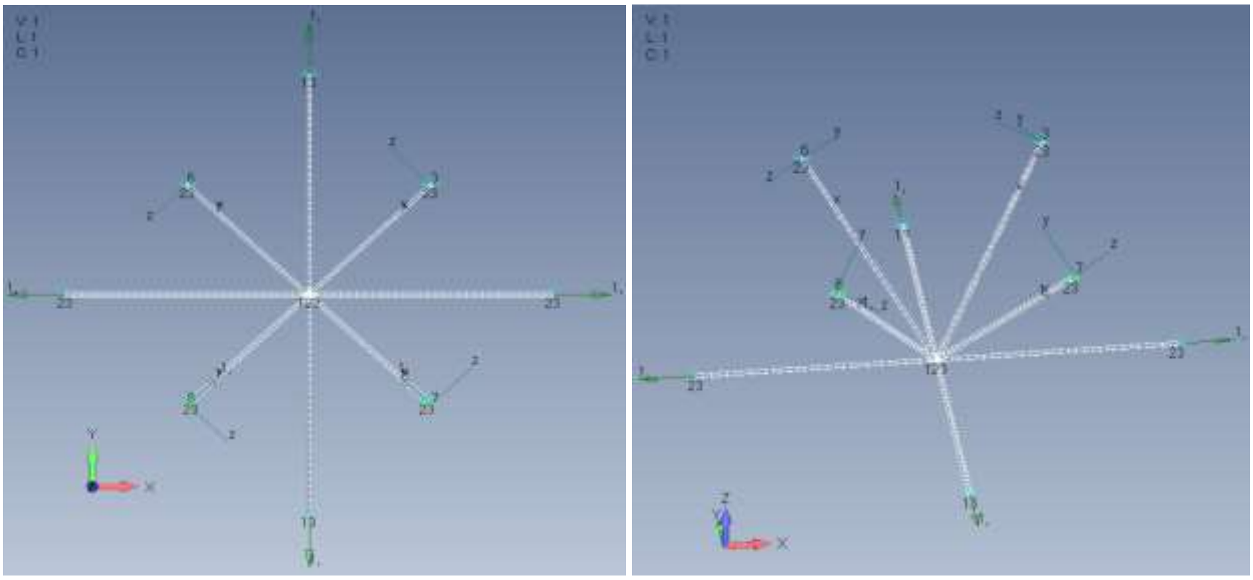


Fig. 5. Calculation scheme

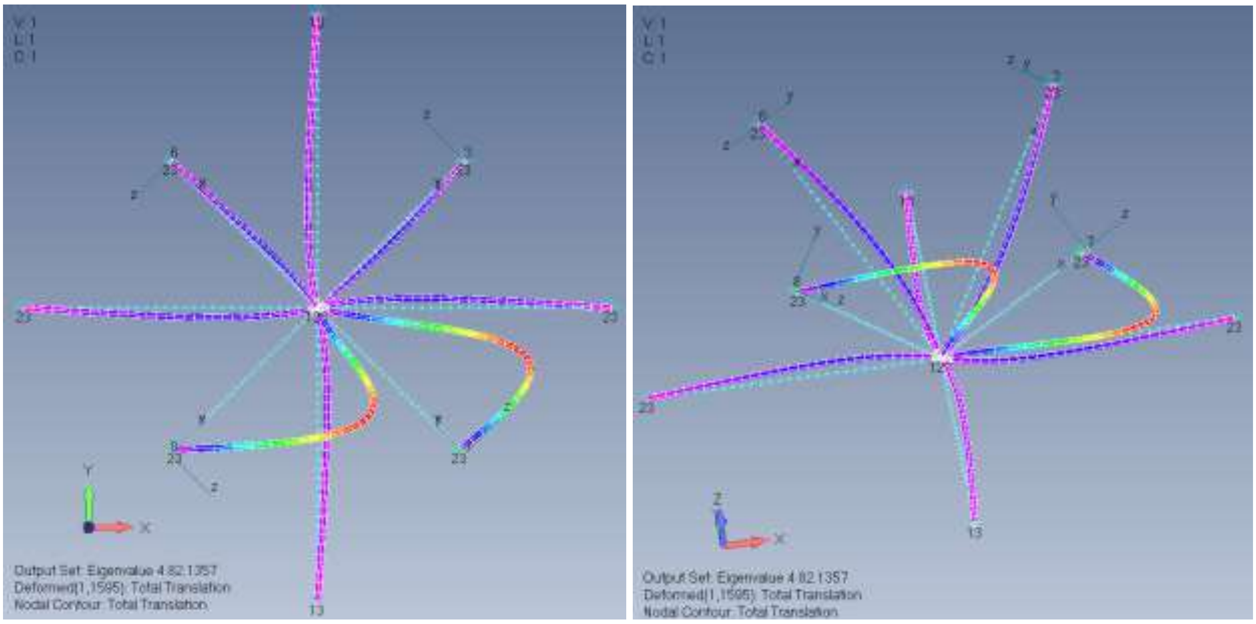


Fig. 6. Deformed circuit with loss of stability option 1

The compressed rods lost their stability at a force of 82.14 kN (Fig. 6), which yields a stability coefficient of 0.776.

The following is an example with a calculation for force ratios taken from an actual design. The loss of stability of the compressed rod occurs at a force value of 143.88 kN (Fig. 7), which yields a stability coefficient of 0.849. Some other combinations of forces and cross-sections of elements that were calculated as this study was being performed resulted in a design length coefficient ranging from 0.77 to 0.88.

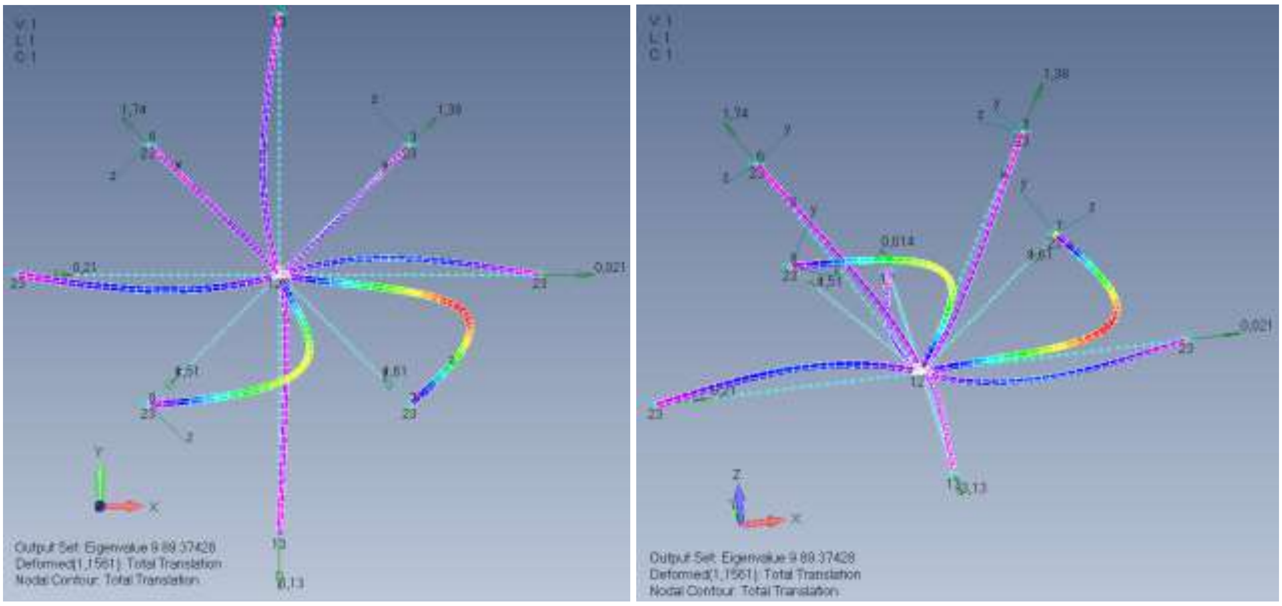


Fig. 7. Deformed circuit with the loss of stability option 2

As a result of the calculations, the following conclusions can be made:

1. The joint of the rod with the connector, where the compressive forces are transmitted through a tight contact, is very close to rigid fastening by the nature of the work while testing the stability of the rod.

2. The design length coefficient for compressed elements ranges from 0.77 to 0.88, depending on the force values in the elements and their cross sections. Based on this, it can be unambiguously concluded that the application of a coefficient of 0.8 for all the compressed rods cannot be theoretically justified.

Discussion and Conclusion. The studies of the coating in the form of a structural constructions of the Kislovodsk type show the correctness of the application of the requirements of SP 16.13330.2011 "Steel Structures" in terms of determining the design length of a compressed rod equal to 1, and the use of a coefficient equal to 0.8 cannot be theoretically justified. On top of that, while calculating and designing such structures, the fact that in a system of this type there are a large number of rods with different forces should be taken into account and considering all possible combinations is a complex task. At the same time, the increase in metal consumption while considering a single element and taking into account only the pipe section is 20–30%, however, if the metal consumption and cost of manufacturing the entire structure is taken into consideration, these savings are significantly offset and are not feasible.

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AA Limantsev: preparation of the manuscript, calculations, formation of the conclusions.

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Methodological Foundations of a Project of Organizing Construction and Development of Territories in a System of Integrated Territorial Development

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Abstract

Introduction. This study is dedicated to development of methodological foundations of territorial development construction project (TDCP) as a tool for managing integrated territorial development (ITD). The existing approaches to construction organization based on the traditional object-by-object construction organization project (COP) fail to provide necessary coordination of resources and logistics. The aim of the study is to develop the theoretical foundations and structure of TDCP and structure of ITD, ensuring effective management of construction processes within the framework of ITD projects and master plans.

Materials and Methods. The research is based on the analysis of theoretical approaches to construction organization, systematization of experience in implementing integrated territorial development projects and identification of the drawbacks of the existing methodologies. Methods of system analysis, comparative analysis, structural and functional modeling and generalization of the theoretical provisions were used.

Research Results. The structure of TDCP has been developed, including the external contour (transport accessibility, engineering infrastructure, engineering protection of a territory, logistics links, natural and territorial conditions, institutional environment) and an internal contour (production facilities, temporary infrastructure, internal engineering networks, transport networks, labor resources, stages of construction). The mechanism of converting the constraints of the external contour into the internal one and compensatory measures ensuring the sustainability of the project is substantiated.

Discussion and Conclusion. The developed TDCP methodology serves as the theoretical foundation for organizing construction within integrated territorial development projects. The introduction of the concept of external and internal contours enables one to systematically account for the limitations and capabilities of a territory, ensuring efficient allocation of resources and coordination of construction processes.

Keywords: territorial development construction project, integrated territorial development, external contour, internal contour, construction organization, resource coordination, logistics, territorial planning

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Оригинальное эмпирическое исследование

Методологические основы проекта организации строительства развития территорий в системе комплексного развития территорий

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Аннотация

Введение. Настоящее исследование посвящено разработке методологических основ проекта организации строительства развития территорий (ПОСРТ) как инструмента управления комплексным развитием территорий (КРТ). В условиях масштабного территориального развития существующие подходы к организации строительства, основанные на традиционном пообъектном проекте организации строительства (ПОС), не обеспечивают необходимой координации ресурсов и логистических процессов. Цель работы — разработка теоретических основ и структуры ПОСРТ, обеспечивающих эффективное управление строительными процессами в рамках проектов КРТ и мастер-планов.

Материалы и методы. Исследование основано на анализе теоретических подходов к организации строительства, систематизации опыта реализации проектов комплексного развития территорий и выявлении недостатков существующих методологий. Использованы методы системного анализа, сравнительного анализа, структурно-функционального моделирования и обобщения теоретических положений.

Результаты исследования. Разработана структура ПОСРТ, включающая внешний контур (транспортная доступность, инженерная инфраструктура, инженерная защита территории, логистические связи, природно-территориальные условия, институциональная среда) и внутренний контур (производственные объекты, временная инфраструктура, внутренние инженерные сети, транспортные сети, трудовые ресурсы, этапность строительства). Обоснован механизм трансляции ограничений внешнего контура во внутренний и компенсационные меры, обеспечивающие устойчивость реализации проекта.

Обсуждение и заключение. Разработанная методология ПОСРТ формирует теоретическую основу для организации строительства в рамках проектов комплексного развития территорий. Введение концепции внешнего и внутреннего контуров позволяет системно учитывать ограничения и возможности территории, обеспечивая эффективное распределение ресурсов и координацию строительных процессов.

Ключевые слова: проект организации строительства развития территорий, комплексное развитие территорий, внешний контур, внутренний контур, организация строительства, ресурсная координация, логистика, территориальное планирование

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Introduction. Over the recent decades, the issues of integrated territorial development (ITD) have become of particular importance in the context of sustainable development of urban and rural areas [1]. The implementation of Federal Law No. 494-FZ "On Integrated Territorial Development" and national projects aimed at enhancing the quality of the urban environment calls for an improvement in the methodology of construction organization [2]. This is particularly the case for regions witnessing an active economic and demographic growth, such as the Chechen Republic, where a large-scale territorial development program is being implemented [3].

The existing approaches to construction organization based on the traditional on-site construction organization project (COP) developed in compliance with SP 48.13330.2019 "Construction Organization" [4] fail to provide the necessary coordination of resources and logistical processes in implementing ITD projects [5]. The traditional COP developed in compliance with the requirements of the Urban Planning Code of the Russian Federation dated December 29, 2004, No. 190-FZ focuses on a separate building or structure and fails to account for the specific features of mass construction in an area with a set of interconnected objects.

Studies by Azari Abramovich Lapidus substantiated the need to develop a territorial organization project (TOP) as an element of integrated territorial development. The further development of the concept led to the idea of a territorial development construction project (TDCP), a comprehensive document integrating solutions for engineering training, logistics and management of construction flows at a territory level [1, 6].

An analysis of the scientific literature shows that the work of both domestic [5, 6] and foreign researchers [7–10] is dedicated to organizing construction within the framework of ITD. However, a comprehensive methodology combining the theoretical foundations and practical tools of construction organization for ITD projects calls for further development.

The aim of the study is to develop the methodological foundations of a territorial development construction project, ensuring effective management of construction processes within the framework of integrated territorial development projects.

To this end, the following tasks were addressed:

– theoretical approaches to construction organization were analyzed, the drawbacks of the existing methodologies in relation to ITD projects were identified;

– structure of the ITD was developed, including the external and internal contours of construction organization;

– mechanism of interrelation of contours and translation of constraints is substantiated;

– traditional TOP and TDCP were compared.

Materials and Methods. The study is based on an analysis of regulatory documentation, scientific literature and practice of implementing projects for integrated territorial development. The following methods were employed:

1. The method of system analysis is applied in order to identify the interrelationships between the elements of construction organization within the framework of ITD and formation of ITD structure. This method enabled us to regard construction organization as a complex system, including the constantly interacting external and internal elements.

2. A comparative analysis method is used in order to compare the traditional TOP and the proposed TDCP according to the major parameters: planning object, scale, logistics approach, engineering support, planning, coordination of participants, resource management, risks and the final result.

3. The method of structural and functional modeling is used in order to develop a conceptual model of the ITD that includes external and internal contours with details of their constituent elements and interaction mechanisms.

4. The method of generalization of theoretical provisions is employed in order to form the methodological foundations of the ITD based on an analysis of the existing approaches to construction organization and territorial planning.

Research Results. As a result of the study, the structure of a territorial development construction project has been developed, which is a comprehensive document combining territorial development planning and construction organization.

TDCP includes two major components: the external contour and internal contour (Fig. 1).

The external contour of ITD covers elements located outside a projected territory, but which have a major impact on construction organization. The external contour includes:

– transport accessibility includes access highways, their capacity, structure of traffic flows and restrictions on movement of freight transport. ITD projects are characterized by a high intensity of cargo transportation, which calls for special solutions for transport logistics organization;

– engineering infrastructure covers the main networks of electricity, water supply, sanitation, heat and gas supply, as well as resource provision sources. The parameters of the backbone networks are critical to the connectivity and call for the design of internal distribution systems;

– engineering protection of the territory includes measures for protection against flooding, landslides, mudslides and other natural impacts. For territories with a difficult terrain, this element is of particular significance;

– logistical connections are critical to the structure of suppliers of building materials, delivery routes, availability of production bases and warehouse complexes in a project implementation region;

– natural and territorial conditions include terrain, geological conditions, climatic factors, hydrological conditions, and environmental constraints;

– institutional environment covers regulatory framework, administrative procedures, system of government regulation, and a project's stakeholders.

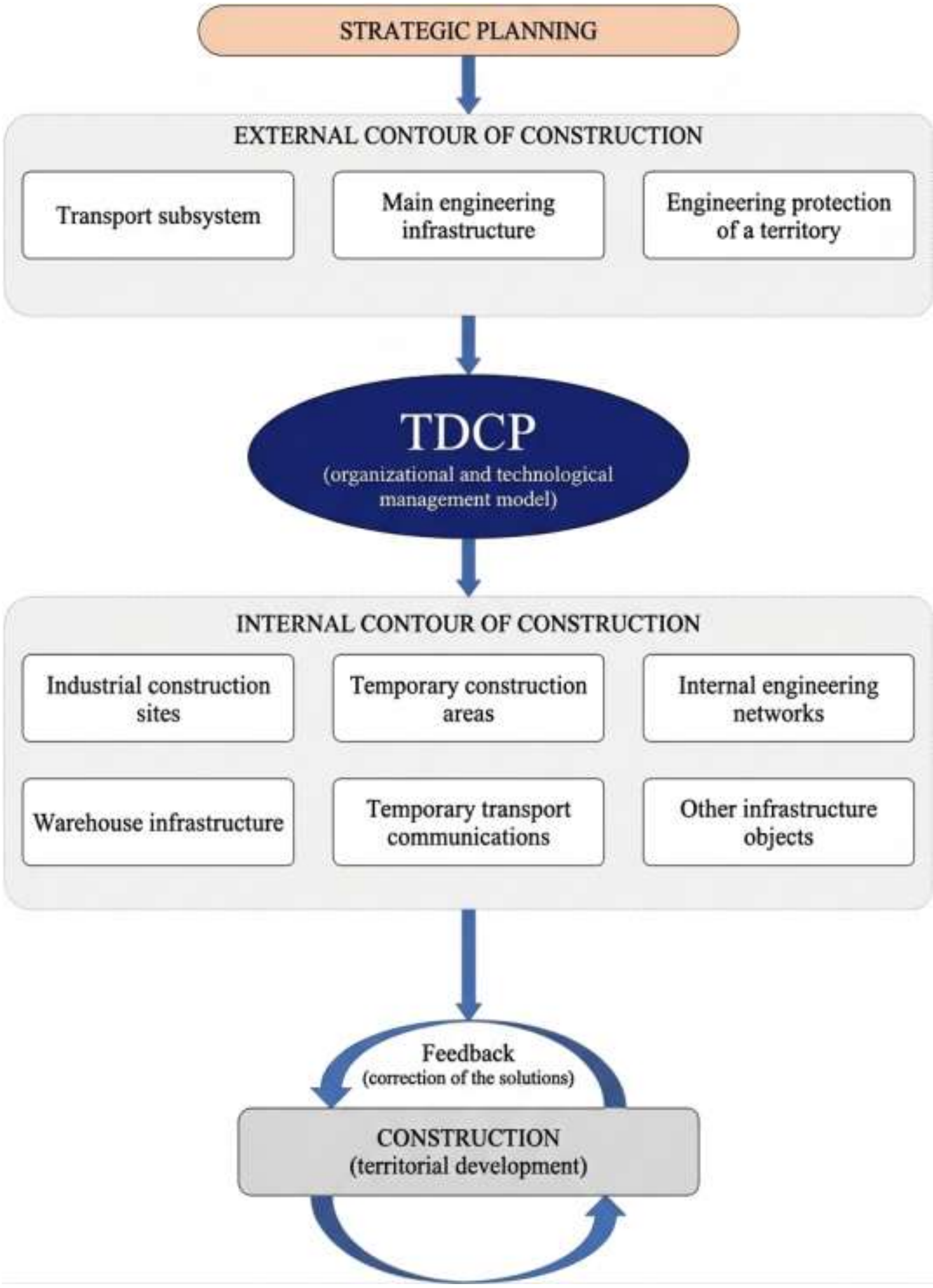

Fig. 1. Structural model of interaction of the external and internal contours in a project system for construction organization and territorial development

The internal contour of the TDCP includes elements located within a projected territory and ensuring implementation of construction processes:

– production facilities are concrete mixing units, reinforced concrete products factories, building materials warehouses, waste processing facilities;

– temporary infrastructure are residential towns, catering facilities, sanitary facilities, administrative areas for the staff;

– internal engineering networks are distribution networks of electricity, water supply, sanitation, heat supply, gas supply and communications;

– internal transport networks are road infrastructure for supporting construction processes, including permanent and temporary access roads;

– labour resources are a system of providing a project with the staff, labour organization, and household services for the employees;

– a construction stage is a sequence of commissioning facilities, synchronization of residential and social infrastructure, adaptive mechanisms for adjusting schedules.

The mechanism of a relationship between the external and internal contours is based on converting constraints. The parameters of the external contour form constraints and set the allowable solution space for the internal contour. The internal contour, in turn, implements compensatory measures aimed at leveling the deficits of the external contour.

Effectiveness of the interconnection of the contours is evaluated by means of the indicator of integral consistency:

$$K_c = \sum(w_i \times k_i), \quad (1)$$

where w_i is the weight factor of the i-th subsystem; k_i is the consistency indicator of the i-th subsystem defined as the ratio of the actual parameters to the required ones.

The value of $K_c \geq 1$ is indicative of sufficient consistency of the contours.

The comparison of traditional TOP and TDCP showed considerable differences in the approaches to construction organization (Table 1).

Table 1

Comparative analysis of traditional TOP and organization of territorial development construction project

Criterion	Traditional TOP	TDCP
Management object level	One object (building / structure)	Territory of ITD/ area (a system of objects)
Target result	Organization of work at an object	Coordination of territorial development and synchronization of flows between objects
Management contour	Mostly indoor (construction site)	Two contours: external (infrastructure/ constraints + internal (construction)
Connection with territorial planning	Indirect (via the original project data)	Direct integration with the master plan / master plan / ITD and development programs
Planning horizon	Construction period of an object	Multi-year development stages (queues of 4–10 years or longer)
Engineering infrastructure scale	On-site temporary/ permanent networks	Backbone networks + step-by-step connection and internal infrastructure of a territory
Transport and accessibility	Entrances to the site, a local layout	Territorial transport subsystem: capacity, nodes, external links
Material logistics	"On-site" delivery	Territorial logistics: sources–routes–nodes–acceptance points, inter-object coordination
Resource coordination	Resources within an object / contractor	Resource balance in a territory: preventing contractors from competing for capacity and supplies
Deficit management	Reactive (in case of faults)	Proactive: identification of the external contour deficiencies and their compensation by means of internal contour solutions
Risks and uncertainty	Object risk analysis	Risks of the life cycle of a territory: infrastructural, logistical, institutional, environmental
Institutional conditions	Typically outside a model (background)	Considered as a subsystem: approvals, regulations, participant interfaces, time limits
Modelling methods	Network / calendar object planning	System-technical model of a territory: indicators of security, deficits, scenarios, optimization of flows
Digital foundation	BIM-model of an object (partially)	BIM + CIM + GIS / digital platforms of a territory, data for monitoring and correction of solutions
Feasibility criterion	Feasibility of an object schedule	Feasibility of the stages of a life cycle of a territory adhering to the security thresholds of the external contour
Practical effect	Optimization of terms / resources of an object structure	Sustainability of ITD: reduction in stage disruptions, consistency of infrastructure and schedules, manageability of housing and communal services

Traditional TOP is focused on a separate building or structure, provides point-to-point construction and makes use of an object-by-object approach. Logistics is based on the principle of supplies to a construction point, engineering is based on connection to the existing networks. Planning is characterized by tight deadlines and volumes, coordination is limited by interaction of the customer and contractor, and resource management is local. The risks remain high due to the lack of reserves causing a object to be isolated.

ITD is focused on a territory with a complex of objects, provides mass construction and makes use of a territorial approach. Logistics is based on an integrated supply chain, while engineering relies on the design of distribution networks. Planning is characterized by adaptive phasing, there is coordination on a multi-level basis with all participants, and resource management is territorial. Risks are reduced due to buffer mechanisms causing a territory to be comprehensively developed.

The practical implementation of TDCP allows the following effects to be attained:

– reducing transport and logistics costs by 15–25% by optimizing routes and creating buffer warehouses;

– reduction in construction time by 10–20 % due to coordinated staging and parallel work;

– increasing resilience of a project to external influences by reserving critical resources;

– integrated development of a territory with commissioning both residential and social infrastructure.

Discussion and Conclusion. The developed ITD methodology expands on the existing ideas on construction organization while introducing a territorial management level and combining logistical as well as resource aspects into a single model. The differentiation of external and internal contours provides an opportunity for a systematic analysis of interrelationships between infrastructural constraints, logistical flows and construction production organization.

The major difference between the ITD and raditional TOP is the territorial approach where a planning object is not a separate building, rather a territory with a set of interconnected objects. This calls for a fundamentally different approach to logistics, engineering, and resource management.

The mechanism of converting constraints of the external contour into the internal one and feedback by means of compensation measures ensures adaptability of a construction management system to changing conditions. The integrated circuit consistency indicator K_c allows one to quantify a degree of contour consistency and to identify system bottlenecks.

The practical significance of the research results is the possibility of applying the developed methodology in the implementation of ITD projects in the country's various regions. The ITD methodology can be integrated into regional development programs and employed by government authorities, design organizations and construction companies.

The prospects for further research are related to developing mathematical models for optimizing the parameters of the ITD, designing software to support decision-making in ITD formation, testing the methodology on specific ITD projects and developing standard solutions for various categories of territories. Hence the developed methodological foundations of ITD serve as a scientific foundation for improving construction organization within the framework of integrated territorial development projects and contribute to improving the effectiveness of urban planning activities.

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Original Empirical Research

Correlations of Damage to the Main Structures of Industrial Buildings from their Service Life

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Abstract

Introduction. Well-known studies on the research subject have failed to specify sample sizes and types of workshops based on their technological characteristics, which prevents us from summarizing the numerous damage data. The downside of the previous studies is the lack of generalizations with designing regression dependences of damage to the bearing elements of buildings on their service life. The subject of the study is the correlation of damage to the main structures of industrial buildings from their service life. In order to design these dependencies, a significant sample of workshops (at least 100 workshops) has been investigated and the frequency of damage to their main structures has been examined. The frequency of damage has been previously studied and published by the authors. The aim of the study is to establish correlations between the degree of damage and the service life of structures.

Materials and Methods. The objects of the study were single-storey industrial buildings of the metallurgical, machine-building, energy industries, as well as production of building materials. The workshops were inspected by means of the standard methods endorsed in the national standards and norms. In order to analyze defects and damages, an electronic database was designed in a tabular processor that recorded all the basic information on types of workshops, materials and structures, defects and damages.

Research Results. The analysis of data on a large sample of industrial buildings made it possible to establish correlations between damage and the service life of structures. It was found that the rate of damage to steel columns is higher than that of reinforced concrete columns. The intensity of damage to steel columns is described by means of an exponential dependence, while that of reinforced concrete columns is a linear one. Brick and reinforced concrete walls are damaged linearly depending on the service life. The dependence "damage - service life" of reinforced concrete floors and coatings is approximated by a polynomial of the second degree, close to a linear relationship. Similar linear damage is typical of steel trusses. The damage rate of steel crane beams does not directly depend on the service life of the building itself, as the beams are replaced if damaged.

Discussion and Conclusion. As a result of the study of the frequency of damage, mathematical models of "damage - service life" of industrial building structures were obtained that can be used in order to optimize maintenance and repairs of buildings. The distinctive features of the accumulation of damages and defects over time for metal frames of industrial buildings associated with human mistakes are noted. In order to reduce accidents and optimize costs throughout the life cycle of buildings, it is suggested that a risk-based approach is applied, the likelihood of errors and cost of restoration measures at the stages of design, construction and operation are assessed.

Keywords: industrial buildings, building maintenance, construction accidents, defects and damages, reliability and safety

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Оригинальное эмпирическое исследование

Корреляции поврежденности основных конструкций промышленных зданий от сроков их эксплуатации

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Аннотация
Введение. В известных исследованиях по теме статьи не указываются объемы выборки и типы цехов по их технологическим признакам, что не позволяет обобщить многочисленные данные о повреждениях. Недостатком ранее проведенных исследований является отсутствие обобщений с построением регрессионных зависимостей повреждаемости несущих элементов зданий от сроков их эксплуатации. Предметом исследования являются корреляции поврежденности основных конструкций промышленных зданий от сроков их эксплуатации. Для построения указанных зависимостей исследована значимая выборка цехов (не менее 100 цехов), и изучена частотность повреждений их основных конструкций. Частотность повреждений была изучена ранее и опубликована авторами. Целью настоящего исследования является построение корреляций между степенью поврежденности и сроками эксплуатации конструкций.
Материалы и методы. В качестве объектов исследования рассматривались одноэтажные промышленные здания металлургической, машиностроительной, энергетической отраслей, а также производства строительных материалов. Обследование цехов производилось по стандартным методикам, утвержденным в национальных стандартах и нормах. Для анализа дефектов и повреждений была создана электронная база в табличном процессоре, в которой фиксировалась вся основная информация по типам цехов, материалам и конструкциям, дефектам и повреждениям.
Результаты исследования. Анализ данных по большой выборке промышленных зданий позволил установить корреляции между поврежденностью и сроками эксплуатации конструкций. Установлено, что скорость повреждений стальных колонн больше, чем у железобетонных. Интенсивность повреждений стальных колонн описывается экспоненциальной зависимостью, а железобетонных — линейной. Кирпичные и железобетонные стены повреждаются по линейной зависимости от сроков эксплуатации. Зависимость «повреждаемость — сроки эксплуатации» железобетонных перекрытий и покрытий аппроксимируется полиномом второй степени, близким к линейной зависимости. Подобная линейная повреждаемость характерна и для стальных ферм. Повреждаемость стальных подкрановых балок прямо не зависит от сроков эксплуатации самого здания, так как балки заменяют по мере их повреждений.
Обсуждение и заключение. В результате исследования частотности повреждений были получены математические модели «поврежденность — срок эксплуатации» конструкций промышленных зданий, которые могут быть использованы для оптимизации технического обслуживания, планирования экспертиз, периодических осмотров, текущих и капитальных ремонтов зданий, разработки стратегии предприятий по замене основных производственных фондов. Отмечены отличительные особенности накопления со временем повреждений и дефектов для металлических каркасов промышленных зданий, связанные с человеческими ошибками. Для снижения аварийности и оптимизации затрат на протяжении жизненного зданий предлагается применять риск-ориентированный подход, оценивать вероятность возникновения ошибок и стоимость восстановительных меприятий на стадиях проектирования, возведения и эксплуатации.

Ключевые слова: промышленные здания, эксплуатация зданий, строительные аварии, дефекты и повреждения, надежность и безопасность.

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Introduction. According to the estimates of the Russian Academy of Architecture and Building Sciences (RAABS) [1] there is a need to develop a regulatory framework based on ensuring the integrated safety of objects throughout their full life cycle. This calls for scientific research to be conducted, including studies of durability of building materials, structures and products as well as of longevity of construction sites.

According to [2–4], the causes of defects and damage to industrial buildings are deficiencies in construction — 28%; violation of operating rules — 26%; poor quality of materials — 28%; design mistakes — 10%; deficiencies in design standards as well as guidelines for manufacture and installation of structures — 8%. The observed frequency of defects in installation of steel and reinforced concrete structures [5] confirms these conclusions. The number of defects in joints and assemblies for steel structures is 56.5%, and for reinforced concrete structures — 57.7%.

According to [2, 3], the proportions of accidents related to the operational stage has gone up from 11% to 35%, which is due to the deterioration of the country's fixed assets (industrial buildings and infrastructure). This trend is particularly evident for industrial buildings that are characterized by large spans and severe operating conditions associated with aggressive environments, lifting equipment, dynamic loads, etc.

Accidents involving coatings on industrial buildings should be considered the most common ones. An analysis of accidents shows that about 53% of all collapses occur as a result of overloading with snow and dust [6]. According to [7, 8], the following damage to rafter trusses occurs for the main buildings of thermal power plants: local rod bends — 28%; wear of anticorrosive coating and metal corrosion — 23%; general bends of rods of trusses — 24%; accumulations of aggressive technological dust on structures — 9%; non-nodal plate support on the upper belt of trusses — 6%.

A damage rate of structures depends on service life [7, 8] (comparison for 20 and 80 years of service life): reinforced concrete and steel columns — an increase from 10 to 20%; reinforced concrete and steel crane beams — from 10 to 70%; steel rafter trusses — from 15 to 40%; coating plates — from 20 to 60%; secondary reinforced concrete platform beams — from 15 to 80%; main reinforced concrete platform beams — from 10 to 25%.

A regular decrease in the number of damages from top to bottom, ranging from coating plates and trusses to columns, as well as rapid fatigue wear of crane beams, was identified. According to [9], the number of damaged crane beams increases from 14 to 73% with an increase in the service life from 3 to 22 years. The most common damage to beams is fatigue cracks in the belt seams of the upper belt.

Studies [10, 11] describe typical defects in steel columns: bends and cutouts of lattice elements, local bends and curvature of tent and crane branches, destruction of attachment points of crane beams as well as changes in a typical structural design of joints. The most frequent damages are used in computer modeling in order to assess a degree of serviceability of metal structures of industrial buildings [12].

The authors of [13] identified five characteristic areas of defects and damage to walls of buildings. It is suggested that they are combined into groups when conducting surveys or monitoring the condition of a facade. This allows one to track the dynamics of negative processes and conduct major repairs of facades in a timely manner. In [14], the damage was analyzed with an indication of their causes. The frequency of damages and defects is determined by the types of structures, severity of defects, as well as their causes.

A study [15] suggests ways to enhance the reliability of industrial building frames: boosting the strength reserve by increasing cross-sections, the strength of materials, and the use of more efficient cross-sections; using statically indeterminate systems with a redistribution of forces at the time of local destruction; incorporating coupling (non-force) elements.

The review showed that damage to industrial buildings has been investigated time and time again, but these studies typically fail to specify sample sizes and types of workshops based on their technological characteristics, making it impossible to summarize the numerous data. The drawback of the previous studies is the lack of generalizations with the construction of regression dependences of damage of load-bearing elements of buildings on their service life, which does not allow to justify and plan a rational program of their operational control.

Materials and Methods. The objects of the research were 100 workshops — single-storey industrial buildings of the metallurgical, machine-building, energy industries, as well as production of building materials. The years of commissioning of the workshops ranged from 1902 to 2016: 1902–1940 — 10 objects; 1941–1970 — 61 objects; 1971–1990 — 21 objects; 1991–2016 — 9 objects.

According to the design scheme, the buildings were divided into frame, wall and frame-wall ones. According to the design and materials: full reinforced concrete frame; full metal frame; mixed frame (reinforced concrete and steel); stone load-bearing walls with various coating options.

Almost all of the buildings had lifting equipment in the form of overhead cranes and truss cranes with a lifting capacity from 2 to 280 tons with various operating modes: from repair to heavy (from 1K to 8K). The degree of aggressiveness of the environment was mostly classified as non-aggressive or mildly aggressive.

The following structures were considered as groups of the same type: foundations, columns, walls, crane beams and trusses, sub-trusses and rafter trusses, lanterns, slabs and coatings, connections along columns and coatings. In material design these structures were divided into stone (brick, cinder block, butte), reinforced concrete monolithic and prefabricated, steel, wooden. In most cases, the roof of the surveyed buildings was rolled, less frequently metal.

The workshops were inspected according to the standard methods endorsed in the national standards and regulations. According to the guideline of the GOST 31937-2024 "Buildings and Structures. Rules for Inspection and Monitoring of Technical Condition" and SP 13-102-2003 "Rules for Inspection of Load-Bearing Building Structures of Buildings and Constructions", the work included identifying the geometric parameters of structures and assemblies, their compliance with the design and regulatory documentation; examination of the identified defects and damage, identification of their causes; instrumental control of the strength characteristics of the materials; specification of loads on load-bearing structures; verification calculations of damaged (defective) load-bearing and enclosing structures; identification of their technical condition; development of recommendations for repairing and strengthening building structures.

In order to analyze defects and damage, an electronic database was designed where the following information was recorded: the name of an object; structural type of a building; year of commissioning and of inspection; a brief description of a building structure; characteristics of lifting equipment; aggressiveness of the workshop environment; type of a structure, element and material of a construction; design and actual strength of a material; type and magnitude damage (defect); frequency of damage in the form of a ratio of damaged elements to the total number of elements in a building; localization and brief description of damage; cause of damage (defect); severity of defects in categories A, B and C; coefficient of reduction in the strength of a damaged element; category of technical condition of a structure; main recommendation for restoration.

Research Results. According to the data in Fig. 1, in terms of the degree of preservation characterized by the frequency of damage and categories of technical condition, steel and reinforced concrete structures are superior to stone ones.

This is directly due to the uniformity of the material, which has an impact on its resistance and durability, as well as the fact that stone walls are more susceptible to atmospheric erosion. On top of that, rapid damage to the corrosion protection of metal with low cross-section massiveness accounts for a frequent transition to a limited working condition of steel structures (45.3%). Roof and wooden roof structures ("other" in Fig. 1) very frequently transition into a limited working and inoperable state (92.5% of damage) due to a relative fragility of the materials and intense wear.

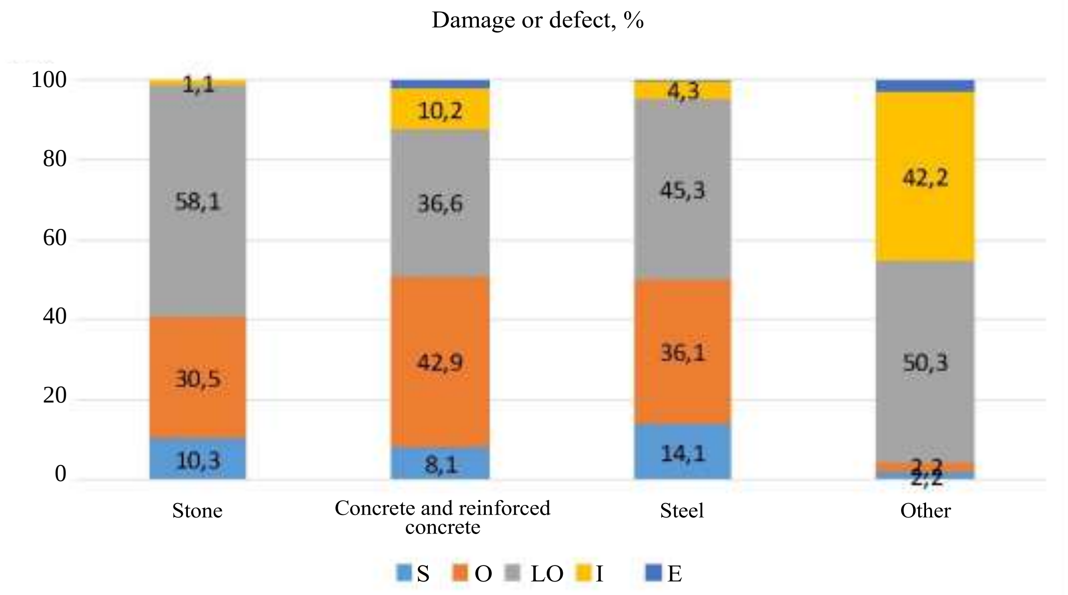

Fig. 1. Comparison of the categories of technical condition of types of constructions in %:
S — serviceable; O — operational; LO — limited operational; I — inoperable; E — emergency

Thus, the greatest degree of damage is observed in the structures forming a building shell: roof and covering, exterior walls, as well as structures located close to it. Studies have shown that the frequency of structural damage drops in the "top-down" direction: roof, covering, trusses, columns, etc.

The main causes of damage are roof and gutter leaks, wear phenomena (corrosion, humidification and defrosting, destruction of protective coatings, etc.), as well as effects of various production technologies (crane cyclic loads, accidental impacts by loads and workshop vehicles, cutouts of elements, aggressive environment, dust and scale accumulations, etc.).

The following are the results of investigating the dependence of damage on operating time. As the graphs (Fig. 2) suggest, reinforced concrete columns are damaged less intensively than steel ones. After 60 years of service life, about 20% of reinforced concrete columns and 30% of steel columns get damaged. After 100 years of service life, almost all steel columns will have been damaged.

A more intense damage to steel columns is primarily due to metal corrosion (over 40% of damaged columns). The intensity of damage to steel columns is described by an exponential dependence (Fig. 2). The accuracy of the approximation is satisfactory: the coefficient of determination R^2 = 0.788, the correlation coefficient is statistically significant.

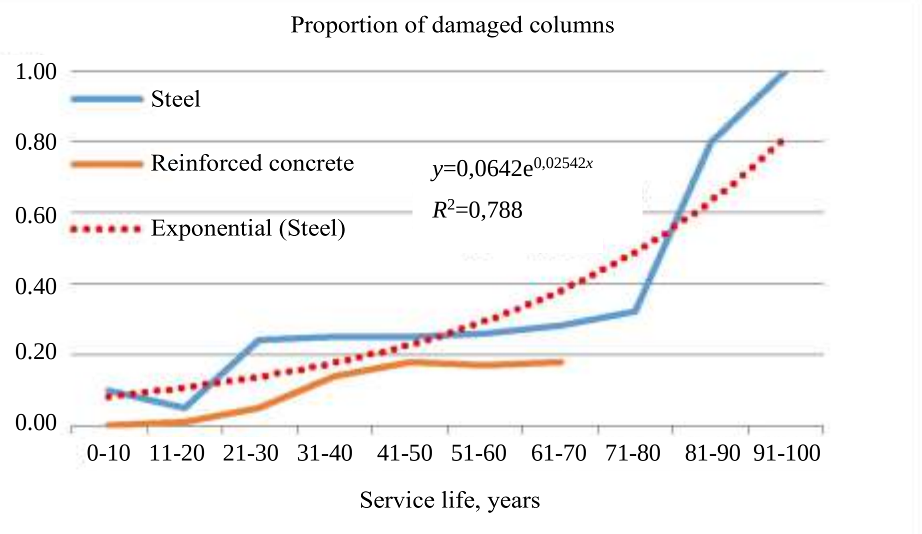

Fig. 2. Dependence of the column damage on service life

The coefficient of determination is calculated as the ratio of the sum of the squares of the regression residuals to the total sum of the squares:

$$R^2 = 1 - \frac{\sum_{i=1}^{n}\left(y_i - \hat{y}_i\right)^2}{\sum_{i=1}^{n}\left(y_i - \bar{y}_i\right)^2}.$$

At $R^2 \geq 0{,}95$ the accuracy of the approximation is high, at 0,8–0,95 — satisfactory, at 0,6–0,8 — weak, at $R^2 < 0{,}6$ — insufficient.

The average approximation error is the average relative deviation of the calculated values from the actual values y_i:

$$\bar{\varepsilon} = \frac{1}{n}\sum_{i=1}^{n}\left|\frac{y_i - \hat{y}_i}{y_i}\right|100.$$

If the error is no more than 10-15%, the model is deemed sufficient.

Brick and reinforced concrete panel walls are damaged linearly depending on the service life and with an approximately equal intensity of 0.7% per year (Fig. 3). After 60 years of service life, about half of the walls get damaged. The intensity of damage to brick walls is described by a linear relationship. The accuracy of the approximation is high (R^2 = 0,967).

The damage-time relationship of reinforced concrete floors and coatings is approximated by a second-degree polynomial close to a linear relationship of 1% per year (Fig. 4). The coefficient of determination R_2 = 0,95 is indicative of a high degree of accuracy of the model.

More than half of the damage to the slabs is due to soaking due to roof leaks and wear of internal communications. After 50 years of service life, about 40% of the plates get damaged, and after 50 years of service life, almost all the plates get damaged.

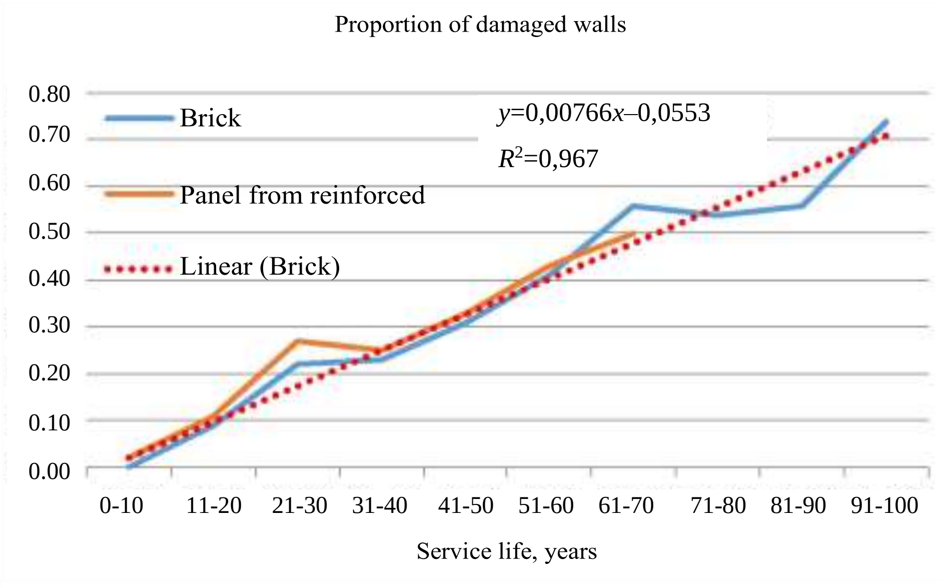

Fig. 3. Dependence of the wall damage on service life

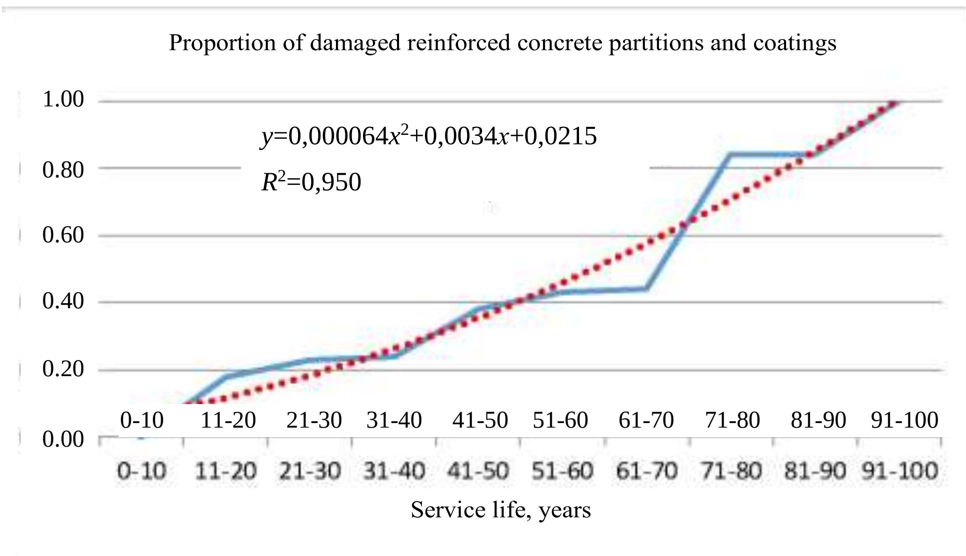

Fig. 4. Dependence of the floor damage on service life

The actual dependence of the proportion of damaged steel trusses (given the truss connections, rather than corrosion) is best modeled by a linear relationship (Fig. 5). The value of the coefficient of determination R_2 = 0.939 is indicative of a high accuracy of approximation.

As can be seen, unless corrosion is considered, about half of the steel trusses are damaged after 55 years of service life. Given the corrosion damage, 70% of steel trusses will have been damaged after 50–60 years of service life, and almost all trusses after 100 years of service life. Half of the damage to steel trusses is due to corrosion caused by roof and gutter leaks. Local bends of the elements have a frequency of 9.8%, the total bending of the rods is 5.9%.

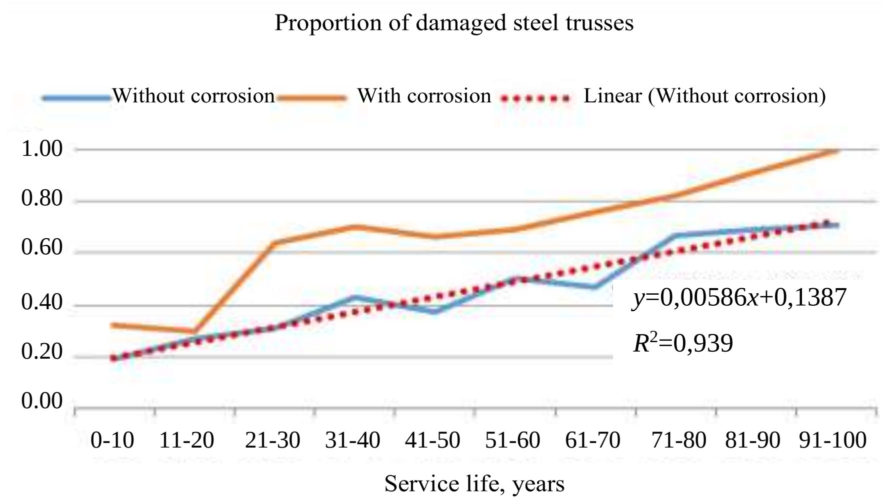

Fig. 5. Dependence of the steel truss damage on service life

From the obtained dependences of damage to the main load-bearing structures (Fig. 2–5), it can be concluded that the maximum service life of industrial buildings is 70-80 years long. By this time, the wear of load-bearing structures reaches 60–70%. During this time, the production technology has progressed significantly, which most frequently requires significant revamping of the production facilities.

The damage rate of steel crane beams does not directly depend on the service life of a building itself, as crane beams are the most damaged structures and are subject to periodic replacement. The most frequent damage to beams is metal corrosion (25.7%), broken fasteners (19.6%) and local damage to shelves and stiffeners (4.3%).

After averaging the damage data over 10 years (Fig. 6), it was found that the beams were changed with a frequency of about 20 years.

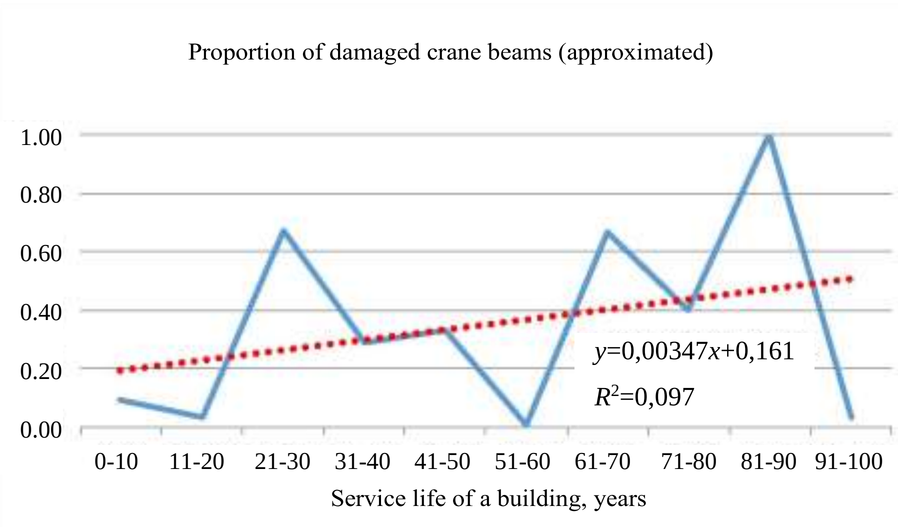

Fig. 6. Dependence of the average damage of crane beams on service life of a building (given a replacement)

According to the broken graph, the minimum damage values correspond to 20, 40, 60, 80 years of service life of a building. The time trend shows that as service life increases, so does the intensity of damage to crane beams, which might be due to a decrease in the overall rigidity of the frame over time [8, 16].

This frequency of beam replacement is confirmed by studies [17, 18] that revealed cracks in the first 20 years of service life of a beam. After 25–50 years of service life, the number of beams with cracks increases up to a quarter. While substituting 25 years of service life into the linear model (Fig. 6), a damage rate of 24.8% is obtained for crane beams, which almost coincides with the data [17].

The analysis showed that service life of crane beams can be divided into conditionally permanent time periods: after up to 25 years of service life, the average damage rate of beams reaches 30% resulting in repairs and restoration, leading to a twofold reduction in the number of damaged structures [6]. According to the graph, such conditionally permanent time periods are about 20 years long. At the same time, with intensive crane operation, periods of repairs and replacement of crane beams are reduced to 5–10 years [9].

Discussion and Conclusion. The results of the study of correlations of damage and service life are based on the previously conducted and published studies by the author of the frequency of damage to structures of industrial buildings [19]. The rate of damage to steel columns of industrial buildings is higher than that of reinforced concrete. The intensity of damage to steel columns is described by an exponential dependence, while that of reinforced concrete columns is linear. The dependence "damage – service life" of reinforced concrete floors and coatings is approximated by a second-degree polynomial close to a linear dependence of 1% per year. The same linear damage is typical of steel trusses. Brick and reinforced concrete walls are damaged linearly depending on service life and with approximately equal intensity — 0.7% per year. Crane beams are the most damaged frame structures and are subject to periodic replacement after an average of 20 years of service life (a specific frequency depends on the operating mode of cranes). The damage rate of crane beams does not directly depend on service life of a building itself. The time trend has shown that as service life of a building increases, so does the intensity of damage to crane beams, which might be due to a decrease in the overall rigidity of the frame and wear of lifting equipment over time.

According to the results of the study of a sample of industrial buildings, the implicit features of damage accumulation have been identified. Thus, for widespread steel columns and coating trusses, in comparison with reinforced concrete ones, defects and damage were observed as early as at the initial stage of operation. Damage after 0 to 10 years of service life for columns is 10% of the total number, damage to the coating trusses is 19–32% of the total number. What is more, the damage to the steel columns of buildings of less than10 years of service life turned out to be greater than for those after 10–20 years of service life. It is obvious that defects and damage at the initial stage of operation are due to the mistakes made during the QMS and acceptance control. Metal structures at the stages of storage, installation, and engineering systems are damaged and accumulate defects (shelf bends, damage to protective primer, fire protection, defects in bolted and welded joints) that are not eliminated in a timely manner before as object is put into operation. The analysis of the inspections and examinations has identified the impact of the quality of the foundations for columns on a building frame — deviations cause mismatches in holes, gaps in connected elements of a metal frame that appear at the assembly stage due to the lack of recommendations developed in project documentation for eliminating them, no additional flat and wedge-shaped gaskets and elongated bolts in standard kits.

In order to increase the period of safe operation and intervals between repairs for newly erected industrial buildings with a metal frame, it is essential to provide measures in advance or as part of maintenance for reducing the number of defects and damage. It is recommended that a risk-based approach is applied to determining such measures considering forecast likelihood of mistakes, frequency of defects and damage as well as restoration costs [20].

The resulting mathematical models of "damage-service life" can be employed in optimizing maintenance, planning periodic inspections, ongoing and major repairs of buildings as well as in developing a strategy for enterprises to replace fixed industrial assets.

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***Claimed contributorship*:**

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AN Potapov: scientific supervision, analysis of the research results, correction of the conclusions.

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Заявленный вклад соавторов:

Д.А. Байбурин: поиск и анализ литературы, разработка цели и задачи исследования, подготовка и обработка статистического материала, подготовка основного текста, формирование выводов.

А.Н. Потапов: научное руководство, анализ результатов исследований, корректировка выводов.

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Original Empirical Research

Innovative Technology of Preparation of Concrete Mixtures

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Abstract

Introduction. Using the example of the production of road concrete in the Rostov region, the article looks at the issues of increasing the efficiency of the technology for preparation of concrete mixtures to ensure the normative indicators of the physical and mechanical properties of concrete without increasing cement consumption and additional costs for reconstruction of existing enterprises in the construction industry.

The aim of the study is to develop a set of prescription and technological measures to ensure regulatory requirements for strength characteristics of concrete produced with the use of local raw materials with no technical re-equipment of existing concrete mixing plants.

Materials and Methods. To this end, while conducting the experimental research, a local raw material base was used that is widely available in the construction industry. The study aimed at developing prescription and technological measures for improving the production efficiency of the road concrete under study was performed using standard methods for assessing the normalized properties of similar materials, concrete mixtures and hardened concrete. In order to optimize the parameters of the developed formulation and technological solutions, mathematical modeling methods of the stochastic system under study based on the theory of experimental planning were applied.

Research Results. The developed innovative technology for preparing concrete mixtures enables an increase in the strength of the concrete under study by 15–20%, or reduce cement consumption by 10–12% without a reduction in the strength of the resulting material.

The results were achieved by means of developing a set of prescription and technological measures including introduction of the suggested carbonate micronutrient and polycarboxylate superplasticizer into the concrete mixture in combination with the developed two-stage method for preparing the concrete mixture.

Discussion and Conclusion. The novelty of the developed technology for preparing concrete mixtures is protected by an invention patent. The results were achieved by means of the combined introduction of the developed limestone-shell filler and superplasticizer ST 5.0 into the concrete mixture in combination with an innovative two-stage technology for preparing concrete mixtures.

Based on the general theoretical understanding of the structure formation of cement concretes as well as of their strength properties, it is possible to extend the results of these studies to production of a broad range of monolithic and prefabricated products as well as structures that are to comply with the regulatory requirements for not only compressive strength, but also for tensile strength. Naturally, in each specific case, additional research will be required that would take into consideration the specifics of the regional raw material base as well as the specifics of a particular enterprise in the construction industry and its product range.

Keywords: road cement concrete, technology of concrete mix preparation, carbonate microfiller, superplasticizer

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Оригинальное эмпирическое исследование

Инновационная технология приготовления бетонных смесей

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Аннотация
Введение. В статье на примере производства дорожного бетона в Ростовской области рассмотрены вопросы повышения эффективности технологии приготовления бетонных смесей, обеспечивающей нормативные показатели физико-механических свойств бетона без увеличения расхода цемента и дополнительных затрат на реконструкцию действующих предприятий строительной индустрии.
Цель проведенных исследований — разработка эффективных рецептурно-технологических мероприятий, обеспечивающих нормативные требования к прочностным показателям бетона, изготавливаемого на основе местной сырьевой базы без технического перевооружения действующих бетоносмесительных заводов.
Материалы и методы. Для достижения поставленной в работе цели при проведении экспериментальных исследований использована местная сырьевая база, доступная для широкого применения на предприятиях строительной индустрии. Исследования, направленные на разработку рецептурно-технологических мероприятий по повышению эффективности производства исследуемого дорожного бетона, выполнены с применением стандартных методик оценки нормируемых показателей свойств сходных материалов, бетонных смесей и затвердевшего бетона. Для оптимизации параметров разработанных рецептурно-технологических решений применены методы математического моделирования исследуемой стохастической системы на основе теории планирования эксперимента.
Результаты исследования. Разработанная инновационная технология приготовления бетонных смесей обеспечивает возможность на 15–20 % повысить прочностные показатели исследуемого бетона, либо на 10–12 % уменьшить расход цемента без снижения прочности получаемого материала.
Полученные результаты достигнуты за счет разработки комплекса рецептурно-технологических мероприятий, включающего введение в бетонную смесь предложенного карбонатного микронаполнителя и поликарбоксилатного суперпластификатора в сочетании с разработанным двухстадийным способом приготовления бетонной смеси.
Обсуждение и заключение. Инновационная новизна разработанной технологии приготовления бетонных смесей защищена патентом на изобретение. Полученные результаты достигнуты за счет совместного введения в бетонную смесь разработанного микронаполнителя на основе известняка-ракушечника и суперпластификатора ST 5.0 в сочетании с инновационной двухстадийной технологией приготовления бетонной смеси.
Исходя из общетеоретического представления о структурообразовании цементных бетонов и формировании их прочностных свойств, можно распространить результаты данных исследований на производство широкого круга монолитных и сборных изделий и конструкций, которые должны удовлетворять нормативным требованиям по прочности не только на сжатие, но и на растяжение. Естественно, в каждом конкретном случае потребуются дополнительные исследования с учетом специфики региональной сырьевой базы и специфики того или иного предприятия строительной индустрии и номенклатуры выпускаемой продукции.

Ключевые слова: дорожный бетон, технология приготовления бетонной смеси, карбонатный микронаполнитель, суперпластифкатор

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Introduction. The article deals with the current issues of improving the efficiency of the technology for preparing concrete mixtures to ensure the normative indicators of the physical and mechanical properties of concrete without an increase in cement consumption and investment costs for renovation of existing construction industry enterprises in the context of the conditions of road concrete production. The relevance of the issues is due to concrete road surfaces becoming more broadly used in most developed countries. The trend is due to the annual increase in the number of vehicles on roadways, an increase in their load-bearing capacity and speed of cargo flows in motorways.

Given all of the above, the issues of the quality of raw materials for road concrete production with the appropriate physical and mechanical properties are gaining momentum [1].

At the same time, deposits of fine sands dominate in lots of regions, with their use in the concrete composition causing increased cement consumption. As a result of numerous previous studies [2–5], methods have been developed to reduce cement consumption in concretes on fine sands by means of introducing micro-fillers into the concrete mixture, such as finely ground sand, ground metallurgical slags and other natural or man-made mineral components. However, the process of grinding these raw materials to produce a micron filler is highly energy-intensive, which ultimately increases concrete costs.

There are also known methods for reducing cement consumption by intensifying the process of mixing concrete mixture [6–9], however, these methods require replacing existing concrete mixers with high-speed ones in most cases, which requires large investment costs.

The aim of these studies was to develop comprehensive formulation and technological solutions to ensure production of concrete mixtures for road concrete at existing enterprises in the construction industry with no additional investments needed for modernization and technical re-equipment or an increase in cement consumption.

Based on the analysis of previous studies [10–15], a working hypothesis was set forth that this aim can be achieved by using micro-filler additives in road concrete composition in combination with optimizing the method of preparing concrete mixtures for road concrete at existing concrete mixing plants. The following tasks were thus solved in stages:

1. Selecting modifying additives of domestic production in the concrete mix.
2. Searching for an effective micro-filler based on a local raw material base.
3. Developing an innovative technology for preparing concrete mixtures with no technical re-equipment of existing concrete mixing units.

Materials and Methods. The road concrete of the design class in terms of compressive strength B35 and flexural tensile strength was adopted as the object of research B_{tb} 4.0.

The following materials were used in the study.

Portland cement produced by Sebryakovcement CEM I 42.5 N with a normal NG density of 26.75 and an actual activity of 29.2 MPa at the age of 2 days, 54.8 MPa at the age of 28 days was used as a binder.

Sandstone crushed stone with a fraction size of 5–20 mm produced by Donskoy Kamen LLC in compliance with the requirements of GOST 26633-2015 was used as a coarse aggregate for manufacturing the road concrete under study.

According to the regulatory documents, sands with a modulus of size $M_к$ = 1.5–3.0 should be used for road concrete. High-modulus sands are not produced in the Rostov region. The delivery of such sands from other regions leads to a significant increase in the cost of concrete mixtures. The most suitable local sand in terms of characteristics is the sand of the Kayalsky quarry with a $M_к$ equal to 1.57 used as a fine aggregate.

A polycarboxylate-based superplasticizer ST 5.0 manufactured by BSR LLC, St. Petersburg, was selected for plasticization of the concrete mixture based on previous information. According to the manufacturer, the specified superplasticizer is characterized by the data in Table 1.

Table 1

Characteristics of the additive ST 5.0

Indices, measurement units	Values
Appearance	Brown liquid
Smell	Weak
Total content of the solid phase, %	25
Density of the solution, g/cm^3	1.070 ± 0.03
pH (undiluted product)	9 ± 1.5

The developed micro-filler for the road concrete under study was obtained using pontic limestone-shell rock from the Kamenskoye deposit in the Rostov region, crushed to a powdery state.

The original limestone-shell rock is a yellow-brown rock, mostly consisting of slightly cemented fragments of shells of various sizes. The content of calcium carbonate in the rock ranges from 87 to 97%. The average density of rock pieces is in the range of 1.5–2.2 t/m^3, depending on the deposit depth. Compressive strength is up to 25 MPa, rock porosity is 15–40% by volume.

Methodological studies aimed at achieving this aim were performed in stages in compliance with the working hypothesis and the suggested research aims. At the same time, use of standard methods for identifying the normalized properties of the original materials, characteristics of the resulting concrete mixtures as well as strength parameters of the hardened concrete were combined by means of mathematical methods of experimental planning theory, polynomial modeling and interpretation of the resulting response functions.

Fig. 1. Appearance of shell limestone

At the first stage of the experimental studies, the effect of the degree of crushing of powdered limestone-shell and its content in the mixture on the strength properties of the mortar part of concrete was examined. The experiments were performed on cement-sand samples — beams measuring 40×40×160 mm with its material compositions modelling the mortar part of the concrete under study.

After the samples had hardened under normal conditions at the age of 28 days, they were subjected to strength tests by means of conventional bending and compression methods.

Based on the results of the strength tests and corresponding mathematical processing of the experimental data using methods of planning theory [16], polynomial models were obtained that adequately describe the dependencies under study at a 5% significance level that were used to identify the rational dosage ranges of the resulting carbonate micron filler based on crushed limestone-shell.

At the second stage of the study, considering the results at the first stage, in order to optimize the technological parameters of preparing the concrete mixtures, experiments were performed on concrete samples-prisms measuring 100×100×400 mm tested at specified times by means of the standard methods for identifying the compressive and flexural strength of the concrete under study.

Research Results. At the first stage of the study conducted on a cement-sand mortar with a ratio of C:S = 1:1.7 corresponding to the mortar part of the road concrete, quadratic polynomial models of the effect of the degree of crushing of the carbonate filler and its quantitative content in the mixture on the flexural and compressive strength of the resulting material were obtained by means of the methods of experimental planning theory. As a result of interpreting the obtained models, it was found that the optimal dosage of the micron filler under study ranges from 60 to 100 kg/m^3 while grinding the original limestone shell to a specific surface area of at least 2000 cm^2/g.

As a result of further studies, an innovative two-stage technology for preparing concrete mixtures with a limestone-shell filler was developed. The priority novelty of this development is protected by the patent of the Russian Federation for invention [17].

The essence of the developed two-stage technology is as follows. At the first stage of the preparation of the concrete mixture, small and large aggregates, as well as micro-fillers, were loaded into the concrete mixer at specified dosages.

The above components were mixed in a dry state until a homogeneous dry mixture was obtained.

At the second stage, Portland cement, water, and a superplasticizer were added to the resulting dry mixture, after which all of the components were mixed until a concrete mixture of the required workability was obtained.

The composition of the concrete mix with a consumption of 420 kg/m^3 of Portland cement with the following ratio between the components (by weight) was adopted as the original one C:N:L = 1:1.69:2.71.

The consumption of the superplasticizer additive ST 5.0 (an aqueous solution of 25% concentration) was 1% by weight of Portland cement. The consumption of the filler additive varied from 40 to 120 kg/m^3 and was introduced into the mixture due to a corresponding reduction in sand consumption. The amount of mixing water in each case was selected from the conditions for obtaining equally mobile mixtures of grade P2 (5–9 cm).

The compositions of the concrete mixtures, methods of preparing them as well as the results of the strength tests of concrete at the age of 28 days of normal hardening are shown in Table 2.

Table 2

Compositions of the concrete mixtures, technology of preparing them and the concrete strength at the age of 28 days of normal hardening

No. of a test	Mixture preparation method	Material consumption, kg/m^3						Concrete strength, MPa	
		cement	water	limestone	sand	ST 5.0	microfiller	R_{tb}	R_b
1	Single-stage	420	168	1140	710	4.2	0	5.1	45.2
2	Two-stage	420	172	1140	650	4.2	60	6.0	51.8
3		420	174	1140	630	4.2	80	6.3	53.9
4		420	176	1140	610	4.2	100	5.8	49.7
5	Two-stage with reduced cement consumption	370	170	1140	680	3.7	80	5.2	45.6

В результате проведенных исследований установлено, что разработанная инновационная технология приготовления бетонных смесей повышает прочность бетона на 15–20% (составы No 2, 3, 4) или позволяет на 12% снизить расход цемента (состав No. 5) по сравнению с одностадийной технологией (состав No. 1) без ухудшения прочностных свойств дорожного бетона. Наглядно эти результаты представлены на рис. 2.

As a result of the study, it was found that the developed innovative technology for preparing concrete mixtures increases concrete strength of by 15–20% (compositions No. 2, 3, 4) or reduces cement consumption by 12% (composition No. 5) compared with the single-stage technology (composition No. 1) without compromising the strength properties of road concrete. These results are clearly shown in Fig. 2.

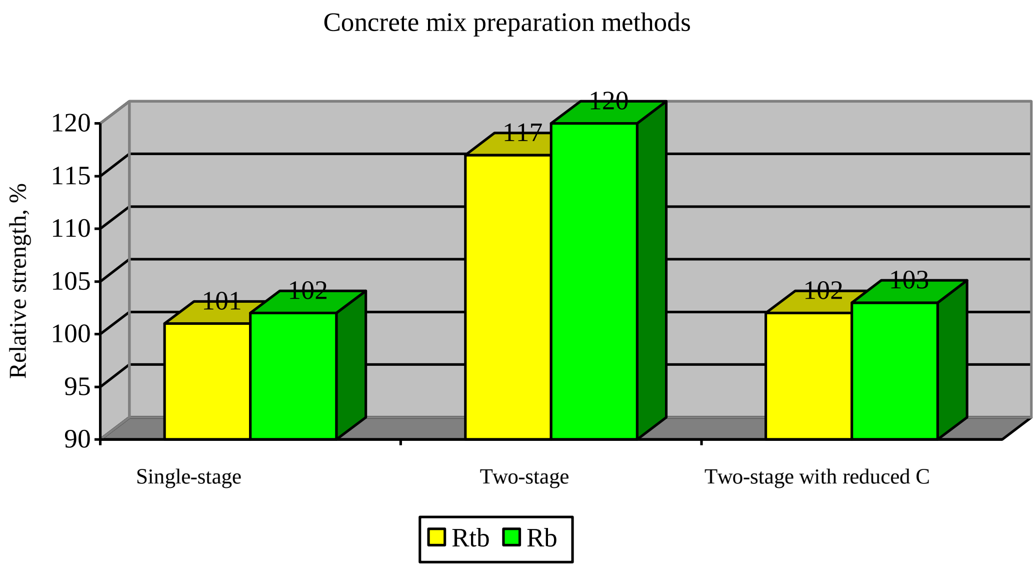

Fig. 2. Dependence of concrete strength parameters on the technology of concrete mix preparation

Discussion and Conclusion. The study results ensure road concrete production in compliance with the regulatory requirements for its strength characteristics, with no additional costs for technical re-equipment of existing concrete mixing units and based on a local raw material base.

The results were achieved by means of a combined use of a micro-filler developed using crushed limestone-shell and superplasticizer ST 5.0 in combination with an innovative two-stage technology for concrete mixture preparation.

The innovative technology of manufacturing concrete mixes for road concrete developed in compliance with the study aims enables a reduction in cement and superplasticizer consumption at existing enterprises of the construction industry with no additional costs for reconstruction and technical re-equipment or compromising product quality.

The suggested set of prescription and technological measures to improve the efficiency of concrete mix production technology can be extended to mass production of concrete mixtures for general construction purposes. At the same time, for them to be introduced into production, technological solutions should be adjusted taking into account the requirements for products, local raw material base as well as specifics of existing enterprise, which can be attained at the factory laboratory.

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URBAN PLANNING, PLANNING OF RURAL SETTLEMENTS

ГРАДОСТРОИТЕЛЬСТВО, ПЛАНИРОВКА СЕЛЬСКИХ НАСЕЛЕННЫХ ПУНКТОВ

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Original Empirical Research

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An Organizational and Technological Model for Construction Waste Management in Dense Urban Development

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Abstract

Introduction. The research is devoted to developing effective methods for managing construction waste on residential sites in dense urban areas. Innovative approaches to waste management are set forth, including separate collection, digitalization of processes and modern primary treatment technologies in compliance with the requirements of ecology and resource conservation.

Materials and Methods. The study was conducted using the method of aggregated calculations of waste volume using the example of an actual housing construction facility with an area of 20,000 m^2. The assessment was performed based on the distribution of waste by categories and stages of production (excavation, monolith, installation, engineering networks, finishing, landscaping). The average coefficients of waste generation and material density were used in oder to calculate the total mass of each fraction. A comparative analysis of two logistics scenarios was carried out: a traditional one (collecting all waste together and sending it to the landfill) and an innovative one (separate collection, on-site pretreatment, recycling priority and digital accounting).

Research Results. The significant economic and environmental effect of the introduction of an innovative approach to construction waste management is indicated. Compared to the basic scenario ("mixed collection → landfill"), the innovative scheme ("separate collection + on-site processing + recycling priority + digital registration") provides a 51% reduction in landfill volumes and a 50% reduction in traffic. The major success parameter is the volume of soil reuse, which reduces the mass of recyclable waste and the number of dump truck trips by almost a half. The financial benefit is complemented by the possibility of commercial sales of secondary resources and introducing digital control tools. The results are presented in tables displaying the monthly dynamics of waste generation. Recommendations on containerization and assessment of the impact of the assumptions on the final indicators are provided.

Discussion and Conclusion. A construction waste management methodology has been developed that reduces the volume of disposal, transport load and provides savings by optimizing on-site recycling. An increase in the level of secondary use and effective primary waste treatment has been ensured.

Keywords: construction waste, waste management, waste sorting, recycling, digital platforms, mobile sorting complexes

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Оригинальное эмпирическое исследование

Организационно-технологическая модель управления строительными отходами в условиях плотной городской застройки

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Аннотация
Введение. Исследование посвящено разработке эффективных методик управления строительными отходами на площадках жилой застройки в условиях плотной городской застройки. Предложены инновационные подходы к обращению с отходами, включающие раздельный сбор, цифровизацию процессов и современные технологии первичной обработки, соответствующие требованиям экологии и ресурсосбережения.
Материалы и методы. Исследование проведено методом укрупнённых расчётов объёма отходов на примере реального объекта жилищного строительства площадью 20 тыс. м². Оценка проведена на основе распределения отходов по категориям и стадиям производства (земляные работы, монолит, монтаж, инженерные сети, отделка, благоустройство). Использованы усреднённые коэффициенты образования отходов и плотности материалов, позволяющие рассчитать итоговую массу каждой фракции. Проведен сравнительный анализ двух сценариев логистики: традиционного (сбор всех отходов вместе и отправка на полигон) и инновационного (раздельный сбор, предобработка на площадке, приоритет переработки и цифровой учёт).
Результаты исследования. Показан существенный экономический и экологический эффект от внедрения инновационного подхода к управлению строительными отходами. По сравнению с базовым сценарием («смешанный сбор → полигон»), инновационная схема («раздельный сбор + обработка на площадке + приоритет переработки + цифровая регистрация») обеспечивает снижение объемов захоронения на 51 %, уменьшение транспортного трафика на 50 %. Ключевым параметром успеха являются объемы повторного использования грунта, что снижает массу перерабатываемых отходов и количество рейсов самосвалов почти вдвое. Финансовая выгода дополняется возможностью коммерческой реализации вторичных ресурсов и внедрением цифровых инструментов контроля. Результаты представлены в таблицах, демонстрирующих помесячную динамику образования отходов. Даны рекомендации по контейнированию и оценке влияния допущений на итоговые показатели.
Обсуждение и заключение. Разработана методика управления строительными отходами, сокращающая объём захоронения, транспортную нагрузку и обеспечивающая экономию за счёт оптимизации переработки на месте. Обеспечено повышение уровня вторичного использования и эффективной первичной обработки отходов.

Ключевые слова*:* строительные отходы, управление отходами, сортировка отходов, переработка, цифровые платформы, мобильные сортировочные комплексы

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Introduction. The relevance of the research is due to the scope of the problem of a significant amount of construction waste typical of modern urban construction, particularly in limited urban construction areas. The high intensity of construction activity generates a large volume of construction debris, creating a considerable burden on the environment and urban infrastructure. Modern cities are faced with the urgent need of addressing issues of rational waste management due to the lack of temporary waste storage, increased requirements for environmental protection and sustainable development [4, 5].

Modern urban planning policy is aimed at developing a closed-loop economy, which provides for a significant increase in the level of recycling of building materials (up to 40% by 2030) [4, 11]. This poses major challenges for new approaches to waste management to be developed, including introducing innovative technologies and practices [5, 12].

Thus, this study addresses the problem of developing a comprehensive strategy for construction waste management aimed at reducing the environmental burden, increasing resource conservation and improving organizational as well as technological approaches to waste management in construction sites [2–4].

The object of the study is waste management organization in a construction site of a multi-storey residential building in dense urban environment. The subject of the study are innovative methods of collection, classification, processing and subsequent disposal of construction waste in specific construction sites.

The aim of the study is to justify and develop efficient solutions for optimizing construction waste management in construction of multi-storey residential buildings in dense urban environments, including modern sorting and recycling technologies in compliance with the environmental standards and restrictions of an urban environment.

To this end, the following research tasks were formulated:

1. Investigating the laws and regulations governing waste management in urban construction sites.
2. Analyzing the types and volumes of waste generated at each stage of residential building construction.
3. Studying modern methods of collecting, sorting and recycling construction waste, including digital management and mobile primary processing.
4. Developing practical recommendations for improving waste disposal during housing construction in dense urban environments.

Urban development is characterized by a significant amount of construction waste generated in cramped conditions, with strict restrictions on noise and traffic levels, as well as the lack of opportunities for their disposal [5, 8]. The traditional method of removing unsorted garbage leads to increased transportation costs, environmental risks, and downtime in construction sites [5].

The main types of construction waste include:

– dismantling waste (brick, concrete, structural fragments);
– excavation waste (excess soil, sand-gravel mixtures);
– construction waste (concrete crumbs, pieces of reinforcement, packaging);
– finishing waste (tile clippings, drywall, paint and varnish remnants).

Managing these flows is complicated by the peculiarities of urban development: lack of space, strict transportation rules and restrictions on the mode of work [5, 8]. In order to comply with the established standards, clear regulations for cleaning, separate collection and prompt disposal of waste are needed to minimize the negative impact on the urban environment and reducing the developer’s costs [2].

The digital waste accounting system helps automate monitoring and increase transparency of waste management [5, 12]. The example of the Moscow AIS OSSiG (Automated Information System Waste of Construction, Demolition and Soil) demonstrates the effectiveness of automatic tracking of dump truck flights using GPS sensors and electronic document management [8]. The system records movement of goods and the point of delivery, preventing illegal dumping of garbage and ensuring compliance with the environmental standards.

Commercial solutions expand functionality by offering IoT technologies (container occupancy sensors, weight sensors of equipment), increasing the accuracy of route planning and reducing logistics costs [10, 12]. Electronic marketplaces enable exchange of recyclable materials between objects reducing the total amount of waste [11].

The environmental significance of the issue is emphasized by the statistics: construction waste accounts for roughly 30% of the total municipal solid waste, which indicates the importance of accounting for such a volume of garbage. Large-scale urban reconstruction and new housing stock annually require solving the problem of recycling thousands of tons of building materials [1, 4]. At the same time, recycled construction waste is a valuable source of recyclable materials needed by industry [2, 9].

The increased state requirements are stipulated by Federal Law No. 89-FZ dated 24/06/1998 "On Production and Consumption Waste" that provides for waste management standards, including control over movement of construction debris. Regional authorities are actively introducing a system for monitoring movement of waste from the site to landfills, combating illegal dumping and unauthorized landfills [5, 8].

Thus, creating a construction waste management model combining separate collection, on-site pretreatment and efficient logistics becomes a prerequisite for compliance with modern environmental standards, reducing the burden on landfills and saving natural resources [2, 5, 9].

The research focuses on analyzing the compositions and volumes of construction waste at different stages of residential building construction in dense urban environments [3, 5]. Basic and improved waste management schemes are being considered, organizational and technical procedures (containerization schedules, RACI responsibility allocation, key KPIs) are being developed, and a feasibility study of each option is being conducted [2, 7, 12].

The practical value is a possibility of applying the developed model to similar projects of mass urban development [5, 8].

Materials and Methods. The calculations for the object were performed on a large scale due to the lack of detailed logs of mass volumes of work. The volume of waste is estimated based on the specific area of the object (20,000 m^2) and types of work (monolith, finishing, etc.) followed by categorization (inert, metal, wood, polymers, mixed material, waste of hazard class III) [3, 5]. Logistics is organized taking into account limitations of a site (one-way entrance, wheel washing complex, temporary entry windows for heavy equipment) [5, 8].

The basic option is the traditional mixed collection of all types of construction debris followed by shipment to landfills. An innovative alternative involves introducing a separate waste collection system in a construction site, pretreatment and priority transfer of suitable materials for recycling by means of digital technologies for controlling traffic flows and waste mass accounting [12].

The calculations were carried out for a specific object (Building 5 at 11 Kantemirovskaya Str., Saint Petersburg) using common initial values of V_i differing only in the organization of the waste stream and proportion of their subsequent reuse.

The calculation method is universal: each type of work corresponds to its own specific waste generation index (coefficient k_i), which makes it possible to identify the total mass of waste:

$$Q_i = k_i \times V_i$$

that is further distributed into the main fractions (inert materials, metals, wood, polymers, a mixture of finishing materials, hazardous waste).

The assumptions in the calculations are as follows:

– the excess soil makes up 25% of the total excavation (density 1.6 t/m³);
– residual concrete — 0.6 % of the volume to be poured (2.4 t/m³);
– rebar scrap — 1.5 % of the total weight;
– packaging of enclosing structures — 0.6 kg/m²;
– the excess of engineering supplies: packaging — 1.5%, technological trimmings — 2% by weight,
– finishing works: a mixed group — 3.5 kg/m², hazardous waste (containers of paints and polishes) — 0.12 kg/m²;
– landscaping: other waste — 0.8 kg/m².

The volume of work on the site: excavation — 12,000 m³, monolithic concrete — 10,000 m³, reinforcement — 1,500 tons, facade fences — 14,000 m², engineering networks — 300 tons, finishing works — 22,000 m², landscaping — 5,000 m² (Table 1).

Table 1

Calculation of waste generation by stages and fractions

Stage → fraction	Unit (V_i)	Initial data / coefficient	Mass (Q_i), t
Ground → excess soil	m³	12,000 × 25 % × 1.6 t/m³	4,800.00
Monolith → inert (remnants of concrete/mortar)	m³	10,000 × 0.6 % × 2.4 t/m³	144
Monolith → metal (rebar scrap)	t	1,500 × 1.5%	22.5
Enclosing/mounting → packaging	m²	14,000 × 0.6 kg/m²	8.4
Engineering systems → packaging	t	300 × 1.5 %	4.5
Engineering systems → technological trimmings	t	300 × 2.0 %	6
Finishing → mixed	m²	22,000 × 3.5 kg/m²	77
Finishing → hazardous (LKM packaging, class III)	m²	22,000 × 0.12 kg/m²	2.64
Landscaping → other	m²	5,000 × 0.8 kg/m²	4
Total			5,069.04

The bulk of construction waste is made up of soils formed during excavation. The total volume of extracted soil reaches 5,069.04 tons, which corresponds to 253 kg per 1 m² of a building. The significant specific gravity of soils is crucial to the strategy of their disposal and transportation, having a major impact on the key project performance indicators (KPIs). An analysis of the accumulation dynamics shows an intensive increase in the volume of waste in the initial months due to excavation work ("the ground peak") followed by a smooth increase in volumes due to monolithic work, facade operations and interior decoration of an object (Table 2). This determines the need for accurate planning of loading and unloading operations and timely release of the site for further construction and installation.

In order to make it possible to reproduce the calculations and verify the validity of the accepted assumptions, the coefficients and densities used in a separate table are summarized (Table 3).

The transition from waste mass calculations to logistics issues clearly illustrates the need for vehicles with the required body volume and number of flights (Table 4). As for the light fractions (such as packaging and mixed waste), the key factor is not their mass rather the volume occupied. The use of presses to seal cartons, films, and other lightweight materials significantly reduces container turnover, efficiently saving transportation resources and reducing the number of trips required.

Table 2

Monthly waste generation by stages (t/month)

Month	Ground	Monolith (inert + rebar)	Enclosing/ assembly	Engineering	Finishing	Landscaping	Total
1	1,600	0	0	0	0	0	1,600.00
2	1,600	0	0	0	0	0	1,600.00
3	1,600	16.65	0	0	0	0	1,616.65
4	0	16.65	0	0	0	0	16.65
5	0	16.65	0	0	0	0	16.65
6	0	16.65	0	0	0	0	16.65
7	0	16.65	0	0	0	0	16.65
8	0	16.65	0	0	0	0	16.65
9	0	16.65	1.05	0	0	0	17.70
10	0	16.65	1.05	0.95	0	0	18.65
11	0	16.65	1.05	0.95	0	0	18.65
12	0	16.65	1.05	0.95	7.24	0	25.89
13	0	0	1.05	0.95	7.24	0	9.24
14	0	0	1.05	0.95	7.24	0	9.24
15	0	0	1.05	0.95	7.24	0	9.24
16	0	0	1.05	0.95	7.24	0	9.24
17	0	0	0	0.95	7.24	0	8.19
18	0	0	0	0.95	7.24	0	8.19
19	0	0	0	0.95	7.24	0	8.19
20	0	0	0	1	7.24	0	8.24
21	0	0	0	0	7.24	0	7.24
22	0	0	0	0	7.24	1.33	8.57
23	0	0	0	0	0	1.33	1.33
24	0	0	0	0	0	1.34	1.34
Итого	4,800.00	166.5	8.4	10.5	79.64	4	5,069.04

Table 3

Accepted coefficients and densities

Parameter	Accepted value	Commentary
Excess soil	25 % from the excavation; $\rho = 1.6$ t/m³	Only the exported volume is considered
Landscaping – other	0.8 kg/m²	Cutting/packaging

Table 4

Calculation of the number of trips for removing soil

Fraction	Mass, t	Estimated density (bulk)	Volume, m³	Bunkers of 20 m³ (total)	Dump truck trips
Soil	4 800,0	–	–	–	≈ 423 (12 t/trip)

The excess soil is removed from the site exclusively by road — directly by dump trucks to specialized landfills. However, in order to make informed management decisions, it is critical to understand which assumptions have the greatest impact on the outcome of the calculations. Thus a brief sensitivity analysis is carried out below, revealing the most significant factor influencing the outcome of the study (Table 5).

Table 5

Sensitivity of the result to the major assumptions

Assumption change	Previous	New	Δ mass, t	Impact
Excess soil proportion ± 5 units	25 %	20–30 %	± 960	Critical (± 19 % of the total)

The operational readiness of the construction site ensures a clear definition of the hazard classes of waste and the types of containers in use (see Table 6 for an example). Such a measure is necessary for proper implementation of separate collection and further coordination of contract-based waste transfer procedures to specialized organizations. A proper classification and labeling of waste helps to avoid violations of sanitary and environmental standards, minimize the risks of fines and litigation, and contribute to improving waste management quality and compliance with the requirements of the state-run inspection bodies.

Table 6

Fractions, hazard classes and recommended packaging

Fraction	Class	Recommended packaging / area
Soil (non-polluted)	V	Direct export; temporary buffer ≤ 1–2 days

These data provide a reliable basis for comparing the considered scenarios. The main source of uncertainty is the level of reuse of excess soil. To take into account possible scenarios, the calculation is performed according to three reuse scenarios: zero use (0%), partial use (25%) and moderate use (50%) (Table 7). This approach makes it possible to identify the dependence of the volume of soil deposited, the number of trips and costs on the actual constraints existing in an urban space (quality of the extracted soil, availability of reception points, allowed time of export).

Table 7

Distribution of flows by fractions (reuse / recycling and burial), t (option: 50% reuse of the soil)

Fraction	Total mass	Base: reuse / recycling	Base: burial	Innovations: reuse / recycling	Innovations: burial
Soil (excess)	4,800.00	0	4,800.00	2,400.00	2,400.00

The final structure of waste streams heavily depends on the selected level of soil reuse. The indicators in Table 7 are calculated according to an optimistic scenario with a reuse rate of 50%. This case reflects a situation where there is a proven infrastructure for receiving soil, and favorable conditions for its use are provided.

However, in order to develop sustainable management solutions, it is suggested that an intermediate option with an equity participation of 25% is considered, which seems more realistic for most urban sites. On top of that, a conservative scenario with a complete absence of soil reuse (0%) is being considered.

Research Results. The results should be interpreted taking into account the above three levels: full utilization (0%), partial utilization (25%) [6] and optimal utilization (50%). The specific numerical values will depend on the clarifying calculations (Table 8).

The major criteria for evaluating scenarios are presented in a concise form of a table-map reflecting the four key indicators: proportion of waste involved in reuse, volume of disposal, logistical costs and unit costs per unit of the recycled area.

Table 8

Integral indicators (mass, logistics, specific ones)

Index	Base	Innovations	Effect
Reuse / recycling, t	36.02	2,593.70	2,557.68
Recycling proportion, %	0.71	51.2	50.5
Burial, t	5,033.02	2,475.34	–2,557.68 (–50.8 %)
Mass for export, t	5,069.04	2,553.84	–2,515.20 (–49.6 %)
Dump truck trips (12 t/trip), items	≈ 423	≈ 213	–210 (–49.6 %)
Specific burial, t/m²	0.252	0.124	–0.128
Specific reuse / recycling, kg/m²	1.8	129.7	127.9

For a fuller picture a cost comparison based on approximate tariffs is provided: transport — 9,000 rubles/trip (12 tons), soil placement — 900 rubles/ton (Table 9).

Table 9

Direct cost scenarios (a design example)

Expense item	Unit	Tariff, rub	Base (volume)	Base, rub	Innovations (volume)	Innovations, rub
Transport (dump trucks, 12 t)	trip	9,000	≈ 423	3,807,000	≈ 213	1,917,000
Soil placement	t	900	4,800	4,320,000	2 400	2,160,000
Total direct costs				8,127,000		4,077,000

The financial result ranges from 8.13 million to 4.08 million rubles (approximately a 50% reduction), depending on the proportion of soil reuse (0, 25 or 50%). The example is focused on an optimistic level of 50%; for a basic urban practice (25%) and a conservative scenario (0%), the cost is additionally adjusted using sensitivity analysis (see below for more information) [7]. The actual operating results should be considered in the specified range, rather than relying on a single value.

Operational advantages (non-financial ones):

- reduction in the number of car trips by almost a half (about 210 trips fewer), which facilitates the traffic situation and unloading of departure "slots";
- reduction in peak waste accumulations on site, thereby improving sanitary and hygienic conditions;
- increased transparency of operations: electronic transportation accounting, simplified control, and electronic reporting reduce the likelihood of administrative sanctions and penalties;
- effective handling of pollution (dust, noise) due to short cycles of pre-crushing and pressing of waste.

The sensitivity of the final result. The soil management strategy has the most significant impact on the overall project economy: a change in the share of reuse by only 5 percentage points (e.g., from 25 to 30%) involves those in the order of 960 tons of cargo and about 1 million rubles in savings. The other coefficients (concrete residues, rate of formation of finishing waste) have a less pronounced effect [7].

Thus while clarifying the initial data, it is recommended that the exact amount of excavation and the actual proportion of returned soil is recorded, availability of a mobile crusher is assessed and tariffs for waste transportation and reception at landfills are inspected [8].

The advantages of an innovative scenario. The implementation of an innovative strategy in an urban environment yields significant benefits:

- reduction in the burial volumes by 51 %;
- reduction in the traffic load by almost a half (–50% of trips);
- direct financial savings amount to almost a half of the initial cost of waste management.

Additional measures to enhance the effect include commercial sales of secondary resources (metal, secondary crushed stone) and mandatory digitalization of reporting, which contributes to transparency and reduces risks of errors [9].

The aim of the innovation scenario. In the construction site, it is necessary to organize the online work (Table 10) in such a way that:

– to maintain cleanliness and environmental friendliness: carry out regular cleaning of contaminated areas, cover bunkers and wash wheels at departure;

– to organize the weighing of goods at departure (if there is a weighing unit) or calculate the mass by volume and density with mandatory documentation (photo fixation);

– to carry out logistics planning between peak hours or at night, taking into account the established time "slots";

– to create a margin of flexibility in the departure schedule (+15% trip reserve);

– to use machinery: dump trucks with a lifting capacity of 12 tons for soil and bunkering trucks with a capacity of 20 m^3 for the remaining waste fractions.

Table 10

Weekly operation cycle (an example)

Day	07:00–11:00	11:00–15:00	15:00–19:00	22:00–06:00
Mon	Collection / fraction replacement	Packaging pressing	Loading the inert materials	Soil export (slots)
Tue	Crushing the inert materials	Screen / magnet	Loading of secondary limestone	Mix waste export

The main driver of positive changes is effective management of the largest mass of waste — of land and inert materials. Their local reuse has a direct impact on the total mass of waste and the amount of required transport. The accepted proportion of soil reuse (0, 25 or 50%) determines the extent of the positive effect: with an increase in the percentage of reuse, the volume of burial decreases and that of transported material decreases (Table 11).

Table 11

Total KPIs according to the scenarios (mass, logistics, specific ones)

Index	Base	Innovations	Effect
Accumulated waste mass, t	5,069	5,069	–
Reuse / recycling, t	36	2,593.7	2 557.7 t
Proportion of reuse / recycling, %	0.71	51.2	50.5 units
Burial, t	5,033	2 475.3	–2 557.7 t (–50.8 %)
Mass for export, t	5,069	2 553.8	–2 515.2 t (–49.6 %)
Dump truck trips (12 t/trip), items	≈ 423	≈ 213	–210 trips (–49.6 %)
Specific burial, t/m²	0.252	0.124	–0.128 t/m²
Specific reuse / recycling, kg/m²	1.8	129.7	127.9 kg/m²

E.g., with a 50% reuse rate, approximately 2.5 thousand tons of recycled soil and recycled crushed stone remain on site [6], whereas as the proportion drops, this positive effect naturally subsides.

Economic assessment. The estimated tariffs have been accepted (replaced by the actual ones): transportation — 9,000 rubles/trip (12 tons), soil placement — 900 rubles/ton, trip (Table 12).

Table 12

Direct costs according to the scenarios (a design example)

Expense item	Unit	Tariff, rub	Base (volume)	Base, rub	Innovations (volume)	Innovations, rub
Transport (dump trucks, 12 t)	trip	9,000	≈ 423	3,807,000	≈ 213	1,917,000
Soil placement	t	900	4,800	4,320,000	2,400	2,160,000

Discussion and Conclusion. As a result of the study, an effective construction waste management technique has been set forth that makes it possible to reduce the volume of landfills by 51% as well as the transport impact on the urban environment by 50% and achieve an economic effect: the total savings of about 4.1–4.2 million rubles (≈ 50% of the base), or ≈ 200–210 rub/m^2 of the total area, i.e. 50% relative to the traditional waste management model.

The resulting savings are achieved due to the three main factors:

– reducing the number of trips by almost a half due to localization of some of the processing directly in the construction site;

– reducing the fees for waste disposal at specialized landfills by increasing the share of reuse of raw materials;

– effectiveness of low-cost pre-treatment operations (crushing, pressing) providing additional benefits within the framework of an integrated approach.

At the same time, separate waste collection, mobile primary processing and digitalization of logistics have a major role to play [13]. The factor of reuse of excess soil turned out to be particularly significant: changing its proportion by only 5% can yield additional millions of rubles in savings. The implementation of the suggested concept provides for development of an integrated action program combining environmentally friendly methods, digital registration and competent logistics. The results will enable the construction of residential facilities with less damage to nature and significant savings in financial resources.

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ГРАДОСТРОИТЕЛЬСТВО, ПЛАНИРОВКА СЕЛЬСКИХ НАСЕЛЕННЫХ ПУНКТОВ

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Evolution of Urban Irrigation and Irrigated Urban Environment in Central Asia

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Abstract

Introduction. The article examines the development of urban irrigation in Central Asia as regards the evolution of the urban environment structure. Agricultural irrigation and the history of Central Asian urban planning have been extensively investigated, while urban irrigation has not been sufficiently studied. There are barely any studies looking at landscape irrigation in the chronological context of urban development for the region. Therefore the aim of the study is to identify the stages of development of urban irrigation and irrigated elements of the urban environment as well as the boundaries and characteristic features of each stage in the historical context.

Materials and Methods. The stages of urban irrigation development were identified by analyzing historical and archaeological materials as well as studies on landscape architecture, urban planning, and hydraulic engineering throughout a variety of historical periods.

Research Results. As a result, the stages of development of irrigation and the irrigated urban environment have been identified and described; a chronographic line of irrigation development has been designed; irrigation schemes of cities throughout various historical periods have been designed.

Discussion and Conclusion. The results can be used in order to substantiate the cultural and historical value of the open-channel irrigation in modern cities of Central Asia, as well as to design adaptation measures for cities whose climates are becoming arid.

Keywords: urban irrigation, Central Asia, history of urban planning, irrigation ditch, open-channel irrigation, arid cities

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Оригинальное эмпирическое исследование

Эволюция городской ирригации и орошаемой городской среды в Центральной Азии

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Аннотация

Введение. В статье рассматривается развитие городской ирригации в Центральной Азии во взаимосвязи с эволюцией структуры городской среды. Сельскохозяйственная ирригация и история центральноазиатского градостроительства обширно исследованы, в то время как городская ирригация изучается мало. Исследований, рассматривающих ландшафтное орошение в хронологическом контексте развития города для данного региона, практически нет.

В связи с этим цель настоящей работы — определение этапов развития городской ирригации и орошаемых элементов городской среды и выявление границ и характерных особенностей каждого этапа в историческом контексте.

Материалы и методы. Этапы развития городской ирригации выявлялись путём анализа исторических и археологических материалов, анализа исследований, посвящённых ландшафтной архитектуре, градостроительству и гидротехническому строительству в различные исторические периоды.

Результаты исследования. В результате работы выделены и описаны этапы развития ирригации и орошаемой городской среды; построена хронографическая прямая развития ирригации; построены схемы орошения городов в различные исторические периоды.

Обсуждение и заключение. Результаты работы могут быть использованы в обосновании культурно-исторической ценности арычного орошения в современных городах Центральной Азии, а также при проектировании мер адаптации городов, подверженных изменению климата в сторону засушливости.

Ключевые слова: городская ирригация, Центральная Азия, история градостроительства, арычное орошение, аридные города

Для цитирования. Спирова А.Ю., Карелин Д.В. Эволюция городской ирригации и орошаемой городской среды в Центральной Азии. *Современные тенденции в строительстве, градостроительстве и планировке территорий*. 2026;5(2):83–96. https://doi.org/10.23947/2949-1835-2026-5-2-83-96

Introduction. The area under study covers the northeast of modern Iran (the eastern Caspian Sea, from Tehran to Kashmir), modern Uzbekistan, Turkmenistan, Tajikistan, Kyrgyzstan, northern Afghanistan (including Kabul), southern Kazakhstan, and the western borderlands of China. The geographical boundaries of the study with the designated main river basins are shown in Fig. 1.

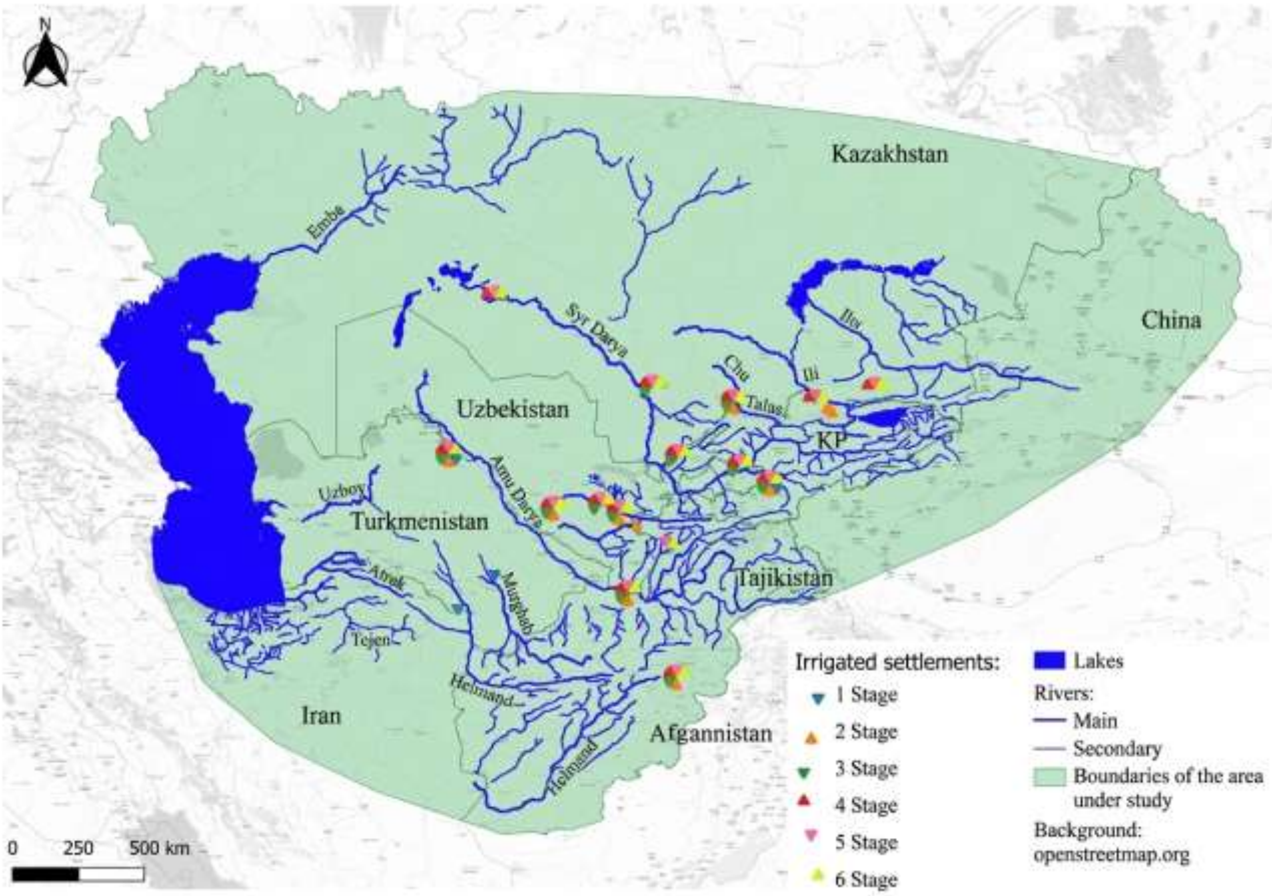

Fig. 1. Geographical boundaries of the study

Urban irrigation in the region is underrepresented in the literature as an independent study. The history of agricultural irrigation has been investigated in great detail [1–4]. Questions pertaining to the history of Central Asian urban planning are time and time again raised by scholars around the world. At the same time, urban irrigation is only mentioned in passing, in spite of the fact that life with no irrigation in many regions of Central Asia is inconceivable even in an urban environment [5]. It was M.I. Ismailov who had considered it in most detail and devoted a few publications to landscape irrigation of the cities of Tajikistan [6, 7]. V.I. Kochedamov described the architecture of sardobes and houses of Samarkand and Bukhara, but with no explicit reference to the historical periods [8]. G.A. Pugachenkova paid a great deal of attention to irrigation in her studies of the architecture during the Timurid dynasty [9, 10]. The history of hydraulic engineering inventions in Central Asia was reviewed by S.M. Mamadzhanova, however, her publications fail to relate to the periods of development of the urban environment [11]. R.K. Mukhiddinova's dissertation was devoted to irrigation of gardens and parks since the 9th century, including in the urban area [12].

The aim of the study is to identify the process of emergence and development of centralized urban irrigation and irrigated urban areas in Central Asia.

The objectives of the study are to identify the historical boundaries of the stages of development of urban irrigation and irrigated urban spaces from ancient times to the present day; to identify the characteristic features and draw up a scheme of the city’s irrigation of for each stage.

Materials and Methods. The object of the study is urban irrigation. The subject is the mutual influence of urban irrigation development and formation of public space in Central Asia.

The description of the stages of development of urban irrigation and irrigated urban spaces was designed using the historical and logical method. In order to identify the boundaries and characteristic features of each stage, the results of archaeological research of the ancient settlements of Central Asia, historical materials (traveler diaries, documents), academic publications by modern and Soviet authors (since historical and archaeological research of some Central Asian regions was mostly ceased in the 1980s) were searched for and analyzed. All references to irrigated urban areas and irrigation facilities that could be related to urban irrigation were collected from these sources. Information on urban planning practices for the corresponding period was searched for in the same manner. Information on irrigation practices was converged with that on urban planning practices.

For each stage, based on the identified characteristic features, the city’s irrigation scheme was designed.

Research Results. Urban irrigation originated simultaneously with Asian cities and responded to each and every change in structure. The stages of formation of urban irrigation are inextricably linked with the evolution of the city itself. The analysis of historical evidence, archaeological reports and related scientific publications allows us to identify six stages in the development of urban irrigation. It is to be noted that the boundaries of each stage are blurred. Thus the scenarios of the 1st and 2nd stages, or the 2nd and 3rd, could exist simultaneously in different places. The chronographic line of urban irrigation development is shown in Fig. 2.

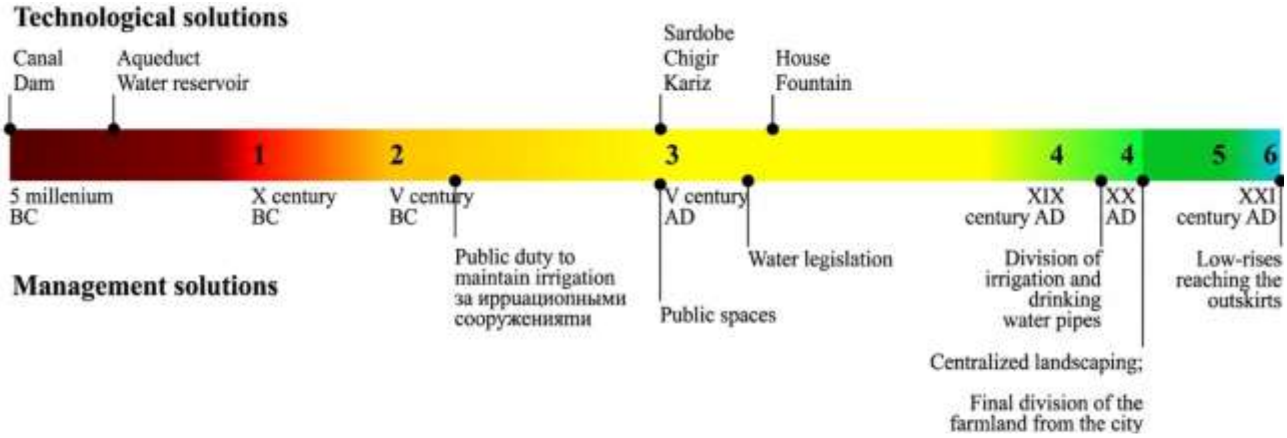

Fig. 2. Chronological line of irrigation development

Stage 1: how the irrigated city originated.

The first stage (late 2nd millenium BC — 1st century BC) is the stage of origination when the "city" were irrigated farmlands located at a short distance.

Artificial irrigation structures (canals and ponds) in Central Asia have been known since the 5th millennium BC. At roughly the same time, the first agricultural oases appeared in the south of Central Asia [3]. In the 3rd millennium BC, farmers living in the territory of modern Tajikistan and northern Afghanistan built dams, water distributors and aqueducts [13].

In late 2nd and early 1st millennium BC, complex irrigation structures and large-scale irrigation systems started appearing in Central Asia where large fortified oasis settlements were emerging (Yaz Tepe, Aravali Tepe, Yelken Tepe, etc. – modern Turkmenistan and northern Afghanistan) [14]. Initially the oasis was a collection of agricultural lands sometimes surrounded by a common fortress wall [2, 15]. The emergence of such settlements on the main irrigation ditches is due both to the convenience of irrigation and the need to protect the key hydraulic structures [15]. Thus, from the onset, the oases were soaked with water. However, in the 11–4 century BC, it was still too early to call them cities in the modern sense [15].

The characteristic features of the period were

– water supply in each area;

– no division of the water supply systems according to the water use methods;

– homogeneity of irrigated elements;

– homogeneity of water use ways;

– invention of the simplest hydraulic structures that would later be used in urban irrigation: dam, canal, aqueduct.

The scheme of the irrigated oasis settlement is shown in Fig. 3.

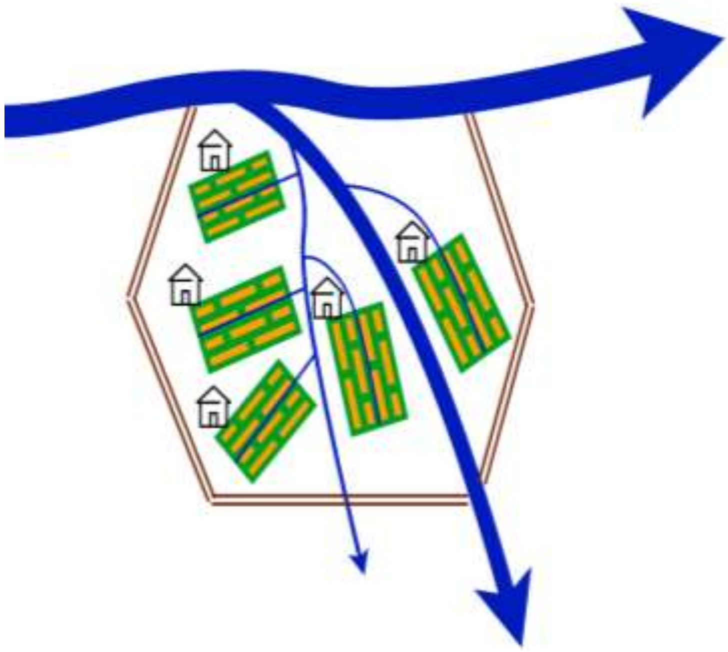

Fig. 3. Scheme of the irrigated strengthened oasis

Stage 2: a city becomes the city.

The second stage is the stage of formation of the urban environment (mid-1st millenium BC – 11th century AD).

In the middle of the 1st millennium BC, Central Asia was swept by a wave of formation of nation states. They brought about a real urban structure to the settlements: streets, crafts, division of labor. From then on, not all residents were engaged in agriculture. Residential areas with no agricultural land appeared in the oases [2].

That was how the oasis city appeared and took a dominant place in Asian urban planning for a period of more than a thousand years until roughly the 8th century [16].

Irrigation technologies in the city and beyond did not differ much at that stage. Surface waters were supplied by canals with intakes upstream of the city, if this is not possible, chigirs were used to raise water to the higher bank [3]. Groundwater is supplied by means of karizes. Various methods of conducting water supply had been mastered and applied: in some cities open-channel irrigation sufficed, in others a clay or even a lead water supply was being laid. Thus, that was when most of the technologies broadly used in urban irrigation these days were invented.

Throughout almost two millennia, there were no fundamental changes in irrigation technologies. The structure of the urban environment was slowly changing. In the 5th–8th centuries AD, a typical Central Asian city consisted of the following basic elements [16]:

1) a citadel, i.e., a fortification inside which palaces were typically located;
2) a shahristan, i.e., an administrative and handicraft part;
3) a rabad, i.e., a commercial and artisanal suburb with farmland.

A shahristan is closest to what is called the city these days [17]. It normally had a regular layout and was built in a planned manner: a grid of blocks was broken up, the main buildings (religious, public ones) were erected, houses were dug out, and then blocks were built up, and administrative offices were moved. There was no constructive division of drinking and irrigation water supply, but the water in Shahristan was mainly used for drinking purposes. Clay culverts or ditches ran from the water intake structures on the rivers, largely in unpolished canals. The network branched off from the main channels or pipes into the residential areas. Archaeological evidence does not show that canals ran along every street, as they do these days, but the main streets of shahristans typically ran along the canals. The houses in Shakhristan were primarily built for drinking water supplies [16].

Here, in the 5th century shakhristan, i.e., in the city of Penjikent, archaeologists witnessed the first landscaping of public space — the remains of vines and stumps along the open-channel irrigation in the territory of a temple [18]. Gardens attached to religious buildings were also found in hillforts of the 6th–10th centuries in the territory of modern Kyrgyzstan, in the Chuy Valley [19].

Lots of cities in the period already had specialized sewage structures. However, e.g., in the cities of Khorezm and in Bukhara, the open-channel irrigation network also became a waste disposal site making the water in the canals not suitable for drinking. Therefore, clean drinking water was sold or provided free of charge in special pavilions [17]. Presumably, the result of the pollution of the open-channel irrigation was the growth of cities upstream of irrigation canals as people sought to settle as upstream as possible. This trend was noted by A.M. Belenitsky and I.B. Bentovich in Termez, Bukhara and Samarkand.

A rabad was a more agricultural area at that stage. Water was mostly used for irrigation. Although the building density of a rabad was significantly lower than that of a shahristan, a canal network of a rabad could be denser due to a greater need for water. Water was also required for artisans, including conversion into mechanical energy: e.g., it set mills in motion.

Characteristic features of the stage were

– specificity of water use depending on the city’s area;

– gradual withdrawal of large farmlands from the urban environment;

– invention of almost all major hydra ulic engineering devices and structures that would be used in urban irrigation before the beginning of the 20th century.

The scheme of the irrigated three-part city is shown in Fig. 4.

Stage 3: public and private.

The third stage (the 7th — early 19th century) started with the flourishing of public space — a park, a public garden.

Since the 8th–9th century, a typical feudal city had been forming in Central Asia. A rabad was evolving as the economic life of the city was concentrated there [6, 20]. A rabad gradually absorbs a shahristan, and agricultural lands form satellite villages, i.e., rustaks [17, 19].

Muslim culture spurred the development of the concept of a public building and space. The territory of a mosque was one of such critical spaces: the peculiarities of Muslim worship services made it the most frequently visited public space. Houses were typically set up in the courtyards of mosques, and the courtyards themselves were well landscaped and irrigated [21].

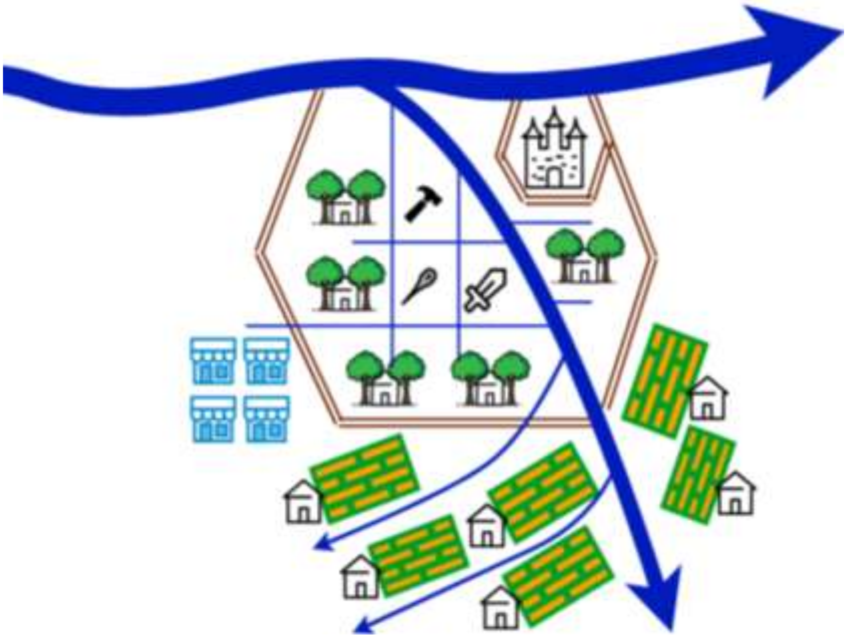

Fig. 4. Scheme of a three-part irrigated city: a citadel, a shahristan, a rabad (based on the materials [16])

In the Golden Horde period, fountains and houses appeared which were reservoirs for accumulation of water. In some cities, drinking water was extracted from underground sources using wells and karizes, while technical (irrigation) water was supplied via open-channel irrigation [22]. Excess water could be collected in reservoirs in the lower part of the city. For sewerage, structures isolated from the water sources were being built.

The Central Asian city witnessed a real boom in the development of landscaping and irrigation during the Timurid era from the 14th to the 16th centuries [7, 9]. Contrary to the legends, the Timurid troops did not destroy all the cities in their path. Lots of captured cities were supposed to serve as evidence of the dynasty's glory, with gardens, open-channel irrigation, public houses and sardobes, i.e., closed large reservoirs for the accumulation of water[1]. Houses were also designed to provide coolness [9]. Their shores were lined with trees [8].

The city of the Timurid era was already completely artisanal and commercial [9]. Even the suburbs performed a recreational rather than an agricultural function [6, 10]. There were lush country gardens for relaxing holidays and walks, with trees chosen rather for decorative purposes; ponds and abundant irrigation were organized [10]. The gardens laid out by Timur and his descendants in Samarkand, Turkestan, and Kabul went down in history[2]; the latter was renovated in 2007. It is worth clarifying that the Timurid country gardens could barely be called public space as the entrance was normally closed to ordinary citizens [3]. The economic function of the gardens has been somewhat retained as along with ornamental crops, fruit crops are also grown, fish are bred in ponds [23]. However, the main agricultural lands got separated from the city.

Technologies for designing parks, squares, irrigation systems, and landscaping methods were transferred from city to city and applied throughout the territory developed by the Timurids. Masters of the art of irrigation, gardening, architecture, and construction were forcibly gathered from all of the conquered territories in order for their knowledge and experience to be exploited [9].

Since the late 17th century, as the Timurid dynasty was facing demise, the time of troubles and Uzbek conquests came. The cities were falling into decline, and the confrontation between the Beks and the khans in matters of water ownership resulted in the disruption of the irrigation systems [7]. Some large cities were completely abandoned, and farmland was abandoned. Even Samarkand, which had no inhabitants for 7 years in the early 18th century [24], failed to escape that fate. There is no need to talk about the development or even maintenance of the irrigation structures during that period as the literature that has survived to these days only mentions the stagnation of those times. It was only by the late 18th

[1] De Clavijo RG *Diary of a Trip to Samarkand to the Court of Timur (1403-1406)*. Moscow: Nauka; 1990. (In Russ.).https://www.klex.ru/z5g (accessed: 19.03.2026)

[2] Babur ZDM *Babur-Nameh. Notes of Babur (1483-1530).* Tashkent: Central Committee of the Communist Party of Uzbekistan; 1958.

century that the relatively strong power of the Uzbek conquerors was established in the Khiva, Bukhara and Kokand khanates [7]. Cities were starting flourishing and reviving.

The characteristic features of the stage are:

– a variety of irrigated elements: their areas, irrigated crops, their irrigation methods;

– almost complete separation of the farmland from the city;

– emerging separation of drinking and irrigation water supply;

– organization of water accumulation;

– recreational function of reservoirs.

The irrigation scheme of the Timurid-era city is shown in Fig. 5.

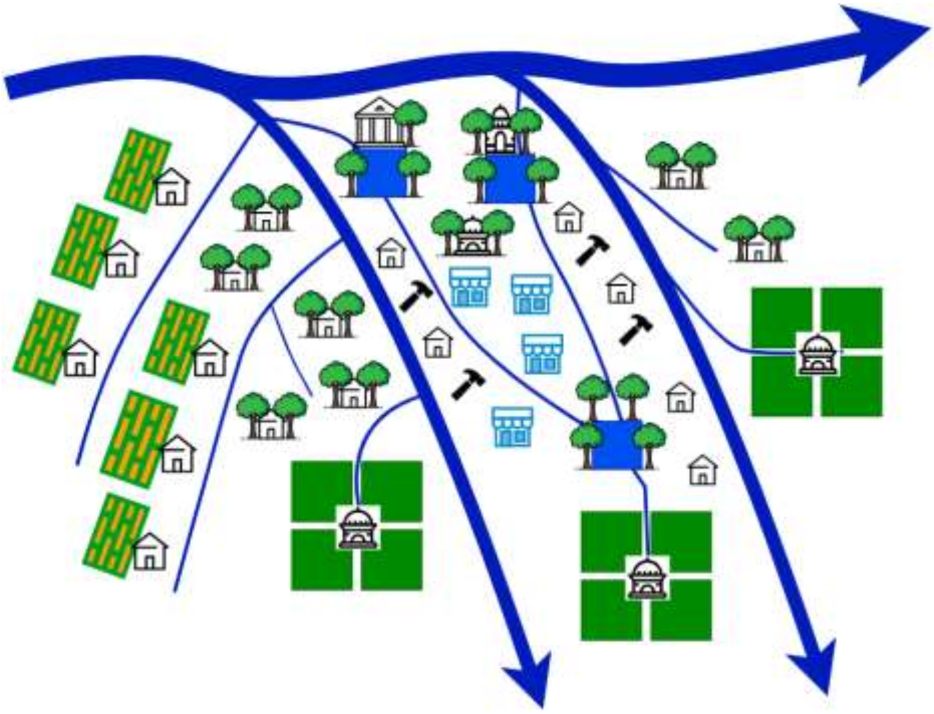

Fig. 5. Irrigation scheme of a Timurid city with houses and a reservoir for excess water

Stage 4: The Russian Empire: a city next to a city.

The 4th stage got underway thanks to Russian engineers. In the 19th century, the Russian Empire was actively developing Turkestan. Russian cities would sprawl in place of rather next to indigenous ones to bear a strong resemblance to Russian county towns [25]. Such a phenomenon could be noted in Tashkent, Samarkand, Namangan, Katta Kurgan [26]. Russian planners brought the European style to the structure of urban landscaping and increased the diversity of landscaped areas with boulevards, squares, alleys [23]. Gardens and parks became public domain [26].

Russian settlers were experimenting with plant species looking for drought-resistant ones; poplar, birch, and lilac were imported from afar. Irrigation technologies were adopted from local residents: irrigation ditches were laid along alleys and boulevards [25], chigirs were used to raise water to a steep bank [27]. Irrigation networks and landscaping were integrated into the city at the stage of the master plan [26].

By adapting technologies, Russian county officials adopted an important trend: getting rid of stagnant water diseases [26]. Lots of cities of Turkestan were plagued by typhus, fever, and parasitic diseases [8], and, according to travelers' accounts, each city boasted a local pathogen. That was why the improvement of networks and the revision of the slopes of open-channel irrigation started, but most importantly, in some cities centralized drinking water supply were being arranged separately from irrigation. According to M.I. Ismailov, that was what exarcebated the sanitary condition of the irrigation water supply with less priority given to cleaning it [7].

Reconciliation of the open-channel irrigation network and the roadway was also a critical step for the Russian administrations of some cities; the problem of building bridges over urban ditches was highlighted by archaeologist A. Mirbabaev [28].

Thus, the stage was marked by:

– the division of the urban environment into a "displaced" and an "indigenous" one;

– an increase in the diversity of types of irrigated territories in the "resettled" part, while the landscape organization of the "indigenous" part remained unchanged;

– the development of new landscaped public spaces on the European model irrigated using Asian technologies due to the aridity of the climate;

– activation of the separation of drinking and irrigation water supply.

The irrigation scheme of the two-part city of Imperial Turkestan is shown in Fig. 6.

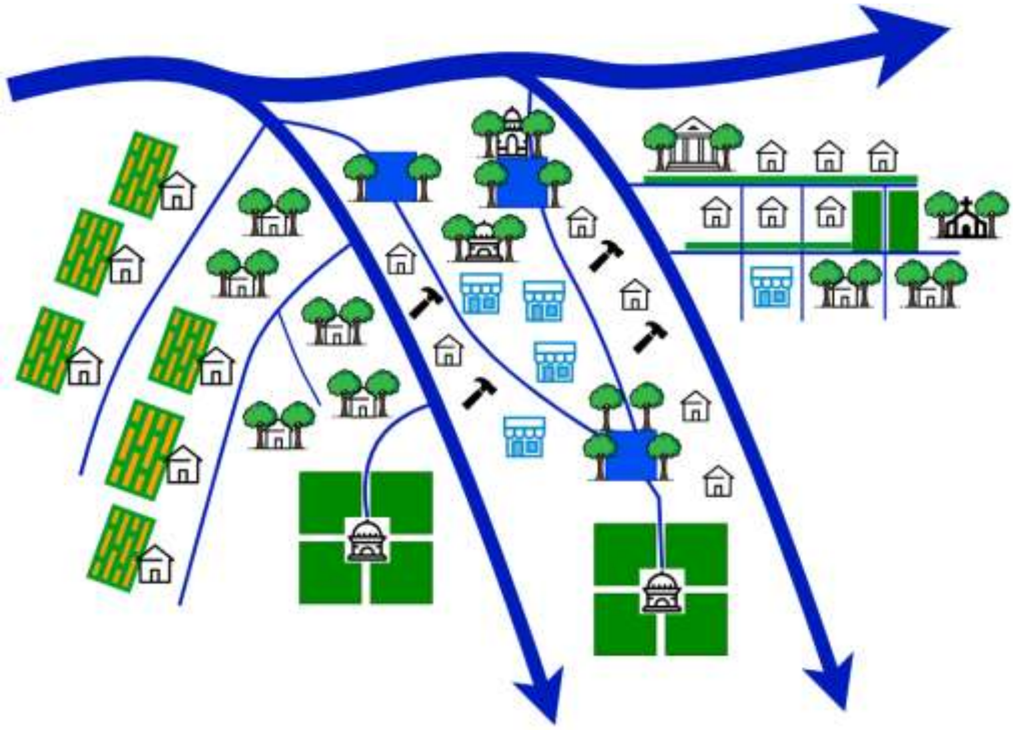

Fig. 6. "The City Next to the City" in the mid-19th century

Stage 5. The Soviet stage: the city was irrigated according to the plan.

The 5th stage saw a combination of Soviet planned urban planning and automation. Soviet urban planners took over the street irrigation and landscaping. A regular grid of streets was being actively used again, which was conducive to the construction of an irrigation network. The ditch network was being laid out along with the new streets to become an essential part of them.

However, Soviet urban planners did not take note of all the traditional elements. Thus, in spite of the effort of historian M.E. Masson to investigate and popularize sardobes [29], they were no longer used. Soon they were completely discarded as an element of engineering infrastructure. Mining operations were no longer relevant as new technologies for ground-water extraction were emerging.

Earlier in the article, the "typical" cities of each era could could only be referred to as such with reservations, since there were lots planning options in parallel, Soviet standardization allows us to talk about a full-fledged typical situation. Within the Soviet part of Central Asia, cities were on purpose being made to become more and more similar to one another.

The Soviet city was absorbing the union of indigeneous and county towns. Living conditions were being made to be more equal. The sanitary struggle was ongoing, and even became radicalized as houses were being demolished as "breeding grounds of infection." Thus, 114 houses were dismantled in Bukhara as part of the fight against parasitic diseases, and only three were restored; a similar situation was in Samarkand [7, 8]. As a result of the liquidation of the reservoirs, the irrigation network that fed from them had also suffered a damage [7].

The separation of irrigation and drinking systems was occurring in more and more settlements, and closed pipelines were being used for drinking water, while the open-channel network for the irrigation was preferred. The division was also occurring at the level of city services: the Gorvodokanal was now engaged in drinking water supply, and the irrigation network was carried out by enterprises tasked with landscaping the city. Drinking and domestic water use received a centralized domestic sewerage system, while the irrigation irrigation network itself remained a sewer for stormwater and other surface wastewater.

In the 1960s residential development got underway. Neighborhoods had become the most urban green areas. A ditch network was being laid along all the passageways. Here, in the neighborhoods, in the 1970s a certain semblance of houses was revived as they were now small pools in courtyards. There was no drinking function in such pools: they were designed for bathing in hot weather and improving the microclimate of courtyard areas [30].

In cities that were in need of accumulating water, pools of daily and decadal regulation started being built. These were large unpolished reservoirs that could also have a recreational function. The pools were included in the irrigation network and regularly cleaned [5].

In the second half of the 20th century (1970 — early 1990s), there was an interest in integrated landscaping and irrigation projects, and in the 1980s, the "urban natural framework" was included in the territorial planning documents [31]. The Central Asian states sought to preserve the idea later: the Bishkek master plan of 2005 inherited the "urban natural framework" from the Frunze master plan of 1970.

In the late 80s, automation of the urban irrigation systems got underway in large cities. Centralized dispatching was assumed to occur. The dampers on the canals were equipped with electric drives. Technical innovations were introduced in Alma Ata and Frunze.

By the late 20th century, cities looked familiar to modern residents: there were open-channel irrigation in every street, drinking water from the central water supply in every apartment, a dedicated underground sewer network, a variety of green spaces and extensive landscaping.

Let us summarize the characteristic features of the fifth stage:

– a normative and constructive isolation of urban irrigation from the other types of water use;

– irrigation network coverage of the entire city area;

– application of dispatching and automation technologies in irrigation management.

– an almost complete loss of sardobes and houses.

The irrigation scheme of the Soviet city is shown in Fig. 7.

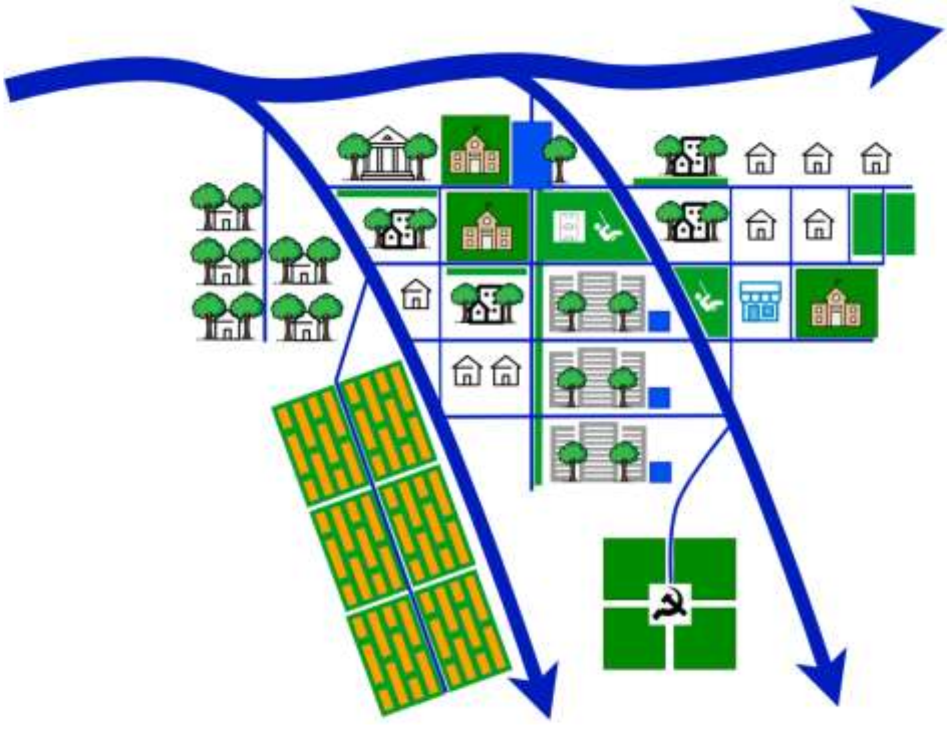

Fig. 7. Orthogonal canal network with pools of daily and decadal regulation and swimming pools

Stage 6. The modern stage.

The latest stage was gaining momentum as the Central Asian countries were gaining independence. It combineds the influence of the modern urban planning trends and the economic realities of the newly formed republics.

In the large cities of Kyrgyzstan and Tajikistan, the process of moving estate development zones beyond the city limits was occurring at an increasing pace under the influence of two factors: demolition for multi-storey buildings within the city and self-occupation of the outskirts due to internal migration [32]. Thus, the modern city was gradually regaining a kind of a "rabad" with a homogeneous type of water consumer — a household plot, a vegetable garden. At the same time, the share of public territories irrigated by the urban centralized irrigation system in the urban area was continuing growing. As most public spaces were accountable to the municipality, a possibility of rationalizing water consumption was on the rise.

Meanwhile, rationalization of water consumption was becoming more relevant with climate change. An increase in the number of heat waves is observed throughout Central Asia these days [33], as well as an intraannual redistribution of precipitation and river flow [34, 35]. These factors are going to inevitably affect the methods of water use [5].

At the same time, modern irrigation methods are barely being used in urban irrigation. Although agricultural irrigation in all Central Asian countries is making use of a wide range of irrigation methods, including sprinkler and drip irrigation, cities are still employing mostly open-channel irrigation. Drip irrigation can only be found at experimental urban sites. In Almaty and Bishkek, centralized sprinkler irrigation is used in some parts of the city, but its share in total irrigation water consumption is small [5].

At the same time, the young republics of Central Asia are increasingly paying attention to their identity and searching for their own cultural path. They are seeking out their own history, develop self-identification and striving to reflect this in their architectural and planning decisions [36, 37]. Therefore the traditional irrigation methods are becoming an element of the urban identity. Irrigation is considered as an element of cultural and historical heritage in Samarkand [38, 39]. In Navoi, sardobes restoration has been set forth as an object of cultural and historical heritage [40], and in Bukhara, canals, reservoirs and sardobes have already been registered and maintained as cultural heritage objects [41]. In Osh, chigir irrigation is being restored to preserve the historical environment of the Sulaiman-Too World Heritage Site [42].

The characteristic features of the current stage are:

– gradual withdrawal of household farms from the urban area and, as a result, the tendency to exclude household irrigation from the urban irrigation system;

– attempts to rationalize water consumption, search for new irrigation methods;

– a trend towards rollback in automation and dispatching technologies in the management of the urban irrigation network;

– search for a new identity in open-channel irrigation.

The latest stage cannot be considered completed — its characteristic features are being shaped, and therefore its current scheme is still close to that of the fifth stage (Fig. 7).

As a result, the historical boundaries of the stages of development of urban irrigation and irrigated urban spaces have been identified since the onset of agricultural settlements in Central Asia. Specific features have been defined for each stage. Irrigation schemes have been drawn up for all the stages except the modern one.

Discussion and Conclusion. Although modern irrigation methods for green spaces in Central Asian cities are very similar to those used two thousand years ago, the scope of their application has undergone a dramatic change. The technologies of grid breeding, water distribution and irrigation have not changed significantly for centuries, but the structure of the urban fabric as well as main users of irrigation have changed. As a result, the basis where urban irrigation developed can be identified:

1) from irrigation for harvesting to irrigation for landscaping purposes;

2) from the multifunctionality of urban water supply systems to the division into separate monofunctional systems;

3) from irrigation for personal purposes to irrigation for public needs.

The latest process is ongoing, so is the evolution of the urban environment.

Following the centuries-old path of development in conjunction with the urban environment, ditch networks are still an integral part of the image of the city and claim the right to be called the cultural and historical heritage of the settled peoples of Central Asia.

However, in order to maintain their functionality taking into account the changing climate, there must be significant adaptation taken to account for the change in river flow and its consequences. Therefore it is recommended that technologies for accumulating and conserving water in open and closed urban reservoirs (houses and sardobes) using modern methods to prevent excessive filtration and development of pathogens are employed again.

The most active development of urban irrigated spaces occurred during the periods of strong intercultural interactions: the Arab conquests (13th century), the Timurid era (14th–16th centuries) and the period of the Russian Empire (the second half of the 19th century), which is indicative of the importance of cultural exchange for the spread of technology. Probably, while developing adaptation measures, it is necessary to refer to the experience of the territories with historically similar climates: the Arabian Peninsula, California, northern India, and the central China. The experience of Central Asia can be used by cities in the regions where the climate is becoming arid. Centralized urban irrigation might become an adaptation measure where it has not been used before.

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LIFE CYCLE MANAGEMENT OF CONSTRUCTION FACILITIES

УПРАВЛЕНИЕ ЖИЗНЕННЫМ ЦИКЛОМ ОБЪЕКТОВ СТРОИТЕЛЬСТВА

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Original Empirical Research

EDN: BOCKDU

Residual Resource of Road Surfacing on High-Traffic Roads

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Abstract

Introduction. An urgent task facing the field of road maintenance is an objective assessment of their residual resource. The existing methods are typically subjective or require that complex procedures are carried out. The aim of the study is to develop a new approach to such an assessment based on instrumental measurements.

Materials and Methods. The object of the study is road surfacing of highways. The method is based on a model that relates the amount of elastic deflection of the coating to the estimated number of loading cycles until the strength has been exhausted. Deflection was measured using a Falling Weight Deflectometer. The technique allows one to adapt the model to a variety of conditions by calibrating the coefficients.

Research Results. Based on the suggested model, a four-level scale of the condition of the road surface has been designed according to the size of the residual resource: a normative, satisfactory, pre-maintenance and critical one. In order to increase the reliability of the estimate, the median value of the resource is used as the calculated value for the measuring point, and its weighted average value is used to characterize the entire site.

Discussion and Conclusion. The developed approach makes it possible to quantify the residual resource based on instrumental data. Implementing this technique would increase the objectivity of diagnostics and assist optimal repair planning. The prospects of the study are related to the further adaptation of the model for a variety of road and weather conditions.

Keywords: elastic deflection, residual resource, FWD, weighted average estimate, non-rigid road surfacing, road surfacing condition category, coating defects

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Оригинальное эмпирическое исследование

Остаточный ресурс дорожных одежд на автомобильных дорогах с высокой интенсивностью движения

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Аннотация

Введение. Актуальной задачей в области содержания автомобильных дорог является объективная оценка их остаточного ресурса. Существующие методы часто носят субъективный характер или требуют сложных процедур. Целью данной работы является разработка нового подхода к такой оценке, основанного на инструментальных измерениях.

Материалы и методы. Объектом исследования выступают дорожные одежды автомобильных дорог. В основе метода лежит модель, связывающая величину упругого прогиба покрытия с расчетным числом циклов нагружения до исчерпания прочности. Прогиб измерялся с помощью установки ударного нагружения (Falling Weight Deflectometer). Методика позволяет адаптировать модель к разным условиям путем калибровки коэффициентов.

Результаты исследования. На основе предложенной модели создана четырехуровневая шкала состояния дорожной одежды по величине остаточного ресурса: нормативное, удовлетворительное, предотказное и критическое. Для повышения надежности оценки в качестве расчетного значения для точки замера используется медианное значение ресурса, а для характеристики всего участка — его средневзвешенная величина.
Обсуждение и заключение. Разработанный подход позволяет давать количественную оценку остаточного ресурса на основе инструментальных данных. Внедрение данной методики повысит объективность диагностики и поможет оптимально планировать ремонтные работы. Перспективы исследования связаны с дальнейшей адаптацией модели для различных дорожно-климатических условий.

Ключевые слова: упругий прогиб, остаточный ресурс, FWD, средневзвешенная оценка, нежесткие дорожные одежды, категория состояния дорожной одежды, дефекты покрытия

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Introduction. Managing road structures is one of the most critical and demanding areas of road management. It requires that clearly defined criteria and rules for assigning the types and timing of repairs, as well as their annual planning are specified. Considering the high economic costs of maintaining road networks in the condition needed, this issue is commonly resolved by means of a preliminary instrumental assessment. However, the approaches to interpreting results of the instrumental assessment in different countries and sources vary broadly depending on the historical experience of designing and constructing road surfaces, as well as the instrumental base conventionally used in particular countries and regions.

All types of management activities in road surfacing management can be geenrally divided into two types: a network and project one. The network layer involves a general assessment of road surfacing, forecasting changes during their service as well as their remaining life, forming maintenance, repairs and overhaul plans, as well as calculating costs during the life cycle under various operating conditions while choosing the most optimal ones. At the design level, the tasks of calculating and constructing road surfacing are solved within the framework of a decision made on major repairs or reconstruction, and economic estimates are provided for a detailed design solution [1, 2].

The main tasks solved at each level of road surfacing management determine the types of models underlying them. Thus, for the network level of state management, largely phenomenological models based on the concepts of reliability theory, probability theory and mathematical statistics are employed. The design level typically involves use of structural models based on the principles of deformable solid mechanics, fracture mechanics, and mesomechanics [3–5].

The assessment of the condition of highways at the network level is based on a large number of different indicators employed in a variety of pavement serviceability ratings (PSR). A detailed description of these indicators is found in [6, 7]. In particular, in [6] all indicators of the condition of road surfacing are divided into operational, structural and combined ones. Performance indicators are focused on the actual longitudinal evenness of the pavement. The most well-known of them worldwide used ones are the IRI (International Roughness Index); the RCI (ride comfort index) indicator; the RN (ride number) indicator; the PSR (pavement serviceability rating) indicator; in some cases an analog of IRI called PI (profile index) is used to assess a roughness index.

The most common structural indicator used at the network level is the structural number calculated based on the modulus of elasticity of the soil and the equivalent single axle load (ESAL). The dependency for identifying the actual structural number has the form:

$$S_{n.eff} = 0.4728(D_0 - D_{1.5Hp})^{-0.4810}{H_p}^{0.7581},$$

where D_0 is the elastic deflection in the center of loading; $D_{1,5H_p}$ is the elastic deflection recorded at a distance equal to 1,5H_p; H_p is the thickness of the road surfacing.

The minimum required total modulus of elasticity identified in the Russian Federation at the design stage can be used as an analogue of the structural number $E_{min}^{тр}$:

$$E_{min}^{тр} = \sqrt{\frac{p}{600}} lg(\sum N_{act}) - 3.55,$$

where p is the design pressure on the pavement surface (800 kPa); $\sum N_{act}$ is the total number of the design loadings.

The combined indicators combine several indices characterizing the roughness index of the pavement and defects on the surfacing. In some cases, indicators characterizing the operational condition and structural indicators are combined. An example is the Pavement Quality Index (PQI). It includes the Structural Adequacy Index (SAI), the Surface Destruction Index (SDI), and the Ride Comfort Index (RCI). The key point in designing a combined index is a structured technique for collecting expert opinions. In fact, this is a calibration with its result should be statistically reliable and free from systematic errors. The dependencies for identifying this indicator might vary in different regions. E.g., the calculation of the PQI indicator using the MnDOT [8, 9] (Minnesota Department of Transportation) method is calculated using the formula:

$$PQI = \sqrt{RQI \cdot SR},$$

where RQI is the ride quality index; SR is the surface rating.

As the index RQI is estimated in the range of 0–5, and SR is in the range of 0–4, the final PQI score can take values in the range of 0–4.5.

In general, it is to be said that all of the above indicators make it possible to effectively characterize the current condition of the road surface, but linking them to the service life is a rather daunting task typically solved subjectively.

The key concept in managing road surfacing at the network level is the concept of the residual resource/service life of road surfacing (in a more modern formulation - the remaining service life). The remaining resource is defined as the number of load application cycles or the number of years until the failure of the pavement according to any criterion occurs. Since 2014 when the document [10] was released, instead of the term residual resource, the concept of residual service life (RSL) has come into practice [11–15] that characterizes the remaining time until a certain type of work has been completed, which can be both preventive maintenance as well as repairs and overhaul of highways.

The main methods and models for RSL forecasting are models based on design practices, or empirical models. As regards models based on design practice, the models provided in the document Mechanical Empirical Pavement Design Guide (MEPDG) are prioritized. These models allow us to identify the residual value of ESAL based on the model of the fatigue failure capacity of asphalt concrete and the model for predicting track accumulation[1] [16–18].

The permissible number of load applications for a specified mode of deformation of asphalt concrete pavement is calculated using the formula:

$$N_{fi}(\varepsilon_i) = C_f K_{f1} {\varepsilon_i}^{K_{f2}} E^{K_{f3}},$$

where $N_{fi}(\varepsilon_i)$ is the number of application cycles of the design load calculated based on the tensile strain at the lower boundary of a set of asphalt concrete layers ε_i; C_f is the calibration factor identified based on the results of field and laboratory measurements; K_{f1} is an empirical parameter in the standard model, K_{f1} = 1.2; K_{f2} is an empirical parameter in the standard model, K_{f2} = –3.291; K_{f3} is an empirical parameter in the standard model, K_{f3} = –0.854.

The model for forecasting the residual life of track accumulation on the surface of asphalt concrete is presented as

$$\frac{\varepsilon_p}{\varepsilon_r} = K_1 10^{-3.4488} T^{1.5606} N^{0.479244},$$

where ε_p is the plastic component of compression deformation in asphalt concrete layers; ε_r is the elastic compression deformation in asphalt concrete layers calculated using the elastic layered half-space model; T is the temperature in degrees Fahrenheit; N is the total load for the period of operation in the ESAL.

The advantage of these models is their good calibration and their capacity to flexibly adjust to the conditions of the design region due to a large number of empirical coefficients. Their disadvantage is that the implementation of the MEPDG methodology requires a large number of source data typically missing during the operation phase of the highway. As a rule, this significantly disrupts accuracy and efficiency of these models.

Empirical approaches to RSL forecasting include design of survival curves, use of the Failure Cox Proportional Hazard method, probabilistic and statistical models such as Bayesian models, Markov chains, as well as linear and nonlinear regression models [19–22]. More modern approaches include use of artificial neural networks and genetic algorithms [23–25].

In general, it is to be noted that in spite of a variety of approaches, forecasting the residual resource is currently a fairly urgent task. On the one hand, forecasting methods should be based on measurable condition parameters found in the highway diagnostic system of each region and country. On the other hand, the model for forecasting the residual resource must be sufficiently versatile and well-calibrated for various conditions. On top of that, it is important to note that as a linearly extended object, a highway is characterized by significant heterogeneity and thereby a continuous change in the values of the residual resource for each characteristic section. In this case, it is of paramount importance to develop an approach to identifying the value of the residual resource that effectively and informatively characterizes the section of the overall highway.

[1] *Mechanistic Empirical Pavement Design Guide*. Washington, DC: AASHTO; 2020. 600 p. URL: https://www.fhwa.dot.gov/pavement/materials/hmec/pubs/module_e/participant_workbook.pdf (accessed: 01.03.2026).

Materials and Methods. It is recommended that the resource of a highway is expressed in the form of the number of applications of the design load over its service life. The estimated resource of a highway is defined as

$$\sum N_{calc} = 0.7 f_{lane} \frac{K_{sum}}{q^{T_l - 1}} T N_{calc} k_n,$$

where f_{lane} is the band ratio that characterizes the percentage of traffic moving in the busiest lane; K_{sum} is the summation coefficient; T_l is the service life; T is the number of design days per yea6/24/2026r; N_{calc} is the traffic intensity in the design axes during the first year of service life; k_n is a coefficient taking account the deviation of the actual flow from the forecast one; q is the coefficient of an increase in the traffic intensity.

$$N_{calc} = \sum N_i k_i,$$

where N_i is the traffic intensity of the *i*-th group of vehicles; k_i is the coefficient of reduction in the vehicle to the design axial load of 115 kN.

Considering that this dependence is based on the idea of increasing traffic intensity according to the law of geometric progression, the number of loading cycles actually experienced by the road structure can be identified for any year of operation of the highway in the form of the value of the actual running time of the road surfacing $\sum N_{act}$. In this case its residual resource can be calculated as $\sum N_{resid}$:

$$\sum N_{resid.} = \sum N_{calc} - \sum N_{act}.$$

The disadvantage of this approach is the lack of a relationship between the actual residual resource expressed as the number of cycles of the calculated load and the actual structural condition of the pavement. This drawback can be bridged by reformulating the dependency in the form of

$$\frac{pD(1-\mu^2)}{l} = \sqrt{\frac{p}{600}}\, 98.65(lg(\textstyle\sum N_{act}) - 3.55),$$

$$l = \frac{pD(1-\mu^2)}{113.91(lg(\sum N_{act}) - 3.55)},$$

$$\sum N_{act} = 10^{\frac{pD(1-\mu^2)}{113.91 l} + 3.55}.$$

Due to the fact that dependence (1) describes the design change in the actual total number of applications of the design load from the initial to some maximum permissible condition of the road structure, a similar change is associated with an increase in the elastic deflection on the surface of the structure. Thus, based on the actual elastic deflection recorded on the surface of the pavement, it is possible to directly identify the value of the actual running time of the pavement, and thereby the residual resource. However, the main issue is that as an extended object a highway cannot be characterized by a single value of elastic deflection, modulus of elasticity, or any other characteristic. A number of the results of elastic deflection recording in public roads belonging to the federal and regional road network are shown below (Fig. 1).

As can be seen in the figures, the elastic deflection is characterized by a sufficiently large spread over the linear extent of the site. E.g., the coefficient of variation is C_v = 0.16 (Fig. 1a), C_v = 0.55 (Fig. 1b), C_v = 0.47 for the third area.

In the framework of the study, it is suggested that a weighted average estimate is used based on the value of the actual elastic deflection recorded on the surface, as an indicator based on which the residual resource of an extended section is identified such as

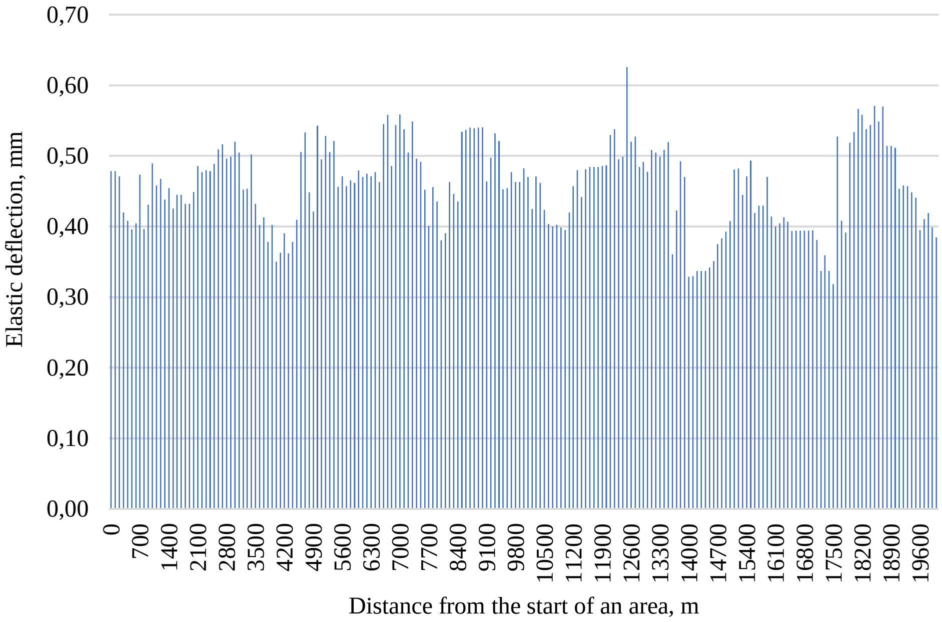
$$\overline{x} = \frac{\sum \omega_i \alpha_i}{\sum \omega_i}.$$

α_i taken as a weighed parameter and the length of the site is assumed to be in a normative, satisfactory, unsatisfactory condition in terms of the residual resource and instead of ω_i the value of the actual operating time of the road structure is taken directly:

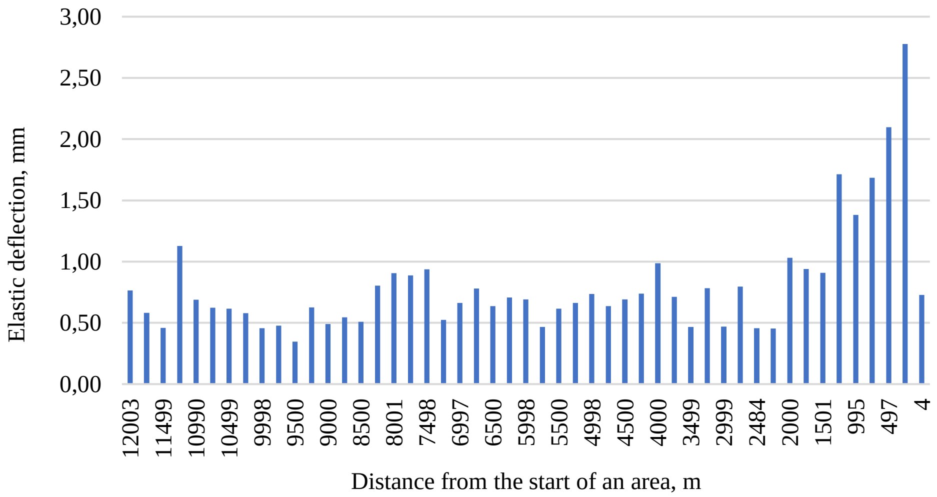
$$\sum N_{act} = \frac{\sum N_{act}^{i} l_i}{\sum l_i}.$$

Thus, the main task for applying this approach is to rank sites on four levels depending on the amount of actual operating time. In order to address this issue, it is recommended that a regression model between the values of the actual operating time and the operational condition of the pavement is designed. This relationship was designed based on tests on 60 sections of the highways (Fig. 2). A FWD Primax 1500 falling load deflectometer was used to record the deflection bowl, and a diagnostic laboratory with a panoramic camera unit was used to record defects on the pavement surfacing.

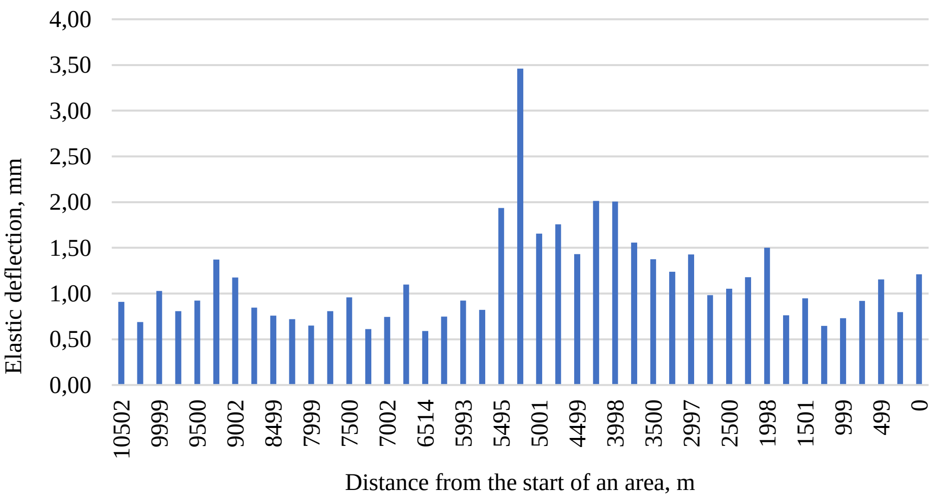
According to the visual assessment, the average score of 1–2.5 corresponds to the presence of defects such as crack grids covering from 30 to 90% of the length of the rolling strip, frequent potholes, single crack grids, a track depth of more than 4 cm. The average score of 2.5–3.5 corresponds to the surfacing defects in the form of longitudinal lateral cracks, frequent transverse cracks, and tracks 2–3 cm deep. Defects rated with an average score of 3.5–4.5 include defects in the form of rare transverse cracks, and defects rated above 4.5 include minor defects in the form of single transverse cracks, a longitudinal central technological crack.



a



b



c

Fig. 1. Actual measurements of elastic deflection on sections of operated highways: *a* is a federal category I highway, *b* is a federal category II highway, and *c* is a regional category IV highway

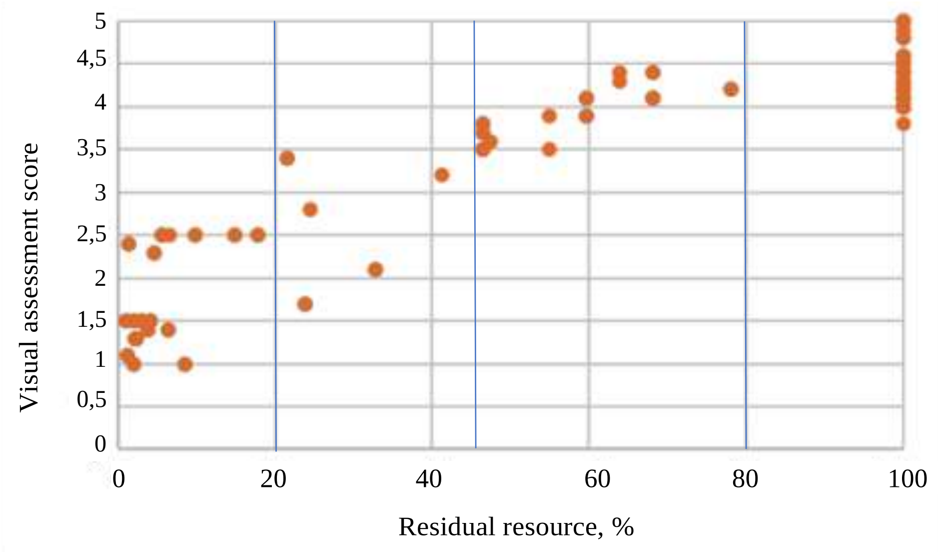


Fig. 2. Dependence of the residual resource on the condition of the surfacing

The final table of the relationship between the value of the residual resource and the corresponding category of condition of the pavement on the site has the following form (Table 1).

Table 1

Classification of the condition of the road surfacing according to the size of the residual resource

Residual resource	Condition category
< 20 %	Critical
20–45 %	Pre-maintenance
45–80 %	Satisfactory
> 80 %	Normative

The median of each range of values of the residual resource is substituted directly into the formula for calculating the weighted average estimate as a value that is not subject to outliers and extreme values.

Depending on the weighted average assessment of the condition of the site, a decision is made on control actions to extend the service life or ensure a standard maintenance service life. Such measures might include road maintenance with a choice of the type of wear layers and protective layers applied to the pavement; repairs with local reinforcement of the base layers or only with replacement of worn coating layers; major repairs involving complete or partial disassembly of the pavement structure.

Research Results. The approach set forth in the research to the operational assessment of the residual life of road pavements based on instrumental measurements of elastic deflection on their surface has been implemented on a number of sections of highways with high traffic intensity. In all of these areas, the values of elastic deflection were characterized by great heterogeneity, which made it challenging to select and assign management decisions on the timing and types of repairs. The results of instrumental studies are indicated below, and the weighted average values of the remaining resource for the surveyed sections of federal highways with high traffic intensity are calculated (Table 2).

Thus, the condition of the surveyed sites can be classified as:

– km 492 + 000–517 + 000 in the straight and reverse direction — a normative one;

– km 877 + 000–892 + 000 in the straight and reverse direction — a satisfactory one;

– km 892 + 000–907 + 000 in the straight and reverse direction — a satisfactory one;

– km 354 + 000–365 + 000 in the straight direction — a satisfactory one, in the reverse direction — a pre-maintenance one;

– km 548 + 000–556 + 000 in the straight and reverse direction — a satisfactory one

– km 1138 + 000–1146 + 000 in the straight and reverse direction — a critical one.

Based on the results, recommendations were formulated to restore the condition in the surveyed areas. Thus, in the areas where the regulatory status has been fixed, the main type of control actions on road pavements is installation and subsequent renewal of wear layers made of crushed stone-mastic asphalt concrete with a frequency of 4–5 years. In the areas in a satisfactory condition, this measure can also be implemented, but taking into account the reduced renewal time of the wear layers, i.e., in the range of 2–3 years. In the areas in a pre-maintenance condition, repairs are underway on the surfacing with local reinforcement of the base layers. Major repairs of the road structure are required in the areas in a critical condition. The specific volumes and forks of work to restore the condition of the pavement to the required operational level are identified during engineering surveys at the design level, involving identifying the modulus of elasticity of the structural layers of the pavement as well as of the most weakened elements and corresponding calculations of the strength of the road structures.

Table 2

Assessment of the residual resource on a section of highway on various sections of the M4 DON highway

Area	Residual resource based on the actual condition of a road structure							
	km 492 + 000–517 + 000							
Direction	Straight				Reverse			
Residual resource	< 20 %	20–45 %	45–80 %	> 80 %	< 20 %	20–45 %	45–80 %	> 80 %
% of the length of the section corresponding to the values of the residual resource	0	2	5	93	2	0	0	98
Median	—	26	67	88	10	—	—	90
Total	85.71				88.4			
	km 877 + 000–892 + 000							
% of the length of the section corresponding to the values of the residual resource	16	16	6	61	6	10	0	84
Median	11	31	59	83	15	25	—	83
Total residual resource	63.05				73.12			
	km 892 + 000–907 + 000							
% of the length of the section corresponding to the values of the residual resource	6	10	0	84	13	16	6	65
Median	17	24	—	91	7	24	55	83
Total residual resource	79.86				62.00			
	km 354 + 000–365 + 000							
% of the length of the section corresponding to the values of the residual resource	8	18	2	71	42	28	10	20
Median	15	26	63	85	7	20	60	81
Total residual resource	68.17				30.74			
	km 548 + 000–556 + 000							
% of the length of the section corresponding to the values of the residual resource	0	23	4	71	22	18	2	58
Median	—	33	72	84	9	31	53	83
Total residual resource	71.54				56.76			
	km 1138 + 000–1146 + 000							
% of the length of the section corresponding to the values of the residual resource	46	54	0	0	55	37	8	0
Median	4	24	—	—	6	22	47	—
Total residual resource	14.80				15.20			

Discussion and Conclusion. Based on the results, the suggested approach is appropriate for large organizations that are tasked with managing the condition of the road network assigned. The current system for assigning repairs based on strength, evenness and adhesion coefficients is excessively rigid, as it calls for major repairs to be performed when the condition is met:

$$K_{str} = l_{act} / l_{req} < 1.$$

where K_{str} is the strength coefficient of the road surfacing; l_{act} is the actual elastic deformation recorded under shock loading; l_{req} is the required elastic deformation for a corresponding road category.

Residual life forecasting systems conventionally used in the software of FWD shock loading setups such as PRIMAX, ELMOD, etc., can be effectively used at the design level while selecting the road surfacing design. However, in this case the system of collecting pavement structures should be focused on calculating and forecasting fatigue damage in asphalt concrete layers and plastic deformations along the rolling strips. In all other cases, the direct application of these approaches also requires adaptation to the conditions of each state or region.

It is to be noted that this approach is relatively effective in the case of adoption and adjustment of planned decisions on the types and timing of repairs. Hence, e.g., in the cases of the section "km 354 + 000-365 + 000" instead of the decision initially considered by the owner of this road to carry out mandatory major repairs in both directions, it was decided to restore the layers of wear in the straight direction and repair the reverse direction of the highway. Also, unlike the method of forecasting the residual service life based on survival curves, this approach requires less statistical data and model refinement based on the results of a set of statistics.

The study presents a new concept for estimating the residual life of non-rigid road surfacing based on data from field measurements of elastic deflection in the center of the deflection bowl by the FWD shock loading unit. The advantage of this model is its versatility and capacity to adapt to any diagnostic tool base involving use of FWD shock loading units.

A scale of road surfacing conditions has been developed depending on the actual residual resource identified based on field measurements. In this scale, for the first time, the concept of a pre-maintenance condition for road surfacing has been introduced, which corresponds to the values of a residual resource of 20–45%. Depending on the condition, recommendations are formulated on the types of control actions that is capable of ensuring the safety of a road structure.

The defects observed on the surface of the surfacing of non-rigid pavement and the values of the residual resource identified by means of instrumental measurements have been found to correspond. It has been found that the critical defects on the surfacing correspond to the values of the residual resource of 0–20%.

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LIFE CYCLE MANAGEMENT OF CONSTRUCTION FACILITIES

УПРАВЛЕНИЕ ЖИЗНЕННЫМ ЦИКЛОМ ОБЪЕКТОВ СТРОИТЕЛЬСТВА

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Original Empirical Research

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Digital Approach to Lifecycle Management of a Low-Rise Capital Construction Facility with Heat and Air Exchange

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Abstract

Introduction. The relevance of using applying air-source heat pumps (ASHPs) in Russia's moderately cold climate conditions is restrained by a sharp decline in their efficiency at low outdoor air temperatures. The aim of this study is to present a digital approach to lifecycle management of low-rise capital construction facilities based on a combined heat pump system with optimized heat and air exchange.

Materials and Methods. A technical solution is set forth incorporating a patented mixing chamber installed in the boiler room to supply an air mixture with a calculated temperature gradient to the ASHP evaporator. The system is integrated with a cross-stream supply and exhaust ventilation (efficiency of 40–60%) and controlled by a digital module based on a microprocessor and PWM regulator. Mathematical modeling of streams was performed using the Bernoulli and continuity equations. For monitoring and automatic control, a set of temperature and pressure sensors was employed to ensure adaptive operation of the compressor, fans, and backup electric boiler.

Research Results. Experimental data have confirmed that the joint operation of the ASHPs with a heat recuperator and mixing chamber allows maintaining a high coefficient of performance (COP) of the system. It was found that the threshold of economic feasibility remains at an outdoor air temperature of down to –15°C and heat carrier temperatures of +30…+45 °C. The developed digital control algorithm optimizes the ratio of outdoor to recirculating air minimizing heat losses and electrical loads.

Discussion and Conclusion. Implementation of the suggested digital modular and functional control scheme ensures rational heat and air exchange, reduces the size and cost of engineering utilities, and enhances the energy efficiency of low-rise buildings. The developed solution can be scaled for a broad range of climatic conditions nationwide contributing to resource conservation and extension of the lifecycle of capital construction facilities.

Keywords: lifecycle management; capital construction facilities; heat pump systems; digital twin; simulation modeling

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Оригинальное эмпирическое исследование

Цифровой подход к управлению жизненным циклом малоэтажного объекта капитального строительства с тепло- и воздухообменом

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Аннотация

Введение. Актуальность применения воздушных тепловых насосов (ВТН) в условиях умеренно холодного климата России ограничена резким снижением их эффективности при низких температурах наружного воздуха. Цель статьи — представить цифровой подход к управлению жизненным циклом малоэтажных объектов капитального строительства на основе комбинированной теплонасосной системы с оптимизированным тепловоздухообменом.

Материалы и методы. Предложено техническое решение, включающее запатентованную камеру смешения, устанавливаемую в помещении котельной, которая обеспечивает подачу на испаритель ВТН воздушной смеси с расчетным температурным градиентом. Система интегрирована с приточно-вытяжной вентиляцией перекрестного типа (КПД 40–60 %) и управляется цифровым модулем на базе микропроцессора и ШИМ-регулятора. Математическое моделирование потоков выполнено с применением уравнений Бернулли и неразрывности. Для мониторинга и автоматического регулирования использован комплекс датчиков температуры и давления, обеспечивающий адаптивную работу компрессора, вентиляторов и резервного электрокотла.

Результаты исследования. Экспериментальные данные подтвердили, что совместная работа ВТН с рекуператором и смесительной камерой позволяет поддерживать высокий коэффициент эффективности (COP) системы. Установлено, что порог экономической целесообразности сохраняется при температуре наружного воздуха до –15 °C и температуре теплоносителя +30…+45 °C. Разработанный алгоритм цифрового управления оптимизирует соотношение уличного и рециркуляционного воздуха, минимизируя теплопотери и электрические нагрузки.

Обсуждение и заключение. Внедрение предложенной цифровой модульно-функциональной схемы управления обеспечивает рациональный тепловоздухообмен, снижает габариты и стоимость инженерных коммуникаций, а также повышает энергоэффективность малоэтажных зданий. Разработанное решение может быть масштабировано для широкого диапазона климатических условий РФ, способствуя ресурсосбережению и продлению жизненного цикла объектов капитального строительства.

Ключевые слова: управление жизненным циклом; объекты капитального строительства; теплонасосные системы; цифровой двойник; имитационное моделирование

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Introduction. Over the recent years, the proportion of demand in the market of air-source heat pumps (ASHPs) has risen tenfold in European and Scandinavian countries that can be called moderately cold. Russia is at the top of the official ranking of the relatively cold countries. Thus possible and effective use of air-source heat pumps of most of the country's climatic conditions remains relevant [1].

It is to be noted that well-known scientists from Russia [2–7] and the West [8–14] have dealt with the issues pertaining to the life cycle of capital construction facilities.

In the modern conditions, continuing on the tradition of resource and energy conservation, development and practical solutions for using (ASHPs) based on digital computer control are becoming increasingly important.

Materials and Methods. A distinct feature of application of ASHPs is that ambient air is used as a source of low-potential heat. If installed outdoors, it might be not effective, particularly at low outdoor temperatures. The authors set forth using a mixing chamber device built into a boiler room [15] to supply the pressure of the air mixture to a heat exchanger evaporator with a calculated temperature gradient (Fig. 1).

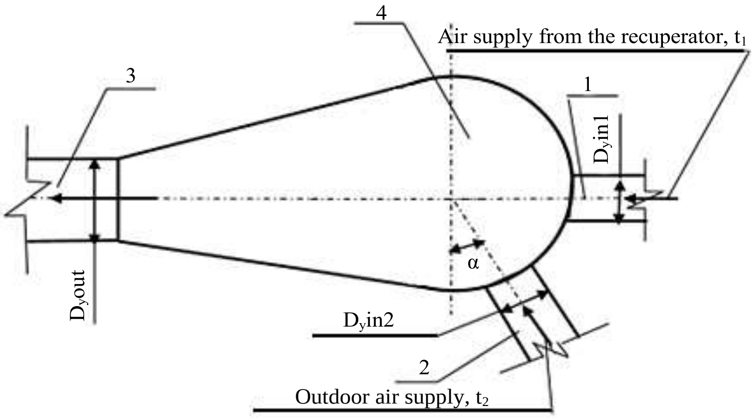

Fig. 1. Device for mixing gas streams

It is to be noted that the operation of the ASHPs mixing chamber is an essential link for recycled air mixing in the evaporator.

The air-stream mixing device itself contains an inlet pipe of an air duct, a pipe for supplying additional stream, and a section of a mixed stream. The nozzle for supplying additional stream 2 is located at an angle within 0 ° ≤ α ≤ 90 ° to the vertical (Fig. 1).

The inlet pipe of the duct is connected to a chamber made in the form of a capsule shaped like a hemisphere connected to a large base of a truncated cone. The diameters of the inlet pipe of the duct and the pipe for supplying additional air stream are equal, with each referring to the diameter of the outlet pipe of the duct directed to the evaporator of the ASHPs ($d_{inlet1} = d_{вх2} = 0.7d_{outlet}$).

This technology of air mixing organization enables an increase in the intensity of equilibrium mixing thus improving the heat exchange conditions. The greatest mixing intensity of the mixed stream occurs once it enters the zone of the truncated cone and chamber, and once it enters the cylindrical part of the outlet pipe of the duct 3, the homogenized stream transitions into a laminar stream. The velocity of the homogenized stream before it enters the cylindrical part of the outlet pipe of the duct 3 increases to the initial velocity of the main stream of the duct 1, equal to the stream velocity before it enters the hemisphere 4 according to the Bernoulli equation:

$$\frac{pv^2}{2} + pgh + p = const,$$

where $\frac{pv^2}{2}$ is the dynamic pressure; pgh is the hydrostatic pressure; p is the static pressure.

Given the cross section of the inlet pipe of the duct 1, the branch pipe for supplying additional stream 2 and the outlet pipe of the flue 3 using the Bernoulli equation, let us write:

$$\left(\frac{pv_{inlet1}^2}{2} + p_{inlet1}gh_{inlet1} + p_{inlet1}\right) + \left(\frac{pv_{inlet2}^2}{2} + p_{inlet2}gh_{inlet2} + p_{inlet2}\right) = \frac{pv_{outlet}^2}{2} + p_{outlet}gh_{outlet} + p_{outlet}$$

As the pressure $p_{вх1}$, $p_{вх2}$and $p_{вых}$ at the cross-sectional levels of the inlet pipe of the duct 1, the branch pipe for supplying additional stream 2 and the outlet pipe of the duct 3 are equal to the atmospheric one, i.e., $p_{вх1} = p_{вх2} = p_{вых}$, the equation takes the form:

$$\left(\frac{pv_{inlet1}^2}{2} + p_{inlet1}gh_{inlet1}\right) + \left(\frac{pv_{inlet2}^2}{2} + p_{inlet2}gh_{inlet2}\right) = \frac{pv_{outlet}^2}{2} + p_{outlet}gh_{outlet}$$

Based on the continuity equation:

$$\frac{v_{inlet}}{v_{outlet}} = \frac{s_{inlet1} + s_{inlet2}}{s_{outlet}},$$

where $s_{inlet1}, s_{inlet2}, s_{outlet}$ are the areas of the transverse sections and $v_{inlet} = v_{outlet}$.

Then $s_{inlet1} + s_{inlet2} = s_{outlet}$. Expressing the parameters $\frac{\pi d_{inlet1}^2 + \pi d_{inlet2}^2}{4} = \frac{\pi d_{outlet}^2}{4}$, we get:

$$d_{inlet1}^2 + d_{inlet1}^2 = d_{outlet}^2$$

According to the specified conditions, d_{inlet1} and d_{inlet2} are equal, as an example let us denote: $d_{inlet1} = d_{inlet2} = 0{,}2m$.

Then:

$$d_{outlet} = \sqrt{d_{inlet1}^2 + d_{inlet2}^2} = \sqrt{0.2^2 + 0.2^2} = 0.28m.$$

Then we get the ratio $\frac{d_{inlet}}{d_{outlet}} = \frac{0.2}{0.28} = 0.7.$

Then

$$d_{inlet1} = d_{inlet2} = 0.7d_{outlet}.$$

The suggested tool enables an increase in the equilibrium mixing intensity improving the heat exchange conditions.

In the studies of the air exchange process, the authors have developed a combined heat pump heat supply system including a mixing chamber, an air exchange circuit with a recuperation element and an additional tubular heating exchanger (THE) controlled by a pulse width modulation regulator (PWM regulator) (Fig. 2).

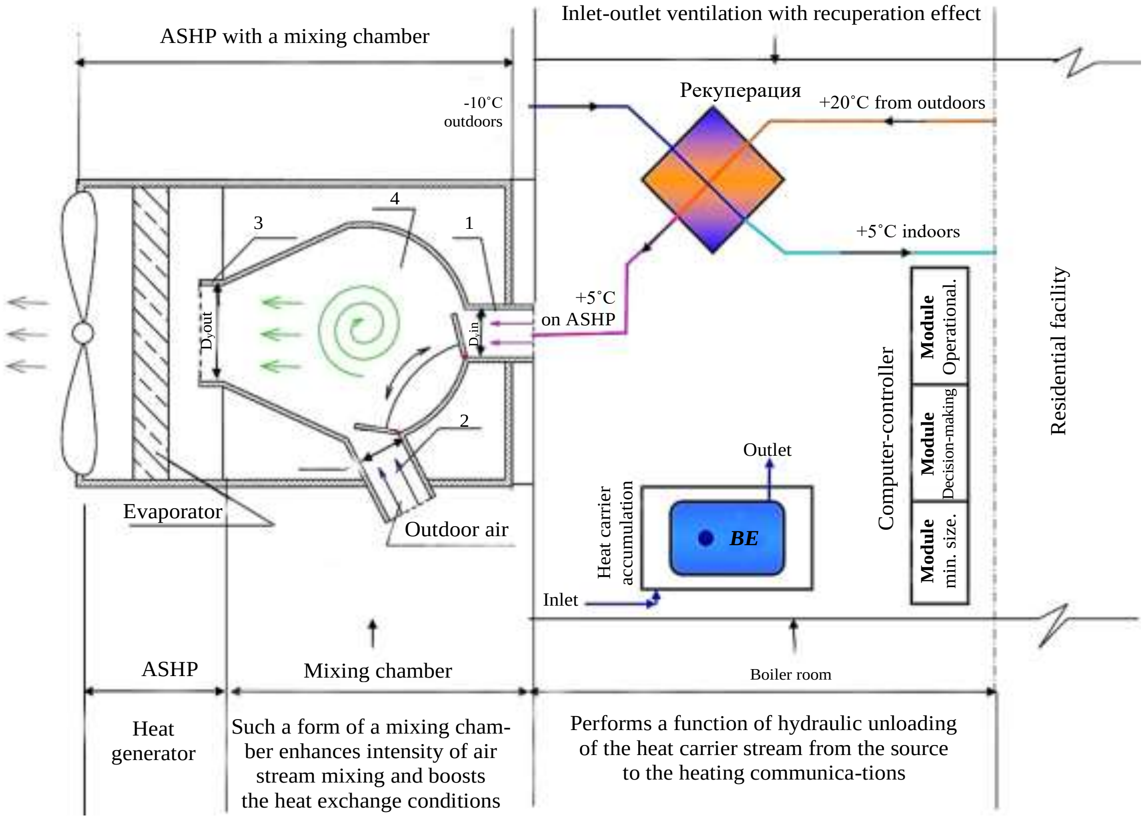

Fig. 2. Combined heat pump heat supply system with mixing chamber, air exchange circuit with recuperation elements

In this case, the ASHP operating mode in a moderately cold climate tends to consistently retain a sufficiently high coefficient of performance (COP) receiving a signal from the digital heat exchange control model of the heat supply system.

Given the specifics of the ASHP air exchange mode with a recovery element, the functionality of the heat exchange system operating through the boiler room and the air ducts of the facility is identified. The existing traditional natural exhaust ventilation found in private homes, apartments, and modern buildings these days is not effective, as while installing sealed plastic double-glazed windows and using highly insulating materials, the air supply is considerably reduced, resulting in an inefficient exhaust system.

Research Results. The central idea of the study is to propose that supply and exhaust ventilation system should be used that employs the principle of heat recuperation, i.e., a process where some of the heat is returned from the exhaust air.

Leaving the facility, the warm air partially heats the counter-cold air stream in the heat exchange system and, according to the configuration, becomes an exhaust at the outlet through the duct guides into the mixing chamber to the evaporator and then through the exhaust air outlet outdoors, and partially heated recuperated air enters the facility.

In such conditions, based on the main goal of creating energy-efficient heating, an air exchange system is obtained that is economically advantageous for the ASHP heat generator.

I.e., as an example, selecting a heat exchanger with an efficiency of η = 50%, the temperature of the mixed air stream supplied to the evaporator is obtained (Fig. 3).

For practical calculation of the effective temperature of the supplied mixed air to the heat pump evaporator, an engineering calculation method based on the influence of the percentage of temperatures according to the "outdoor-indoor" formula is employed. Let us make use of the solution for a special case from the heat balance conservation equation to identify the temperature relative to the warm air stream removed from the facility and supplied to the ASHP evaporator (Fig. 3).

Entering information to the mixing chamber operation control module, the microprocessor programmatically provides the appropriate current solution.

$$\frac{\eta \cdot t_1 + \eta \cdot t_2}{100} = \frac{50 \cdot 24°\text{C} + 50 \cdot (-10°\text{C})}{100} = \frac{(1200 - 500)°\text{C}}{100} = 7\ °\text{C}$$

where t_1 = 24 °C is the temperature in the facility, t_2 = –10 °C is the outdoor temperature.

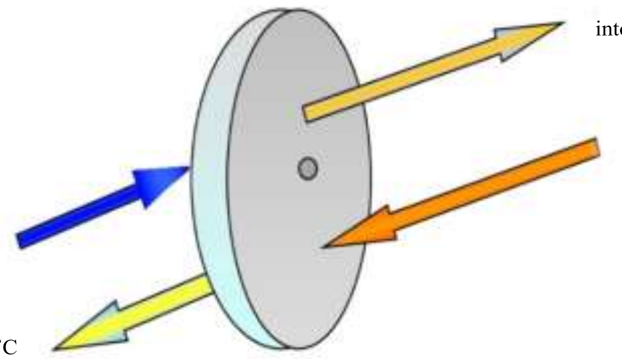

Fig. 3. Change in the temperature of air streams in air ducts with recuperation

In Finland, e.g., prior to commissioning an air duct with recuperation following testing, an annual regeneration coefficient is employed that is retained for Helsinki and Lapland.

When ASHPs and supply and exhaust ventilation work jointly with integrated heat recovery elements, the technical parameters of the system must comply with the following:

1. The diameter of the connected duct used for the above conditions is 125 mm.
2. The number of connected air ducts with ASHP is 4.
3. The heat recuperation efficiency (η) ranges from 40 % to 60 %.
4. Cross type recuperation.
5. The recuperation plate material is polymerized cellulose.

Structurally, the recuperator is located in a confined facility of a thermal unit (boiler room) with air ducts connected on one side to the exhaust air supply to the evaporator, on the other, it provides a facility with slightly heated fresh air through the air ducts (Fig. 4).

The technology of employing such a configuration of air ducts with recuperation allows for an effective increase in the heating capacity of high-pressure vessels in the heating mode of facilities with a moderately cold climate.

According to the tests, the graph (Fig. 5) shows how much more efficiently a recirculating heat pump with a recuperation element and an integrated mixing chamber operates. The "threshold" of savings (losses) is not attained until the outdoor air temperature reaches –15° C at a temperature of the coolant (water) from +30° C to +45° C.

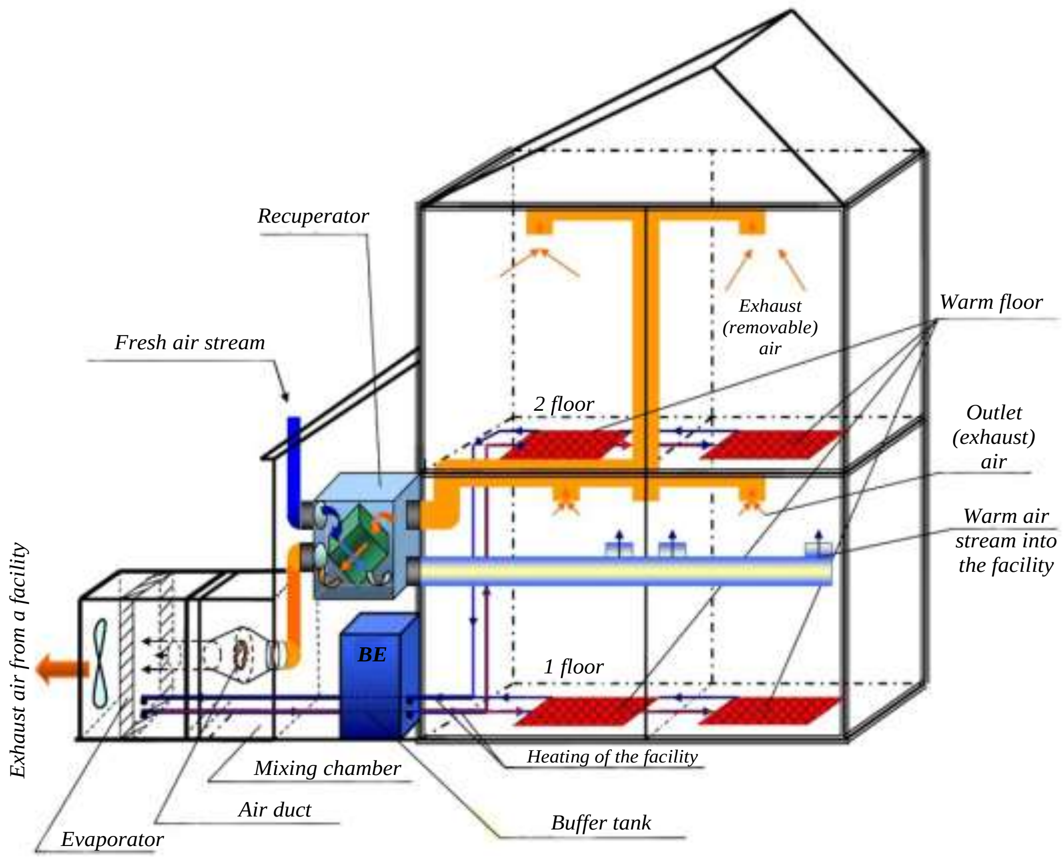

Fig. 4. Effective air exchange system of ASHP

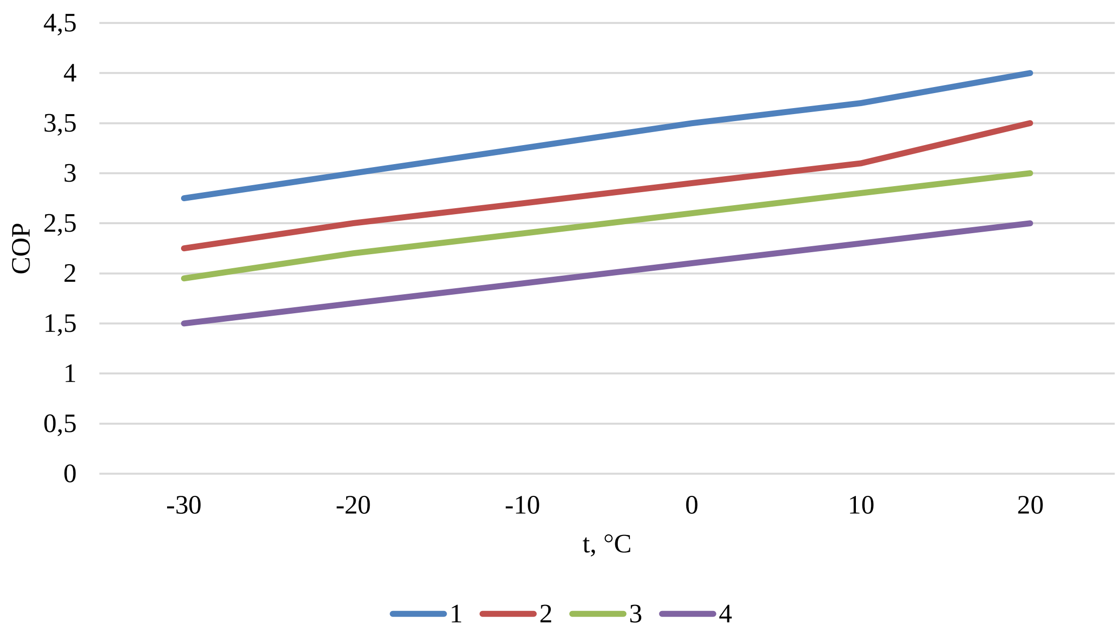

Fig. 5. Graph of the result of the operation of the heat exchange heat pump system of the experimental facility at the outdoor temperatures: 1 — 30 °C; 2 —35 °C; 3 — 40 °C; 4 — 45 °C

The buffer storage tank in such a combined heating system performs the function of hydraulic decoupling of the volumetric streams of the heat source and heating, equalizes the moment of the electric energy on/off switch, compensating for the heating capacity of the heat generator and heat consumer, and at the same time partially covers the tariff periods of electricity.

In the actual experimental conditions (Fig. 6), long-term extensive studies were performed on digital processing of this technological process. The resulting data made it possible to develop an experimental circuit solution (algorithm) for a broad range of various climatic conditions.

Fig. 6. Air mixing chamber [15] and an ASHP installed in a low-rise residential building in the Ivanovo region

It is suggested that a digital technological model is developed for controlling heat exchange through an air heat pump for low-rise and cottage buildings (Fig. 7). This scheme is detailed in Table 1.

Table 1

Regulator sensor description

Sensor 1 — D/t_1	Installed in a buffer tank. The signal is sent to the a pulse width modulation regulator (PWM) when the boundary values of the set temperature and the output heat output are changed. ASHP is on or off.
Sensor 2 — D/t_2	Installed on the evaporator. The signal is sent to the PWM when the set parameter values change. The fan blowing the evaporator changes the rotation speed according to the parameters.
P_1, P_2, P_3	Compressor protection sensors. In normal operation, the contacts are closed. When the set parameters are exceeded or lowered, the sensor contacts open, which causes the compressor to stop.
Sensor 2 — D/t_3	Installed outdoors and gives a temperature status signal to the PWM controller module of the electric boiler. The PWM controller module has its own program — setting the electric boiler control.
Sensor 2 — D/t_4	Installed in a volumetric mixing chamber and sends a signal to the mixing chamber operation control module. A software setting that is wired for 10°C by reacting to temperature changes inside the mixing chamber. The aim of the damper mechanism is to respond to changes in the proportion of the incoming air from outdoors and from the facility, ensuring that the desired temperature is obtained on the evaporator.

The power modules provide the necessary electrical power to loads. Loads include: PWS compressor, evaporator fan, defrost solenoid, circulation pump, electric boiler heating elements, damper drive mechanism. The module of power (MP) ensures the operation of the entire circuit.

Digital technological model of regulating heat exchange through the air-source heat pump in a low-rise building

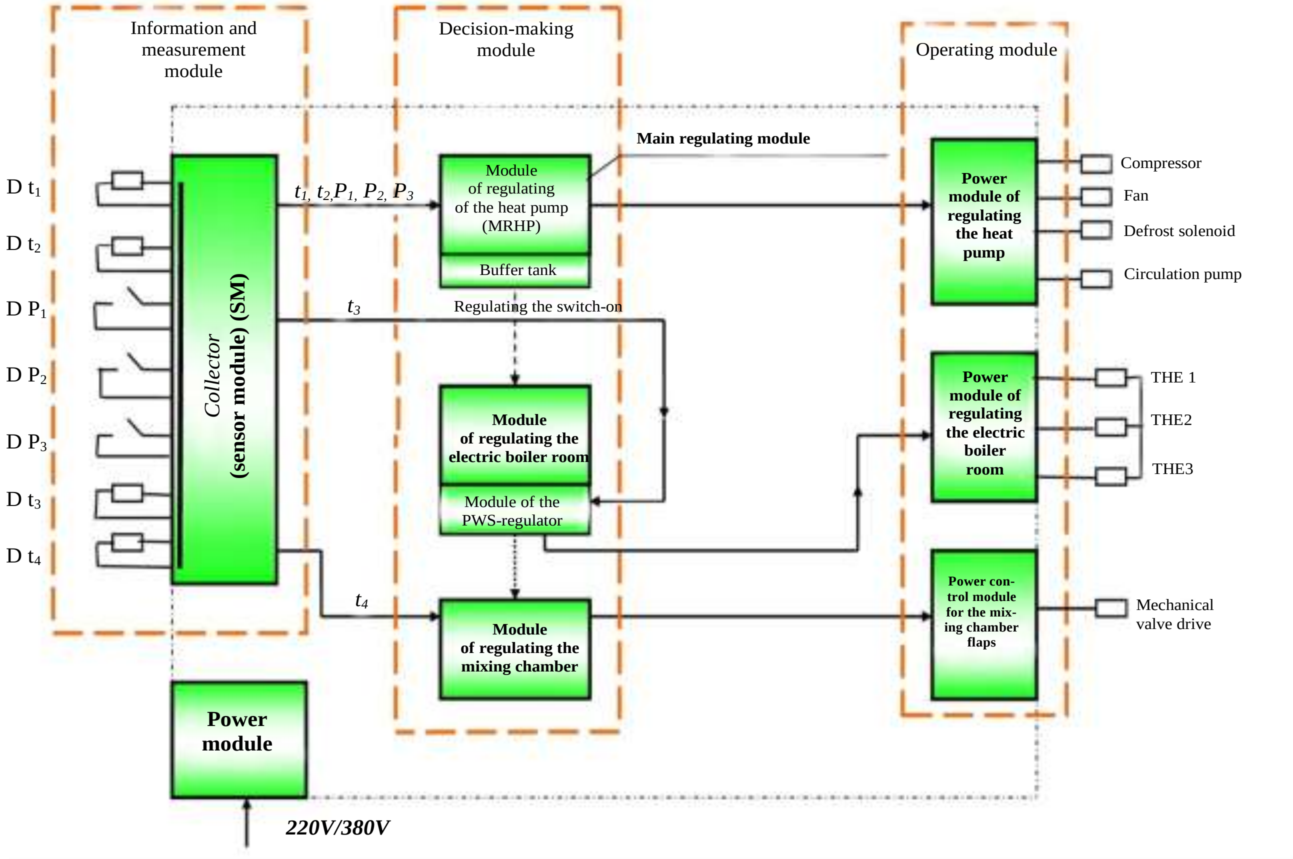

Fig. 7. Digital combined ASHP regulator module (computer controller)

Discussion and Conclusion. The use of information and digital solutions opens up a broad range of functionality for technological control and regulation of the temperature and humidity state in a facility, outdoor temperature gradient along with the collector module of the Dt_1 — Dt_4 sensors. The digital module for recognizing the parameters of this technological process enables ASHPs to regulate an alternating stream of warm and cold air, implementing this process through a computer controller on the operating mechanisms of the an exchange system. Such digitalization allows for a reduction in excessive heat and electrical load, as well as minimization of the volume, area of the boiler room and air exchange communications. As a result of using these solutions, costs and sizes of facilities are cut down.

Having set forth a digital modular functional structural scheme of heat generation from the environment, i.e., relying on the energy efficiency of a circuit solution with an electric boiler, buffer tank and a patented mixing chamber [15], we have attained a rationally controlled heat and air exchange of a combined heat pump system.

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SV Fedosov: scientific supervision, formation of the basic concept, aims of the study.

VN Fedoseev: formation of the basic concept, revision of the manuscript, correction of the conclusions.

VA Voronov: carrying out the calculations, analysis of the research results, preparation of the manuscript, formation of the conclusions.

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All authors have read and approved the final version of manuscript.

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С.В. Федосов: научное руководство, формирование основной концепции, цели и задачи исследования.

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