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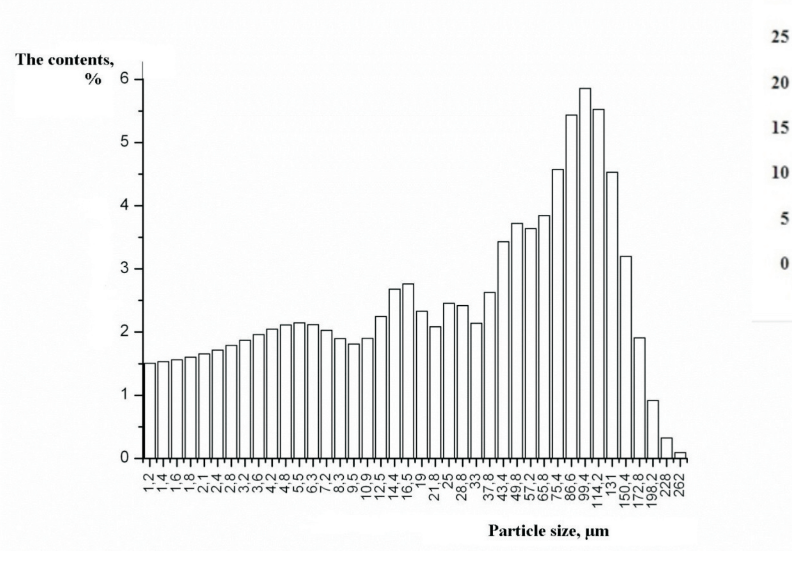

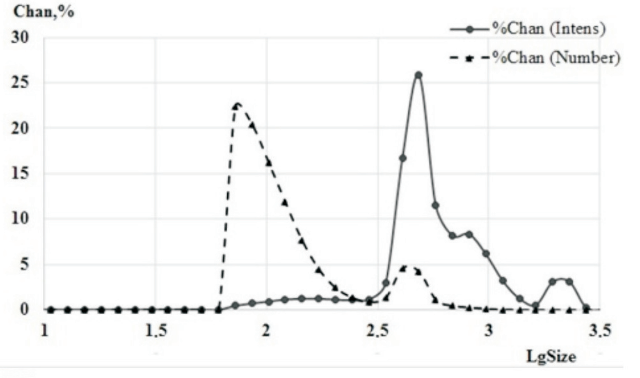

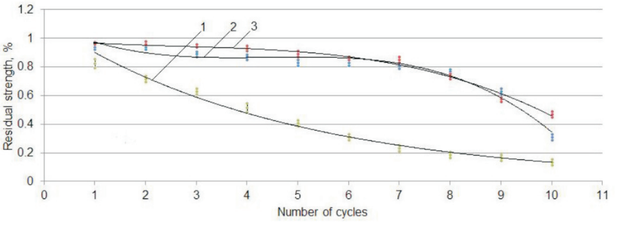

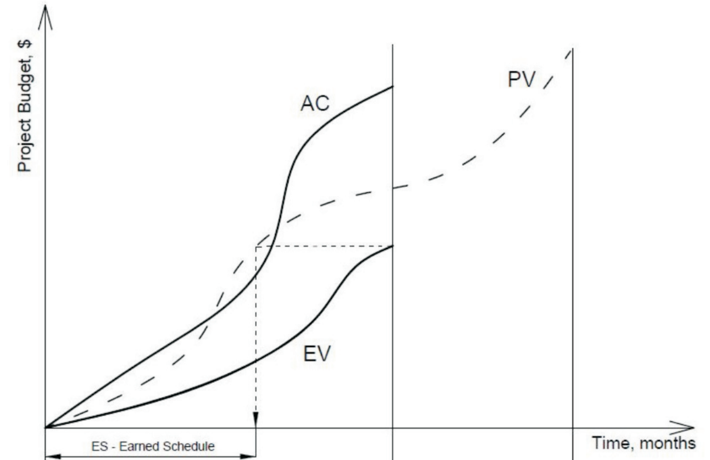

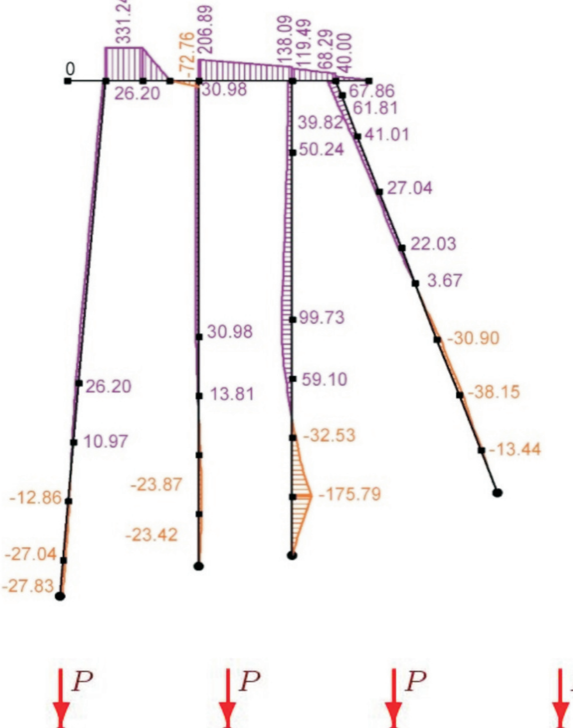

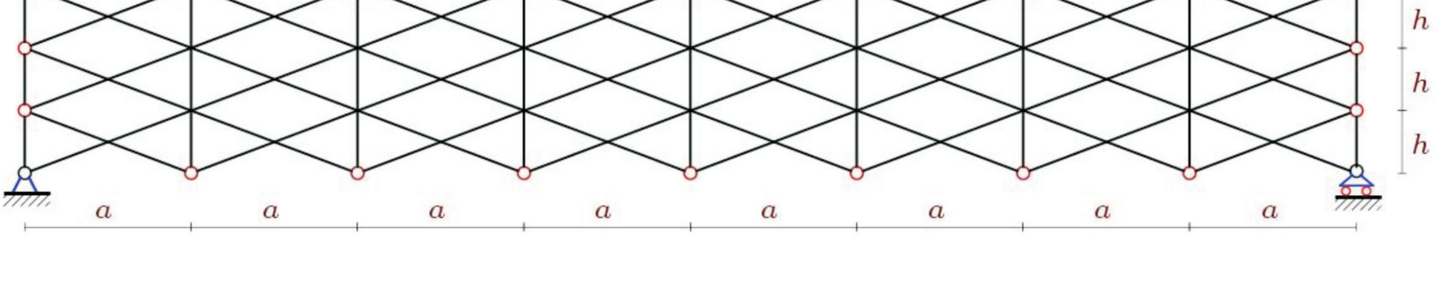

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Recycling of spent battery electrolytes for construction material production

Получение строительных материалов из отработанных аккумуляторных электролитов

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Key words: spent battery; electrolyte; recycling; construction material

Ключевые слова: отработанный аккумулятор; электролит; переработка; строительный материал

Abstract. The issue of spent battery electrolytes recycling in the process of construction material production and also the negative impact of such wastes on environment are considered in this article. Modern methods of processing as spent batteries and their electrolytes are analysed. In this article proposed to recycle spent acid battery electrolyte, applying it as additional component of the mixing liquid for phosphate systems, featured by high mechanical properties, heat and acid resistance. Also authors proposy to recycle spent electrolyte as preparation liquid for mortar made of blast-furnace slag, which is used for construction material production. Values of thermodynamic functions, obtained as a result of calculations, show that hardening reaction of the phosphate system is spontaneous ($\Delta G°298$ = -423.6 kJ/mol) and exothermic ($\Delta H°298$= -719.6 kJ/mol). Hardened system represents strong artificial stone which can be used, for example, in construction of floors in workshops with aggressive media, for lining of towers, etc. Test results have shown that maximal strength value of artificial stone (17 MPa) is reached when 15 % of spent acid battery electrolyte is added into phosphate system. The method of geoecological assessment of designed technological solutions is also described.

Аннотация. В статье рассмотрена проблема утилизации отработанных аккумуляторных электролитов с точки зрения негативного воздействия на окружающую среду. Проанализированы современные способы переработки, как отработанных аккумуляторов, так и их электролитов. Авторами предложено новое технологическое решение утилизации отработанных аккумуляторных (кислотных и щелочных) электролитов. Суть решения заключается в применении отработанных аккумуляторных электролитов в качестве дополнительного компонента жидкости затворения в фосфатных системах, отличающихся высокими механическими свойствами, термо- и кислотостойкостью. Также предлагается использовать отработанный электролит в качестве жидкости затворения при получении шлакощелочных материалов. Полученные значения термодинамических функций свидетельствуют о том, что реакция твердения фосфатной системы является самопроизвольной. Результатом отвердевания фосфатной системы является прочный искусственный камень, который рекомендуется использовать для устройства полов в цехах с агрессивными средами, для футеровки башен и т.д. Результаты испытаний показали, что максимальное значение прочности искусственного камня (до 17 МПа) достигается при введении в

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фосфатную систему до 15% отработанного кислотного аккумуляторного электролита. Описан метод геоэкологической оценки разработанного технологического решения.

Introduction

Nowadays reality implies that there is no any sector of national economy, which could work without batteries, while their average lifetime is not exceeding five years [1]. Among wide range of the batteries the most environmentally dangerous are acid [2–4] (mainly automotive) and alkaline batteries [5] (e.g. Ni–Cd, Cd-Ni, Ni–Fe). For example, according to the data by author [6] in Moscow only up to 4000 tons of waste batteries are piled up annually. Each battery is container filled with an electrolyte for 11-30 % of the total [7, 8], which represents aggressive substance and it is dangerous to the environment.

During operation of acid batteries, electrolyte (most commonly it is sulphuric acid) accumulates highly toxic substances, such as lead (16.9 %), its compounds: $PbSO_4$ (19.7 %), PbO_2 (17.9 %), PbS (3.6 %) and other components: Sb (0.6 %), H_2SO_4 (18.1 %), H_2O (10.1 %), $[-CH_2-CHCl-]_n$ (3.1 %), $[-CH(CH_3)-CH_2-]_n$ (10.0 %) [9]. Lead is polytropic toxicant with negative effects on immune, nervous and cardiovascular systems due to, first of all, its denaturizing effect upon tissues and cells of the body [10]. In addition, lead has mutagenic activity. Being a heavy metal, lead has toxic impact on human health [11, 12] and it is classified by the Russian Standard GOST 12.1.007-76 “Occupational safety standards system. Noxious substances. Classification and general safety requirements” as first hazard class. On the other side lead as a waste is classified by [13] as moderately dangerous and by [14] mirror non-hazardous.

Lead processing facilities, as battery scrape consumers, apply technologies which do not involve recycling of electrolytes, and direct recycling of electrolytes is performed by special chemical plants using rather expensive technology. Furthermore, the widespread disposal way of the spent electrolyte by the unscrupulous consumers is its discharge into sewers or upon soil at unauthorized dumps. Recovery period of environmental components after such negative impacts is over 30 years after complete removal of exposure source.

Spent alkaline battery represents polymer container filled with electrolyte. Main components of this electrolyte are Ni (23.5 %) and Cd (22.7 %) compounds, which also have first hazard class. Also electrolyte of alkaline battery includes Na_2O (3.2 %), LiOH (5.5 %), H_2O (13.9 %), Fe compounds (0.2 %) and $[-CH_2-CH_2-]_n$ (31.0 %) [9, 15, 16]. Being both heavy metals, Ni and Cd can accumulate in human body, causing diseases of various kinds [10]. For example, excess of Ni causes changes in liver, kidneys, cardiovascular, nervous, and digestive systems, it damages metabolic processes, and can lead to development of asthma, atherosclerosis, anaemia, diabetes mellitus, dementia, nasopharyngeal and bronchial cancers, and other diseases. Cadmium is not less dangerous, too. When entering into human body, it can block functions of some enzymes, affect the liver, kidneys, pancreas, lead to lung cancer and bone deformities, impact central nervous system, and so on. Biological half-life of cadmium in human body lasts 20 to 30 years.

In some countries the recycling process of spent batteries includes the following steps: collecting the life-expired batteries, pouring out and neutralization of the electrolyte, the separation of the battery lead and plastic parts, recovery of lead by smelting lead scrap and the refining of lead bullion obtained. Widely used in practice methods of recycling spent batteries described in works D.C.R. Espinosa et. all [17], J. Xu et. all [18], X. Zeng et. all [19], G. Pistoia et. all [20], J. Nan et. all [21], E. Sayilgan et. all [22], T. Georgi-Maschler et. all [23], A. Zabaniotou et. all [24], N. Zhu et. all [25], A. Chagnes and B. Pospiech [26].

Meanwhile methods of recycling spent battery electrolytes may be tentatively divided into three groups:

1. Electrolyte neutralization up/down to pH value 7 followed by discharge of neutralized electrolyte without any further use of resulting products. It should be noted that neutralization is used only when a small amount of electrolyte is gained, and if it does not contain organic impurities.
2. Spent battery electrolyte application in the other manufacturing processes, for example, recycling of the acid electrolyte in production of sulphate fertilizer.
3. Regeneration of the electrolyte in order to obtain a commercial product, such as sulphuric acid – this method nowadays is one of the most common ways of using spent battery electrolytes. This method includes: thermal decomposition, extraction of organic impurities, adsorption, catalytic oxidation with hydrogen peroxide, coagulation and evaporation. Among listed above

methods the most often used is thermal decomposition (pyrogenic method), in which under the action of high temperatures (about 950-1200°C) regeneration of acid occurs.

Also, some technological approaches are known, which allow, on the one hand, to neutralize acid or alkaline electrolyte, and, on the other hand, to block the toxic substances, for example, heavy metals. In the publications [27–29] authors propose to perform neutralization of the electrolyte with converter sludge in mass ratio (0.65–0.95):1. Into the vitreous viscose mass formed, granulated blast-furnace slag is added as a binder under stirring.

In spite of considerable progress in this area the problem of spent battery electrolyte recycling still remains relevant primarily because of the high cost of the proposed methods.

In this connection the aim of this article to develop technological solution for the recycle of spent battery electrolytes. To achieve this aim the following tasks are set:

– to conduct lab tests on the application of spent battery acid electrolyte for obtaining phosphate systems;

– to conduct lab tests on the application of spent alkaline battery electrolyte as preparation liquid for mortar made of blast-furnace slag;

– to carry out comparative evaluation of the developed technological approach with already known ones.

Methods

In this paper, phosphate systems were obtained in the laboratory by mixing *FeO* powder, Cambrian clay and H_3PO_4 sand, diluted with spent battery acid electrolyte. The main chemical reaction which occurs during production of phosphate materials schematically can be described with equations (1) and (2).

$$Al_2O_3\cdot 2SiO_2\cdot 2H_2O + 6H_3PO_4 \rightarrow 2Al(H_2PO_4)_3 + 2(SiO_2\cdot 2H_2O) + H_2O \quad (1)$$

$$Al_2O_3\cdot 4SiO_2\cdot 2H_2O + FeO + 2\tfrac{2}{3}\, H_3PO_4 + 3\tfrac{1}{3}\, H_2O \rightarrow \rightarrow \tfrac{1}{3}\, Fe_3(PO_4)_2\cdot 4H_2O + 4(SiO_2\cdot H_2O) + 2Al(OH)_2H_2PO_4(s) \quad (2)$$

Taking into account wide range of recently known recycling and application methods of spent battery electrolyte, it is reasonable to carry out comparative evaluation of the developed technological approach with already known ones. For such evaluation, property quality (PQ) method [34] can be used, which allows carrying out comparison of different technological approaches within single scale. Following aspects of comparison recycling methods for spent battery electrolyte are selected:

- aspect of geo- and ecology protection (PQ^1);
- economical aspect (PQ^2);
- operational aspect (PQ^3).

Due to environmental hazard of spent battery electrolyte, importance of comparison aspects can be assigned as 50%, 30% and 20%, respectively.

For the aspect of geo- and ecology protection, following properties are taken into account:

- regeneration possibility (PQ^1_1);
- absence of wastes after regeneration (PQ^1_2);
- possibility to obtain new useful product (PQ^1_3).

In this case importance of the properties can be assigned as 40 %, 30 % and 30 %, respectively, depending on their effectiveness.

For the economic aspect, following properties are taken into account:

- utilization cost (PQ^2_1);
- final product cost (PQ^2_2).
- Importance of the properties 50% for each.

For the operational aspect, following properties are taken into account:

- the need for special equipment (PQ^3_1);
- processing time(PQ^3_2);
- availability of method in the Russian Federation(PQ^3_3).

Importance of the properties e assigned as 40 %, 30 % and 30 %, respectively.

In accordance with the method for determining PQ index, the best condition is assigned to value 1, the worst condition is assigned to value 0.

PQ index for each area is calculated by the formula (3):

$$PQ=0.5\cdot\left(0.4\cdot PQ_1^1+0.3\cdot PQ_2^1+0.3\cdot PQ_3^1\right)+0.3\cdot(0.5\cdot PQ_1^2+0.5\cdot PQ_2^2)+0.2\cdot(0.4\cdot PQ_1^3+0.3\cdot PQ_2^3+0.3\cdot PQ_3^3) \quad (3)$$

Results and Discussion

The term "recycling" in this article means application of spent battery electrolyte to obtaining environment-friendly product, which has useful consumer properties. As elaboration of works led by L. Svatovskaya [30-34], who had shown that phosphate systems possess unique environment protection properties due to capabilities to neutralize and block pollutions, this article proposed to recycle spent acid battery electrolyte, applying it as additional component of the mixing liquid for phosphate systems, featured by high mechanical properties, heat and acid resistance.

Phosphate hardening is based on the chemical process of interaction between oxides of d-metals and finely ground special compositions with orthophosphoric acid. Binding properties of the system "metal oxide – orthophosphoric acid" depend on ionic potential, which represents ratio of electronic charge of the ion to its effective radius. As far as ionic potential of cations in groups with homogeneous electronic structure decreases, hardening process accelerates, while increase of ionic potential causes deceleration of this process.

Synthesis of phosphate binders is based on high chemical activity of phosphate solutions towards powders of different chemical compositions. In this case phosphate compositions are considered as dispersed systems of "solid – liquid" type, where (with a certain speed) irreversible chemical reactions of acid-base interaction type take place. It is appropriate to use mineral products of complex chemical and mineralogical composition, e.g. clay, as primary solid components of these phosphate systems.

Values of thermodynamic functions, obtained as a result of calculations, show that hardening reaction of the phosphate system is spontaneous (ΔG°_{298} = -423.6 kJ/mol) and exothermic (ΔH°_{298}= -719.6 kJ/mol). Considerable amount of the heat produced by reaction (1) promotes removal of excessive moisture from the system and helps crystallization of products. These factors result in formation of practically insoluble and safe for environment phosphate bonds. Thus, hardened system represents strong artificial stone which can be used, for example, in construction of floors in workshops with aggressive media, for lining of towers, etc.

Test results have shown that maximal strength value (17 MPa) is reached when 15 % of spent acid battery electrolyte is added into phosphate system. After saturation of the material with water during two days its strength practically does not change due to formation of phosphates which have low solubility, increased thermodynamic stability and tendency to easy crystallization. Analysis of the aqueous extracts from obtained artificial stones shows that these extracts do not contain contaminants.

Identifying feature of these phosphate materials is small setting time, which in most cases is a positive feature of material, allowing quick extraction of products from moulds. However, in some cases quick setting is undesired. Studies have shown that addition of spent battery electrolyte to phosphate system retard setting time up to 1.5 hour, which increases convenience of placing binder mixture.

Thus, not only a technological approach, which provides comprehensive utilization of spent acid battery electrolyte is proposed, but also a new material with high strength properties is obtained.

Authors proposed to recycle spent alkaline electrolyte as preparation liquid for mortar made of blast-furnace slag, which is used for construction material production. Materials made of blast-furnace slag are obtained by mixing finely ground, granulated blast-furnace slag and sand with an alkaline component, such as sodium silicate. Hardening of the mortar made of blast-furnace slag and alkaline component is based on the following chemical reaction (4):

$$2.5(C_2AS)(s)+0.5(Na_2O{\cdot}H_2O)(s)+12.5SiO_2{\cdot}H_2O(s)+Na_2SiO_3(s)\rightarrow \rightarrow 2.5[Na_2O{\cdot}Al_2O_3{\cdot}4SiO_2{\cdot}2H_2O](s)+C_5S_6H_{5,5}(s)+3.5H_2O(l) \quad (4)$$

Thermodynamic calculations have shown that reaction (4) is exothermic (ΔH°_{298} = -3042.6 kJ/mol). Physical and mechanical properties of obtained materials are presented in Table 1.

Table 1. Physical and mechanical properties of material containing spent alkaline battery electrolyte

№	Composition			Compressive strength, MPa, after 27 days
	Slag, %	Sand, %	Spent alkaline electrolyte, % of slag	
1	70	30	0	20.5
2	70	30	3	21.5
3	70	30	4	21.0
4	70	30	6	25.3
5	70	30	8	25.4
6	70	30	10	25.4
7	70	30	12	13.2
8	70	30	15	12.5
9	50	50	0	11.5
10	50	50	2	12.3
11	50	50	4	13.0
12	50	50	6	12.8
13	50	50	8	8.7
14	50	50	12	7.8
15	50	50	15	7.0

Addition of 3 to 10 % spent alkaline battery electrolyte into slag-alkaline system increases material strength up to 24 %, as compared to the reference sample and it amounts 25.4 MPa. Aqueous extracts from obtained materials do not contain contaminants.

As a result, each method of recycling can be estimated by the PQ-method in interval from 0 to 1, which gives possibility to conclude which recycling method of spent battery electrolyte is most advantageous. Preliminary calculations show that PQ-index of proposed technological approaches exceeds PQ-indices of currently used technological approaches.

Conclusions

4. Studies have shown that spent battery electrolytes (acid and alkaline), which are dangerous for the environment, can be recycled by application as additional components for production of either phosphate or blast-furnace slag and alkaline binding systems, which are based on spontaneous exothermic chemical reactions.

5. Resulting products of such reactions have low solubility product values, and it allows concluding that deactivation of spent battery electrolytes is based on blocking them in artificial stone, which represents a useful and environmentally safe product.

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Computation of mooring quay in the form of pile grillage

Расчет причальной набережной в виде свайного ростверка

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Key words: hydraulic engineering; construction; high pile grillage; calculation method; coefficients of subgrade reaction

Ключевые слова: гидротехническое строительство; конструкция; высокий свайный ростверк; метод расчета; коэффициенты постели

Abstract. The calculation of a pile grillage quay, as a frame with rigid anchored legs, using in technical literature and normative documents, is called a method of Gersevanov. The transformation of a manual calculation to a computer calculation almost preserves an existing design diagram. The article presents the practical implementation of a proposed early an engineering universal calculation method for quays. In the proposed design diagram of a frame with an elastic anchorage of the bases of frame legs is used instead of the rigid anchorage. It uses stiffness characteristics of soil as the variable coefficients of the subgrade reaction. An engineering solution of determining a diagram of a lateral earth pressure in a silo of variable width and the engineering calculation method of the pressure redistribution on the back sheet pile wall from external ground pressure on the inclined pile row are showed in the article. Taking into account the deformation characteristics of soil the comparative calculations showed their significant impact on the internal forces in the elements of the quay in comparison with the N.M. Gersevanov’s method.

Аннотация. Расчет свайной набережной ростверкого типа в виде стоечной рамы с жесткими заделками стоек, используемый в технической литературе и нормативных документах, представляет метод Н.М. Герсеванова. Перевод ручного расчета на компьютер практически сохранил существующую расчетную схему. Приведена практическая реализация, предложенного раннее инженерного универсального метода расчета причальных набережных. В предлагаемой расчетной схеме стоечной рамы вместо нижней заделки стоек принимается упругая заделка. При этом используются жесткостные характеристики грунта, в виде переменных коэффициентов постели. Получено инженерное решение определения эпюры бокового давления в силосе переменной ширины, показан инженерный прием вычисления перераспределенного давления на заднюю шпунтовую стенку от внешнего давления грунта на наклонный свайный ряд. Сопоставительные расчеты с учетом деформационных характеристик грунта показали их значительное влияние на усилия в элементах набережной по сравнению с методом Н.М. Герсеванова.

Introduction

The research object is a construction of the high pile grillage quay. This construction was used extensively in the last century in USSR and now used abroad. Increasing demand of quay constructions in the form of the high pile grillage is associated with increase of depths near quays. In addition, the use of reconstruction variants of urban quays in the form of the pile construction with inclined piles is more economical compared to a low pile grillage. The transverse section of the quay in the form of the high pile grillage with the back sheet pile wall is shown in Figure 1, a.

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A significant amount of works by Russians and foreign scientists describe the development of rational methods of the static calculation of the pile grillage quays.

In the first time, N.M. Gersevanov gave the general solution of the calculation problem of the pile quays with the rigid grillage, treating them as a frame with an absolutely rigid beam on elastic supports – piles with anchored bases (1913). Equating of the unknown displacement of the grillage lower edge (vertical, horizontal, and its rotation) N.M. Gersevanov received three canonical equations of the deformations method. Their solution allowed passing from a displacement of grillage to internal forces in the piles. These equations are usual conditions of the equation of the rigid body equilibrium.

Except the analytical method of calculation developed by N.M. Gersevanov, Russian scientists B.M. Lozovsky, F. Dimentbergom, V.S. Hristoforova proposed the solution of the same problem by graphical methods. The calculation methods developed by Vyunshem and Nekkentvedom are special cases of solutions of N.M. Gersevanov. F. Dimentberg developed the method of calculating of the pile quay with regard to a stage of destruction.

G.S. Spiro developed calculation of spatial pile constructions with the rigid grillage as the development of the calculation method of N.M. Gersevanov. Note, that the solutions proposed above are associated with a transfer of the origin of coordinates in the elastic center, which reduced the number of the canonical equations of the deformations method.

In recent years works associated with the calculation of the pile grillages, usually use the classical design diagram [1–3]. This definition of piles anchorage in the ground cannot be correct because it uses Blum-Lomeyera's method, which does not take into account the relationship between the pressure and the displacement of a pile in the ground. For this reason, the passive pressure on the foundation to the vertical wall is untrue.

Note the works of foreign scientists on the calculation of the pile grillage quays [4–16]. In the last 10–15 years abroad, the calculation of the berthing facilities is performed in the program Plaxis, using the finite elements method for the continuum model. This model developed for construction materials is not exact for soil. It gives the approximate picture of the behavior of the pile foundation in the discrete soil medium.

Review modern status of the problem shows that the solution N.M. Gersevanov is used in technical literature and normative documents for the calculation of the pile grillage construction as frame with rigid anchored bases of piles. At that, an appointment of the depth of the anchorage of piles is arbitrarily and determined by the method of elastic line does not match an actual structures work. Assessing an originality of the calculation method of grillage-type structures, which was proposed N.M. Gersevanov, note that this method misrepresents a design diagram of structure because it is the deformation-free method for an interaction of pile foundation in ground. This introduces a certain imprecision in a determination of internal forces in elements of a construction.

In existing technical literature [17–19] and normative documents [20, 21] each type of a port hydraulic engineering structure has its own calculation method. The transformation of a manual calculation to a computer calculation reduced substantially complexity of computing works, but did not use completely great automation capabilities, because most of calculation methods are based on provisions and assumptions, which were laid in the era of lack of computers.

The calculation of mooring constructions in software systems which using the model of continuous medium model allowed to obtain more complete picture of working constructions in soil. However, the calculation of these constructions in programs, which use the model of continuous medium, is designed for structural materials and is not strict. It gives the approximate picture of a behavior of the pile foundation in discrete soil medium. It is required the additional verification of the calculation results, due to a wide values range of the initial data and in cases of adoption of rigid anchored bases of piles in ground [22–24].

In 2002, the universal engineering method of calculation was offered to simplify the understanding of a mechanism of interaction elements of a structure with soil and use a computer maximally. This method can be applied to any construction of a quay [25] and complemented in the work [26].

In the settlement relation, every type of the quay structure is submitted in the form of the specific combination of beams interconnected by condition of a joint deformation of individual elements and by common structure, each of which has different forms of interaction between themselves. This approach, on the one hand, unifies calculation methods and, on the other,– respectively allows refusing from some simplifications adopted in existing calculations, for example, in the frame type of quays, the pile grillage type: rigid anchored bases of piles in ground and a hinge support in the grillage, a limit lateral pressure and etc.

The purpose of the proposed work is the practical implementation of the engineering universal method for the calculation of the quay grillage constructions. At the same time the design diagram and impact of assumptions, used in the departmental standards, on internal forces in constructions are measured in this work.

Theoretical background

Calculation model

Structure dimensions of the non-rigid grillage are defined in initial data [19]. The determination of internal forces in piles is necessary for determination burial depths of the piles. The characteristic feature of the non-rigid girder is the distribution of vertical and horizontal loads on supports through a girder. It is considered as a simply supported beam. The pile foundation is not represented as an integral unit but as a system of independent supports united only by the non-rigid girder. The vertical pressure on every pile or the inclined pile is defined as reaction of the simply supported beam. The horizontal loads (H) are equal with the resultant active pressure on the vertical, passing through the back. The value of the horizontal loads is usually absorbed partially by single inclined piles and mainly – paired inclined piles.

Obtained vertical and horizontal forces from the condition of the non-rigid grillage determine the cross section of elements of the high pile grillage previously. Subsidence of the piles is determined previously by values of internal forces in compression elements and the value of the coefficient of subgrade reaction on the pile tip.

In the calculation in the program SCAD the high pile grillage is represented as the statically indeterminate frame of spacer type with spatial elements partially submerged in foundation soil, which is described by a model with two coefficients of subgrade reaction (horizontal along the length of piles and vertical at the bottom end of pile) (Fig. 1, b).

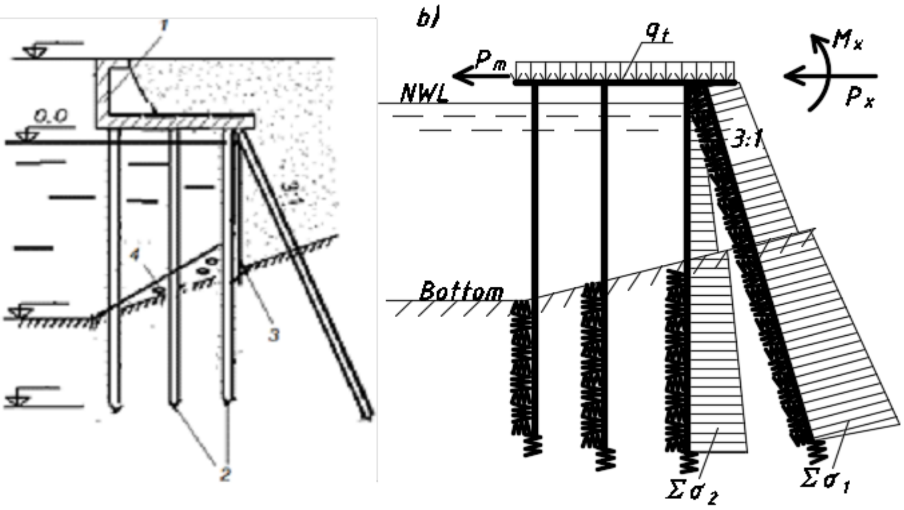

Figure 1. High pile grillage with back sheet pile wall: a –transverse section: 1 – grillage;2– piles;3 – protection of soil from dornit; 4 – riprap; b – design diagram. The legends: q_t – total vertical load;P_m – mooring load on the one running meter (1 rm); P_x, M_x – resultant active pressure on the grillage (superstructure) and moment from its; $\Sigma\sigma_1$, $\Sigma\sigma_2$ – diagrams of resulting pressure on the sheet pile wall and inclined pile. The darkened part of the piles' elements of the construction in ground (in the form of springs in two directions) represents the use of the generalized model of Fuss-Winkler

A geometric image of the design diagram is performed by entering nodes of the frame elements of the design diagram, and then – image of elements themselves (entering elements). Each element of the design diagram is introduced individually. It is recommended to assign the origin of coordinates at the first node.

The design diagram of frame is set by plots’ coordinates of reference nodes. The spatial framework is chosen in the calculation of the framework. It is recommended to divide front wall sections, which interact with ground through the variable height coefficient of the subgrade resistance, into elements.

Front wall sections interacting with ground through the variable height coefficient of the subgrade resistance it is recommended to split into elements of a width of 1.0 m and of 2.0 m for remaining sections.

The numerical description of stiffness characteristics is used in this work. The longitudinal and flexural stiffness of frame element EI contains the deformation module of steel $E = 2.1 \times 10^8$ kN/m^2 and the deformation module of reinforced concrete $E = (2.7 – 3.67) \times 10^7$ kN/m^2, an area A, m^2 and a moment of inertia I, m^4 on one meter-wide. The stiffness of the pile row is divided into a number of pile steps.

Deformation characteristics of soil

The Fussa-Vinkler's model became widespread in the practice of port design in the calculation of thin quay walls. This model uses the deformation characteristic of soil in the form of the coefficients of subgrade resistance. In addition, this model has been criticized for neglect of the soil distributive capacity. However, the revision of the continuum model, which is used in program complexes with data of natural researches, showed that the influence of distributive capacity of soil, which has discrete structure, is overestimated.

The solution of the differential equation of the bending beams leaning on the continuous (winklerovsky) elastic foundation, according to the method of local deformations, does not represent special difficulties, but contains four integration constants, which need to be defined from initial conditions.

Direct experiences show that the coefficient of subgrade resistance for natural soil is not constant, but depends on the value of specific pressure upon ground and the area of transfer of loading that it is necessary to consider at calculations.

Generalized methods of the definition of soil deformations consider general elastic and local inelastic soil deformations. Of these methods, we note P. Pasternak's and V. Vlasov's methods of the two-parameter elastic foundation according to which the soil foundation is characterized by the coefficients of subgrade resistance of compression and local elastic shift. Let us note I. Cherkasov's and G.K. Klein's method of structural-revitalizing deformations, which considers general revitalizing deformations (elastic and adsorptive) and residual (structural). In the latter method, revitalizing deformations are accepted as linearly deformable and characterized by the coefficient similar to the coefficient of elastic half-space.

The calculation theory for structures lying on the deformable foundation with one and two coefficients of subgrade resistance was developed in works, which was made by: A. Dinnik, P. Pasternak, M. Gersevanov, N. Zhukovsky, A. Krylov, A. Umansky, G. Dutov, V. Kiselyov, S. Golushkevich, B. Korolyov, N. Snitko, V. Vlasov, N. Leontyev and many others. Among foreign scientists engaged in this task: H.M. Westergaard, H. Bufler, H. Lieb, G. Meier [27], Y.K. Cheung, O.C. Zienkiewicz [28], C.S. Desai, J.T. Christian[29], A.M. Ioannides, etc.

Due to changes of the coefficient of subgrade resistance on depth (for piles) and in plan (for slabs) used the so-called coefficient of soil stiffness. In the case of the calculation of different types of mooring facilities is proposed to use function of the stiffness coefficient of foundation that connects the stiffness coefficient of soil from elastic and limit states for the entire load cycle [30]. In practical use this function is piecewise linear values in each section of the structural element of five intervals of the entire load. The coefficient of subgrade resistance of foundation for the sheet pile wall considering plastic properties of soil is:

$$К_s = y \cdot К \cdot К_a \cdot К_{pl}, \tag{1}$$

where y – depth of a point under consideration;К–a proportionality coefficient(ED 31.31.55-934); $К_a$ – an anisotropy coefficient for non-cohesive soils is 1, for clay – 0.7–0.8; $К_{pl} = 0.6 – 0.8$ – a coefficient account of the plastic properties of foundation in front of a wall are accepted for burial depths 1–2 m (lower value for loose and weak soils).

For the pile elements are used calculated values of the coefficient of subgrade resistance on the lateral surface of the pile and on a tip of the pile are accepted to JV 24.13330, Pile foundations, Moscow 2011.

Side loads from ground on gantry elements piles

Main lateral loads on the pile grillage with the back sheet pile wall and the gantry sheetpile bulkhead operate on the vertical sheet pile row and the inclined anchored piles with slope of 3:1

(Fig. 2, a). Note that the gantry sheetpile bulkhead is obtained by discarding two front piles in the pile grillage (Fig. 2, a).

Pressure of the ground zone II on the inclined pile row represents a sum of lateral pressure on the inclined plane (σ_{aa}) and the load pressure of soil hanging on piles (σ_h) (Fig. 2, a).

The active pressure on the inclined plane is equal to:

$$\sigma_{aa}= (q +\sum \gamma_i h_i)\cdot\lambda_{aa}, \quad (2)$$

where λ_{aa} – a coefficient of a lateral pressure on an outer row, when $\alpha = 18^0, 25^{II}$.

The active pressure from soil hangs on pile equal (Fig. 2, b):

$$\sigma_h = (q +\sum\gamma_i\cdot h_i)\cdot m\cdot\sin^2 \alpha, \quad (3)$$

where m = 2·d/n·ctgφ (d and n – a diameter and the pitch of piles); $\alpha= 18^0, 25^{II}$ – the angle of inclination to the vertical of the pile.

The active resultant pressure on the pile row is difference of the pressures on its outward from zone II and the zone with internal I (Fig. 2, b):

$$\sigma_r = \sigma_{aa} + \sigma_h - \sigma_i = (q +\sum \gamma_i\cdot h_i)\cdot\lambda_{aa} + \sigma_h - \sigma_i, \quad (4)$$

where $\sigma_i = (\sum\gamma_i\cdot h_i)\lambda_{-aa}$ – the internal pressure on the inclined rear piles row to the angle α.

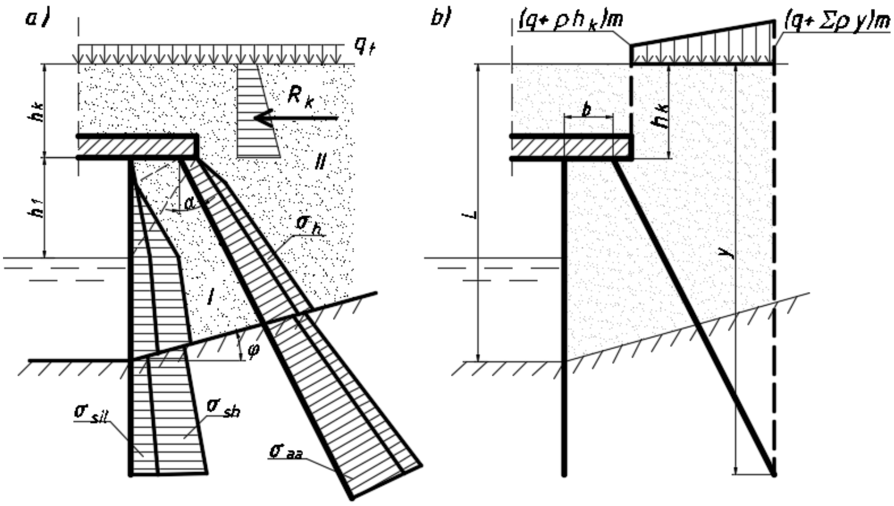

Figure 2. a – diagrams of the lateral pressure on the gantry elements of the piles. Conventions: R_k – resultant of active pressure on the grillage; σ_{sil} – pressure on the sheet pile wall in the conventional silo; σ_{sh} – additional pressure on the sheet piles from the shielding pile row; σ_{aa} – active pressure on the inclined plane of pile row; σ_h – the pressure from the load of soil hanging on piles; b – the load on the inclined anchor piles

The soil pressure in the zone I upon the sheet pile wall of the pile grillage or the gantry sheetpile bulkhead consists of the soil pressure in this zone (conventional silage, σ_{sh}) and the resulting additional pressure of the pile row (σ_r) considering distributing capacity of the piles.

The lateral pressure on the sheet pile wall represents a sum from a conditional silage pressure σ_{sil} in the zone I and the additional pressure from a shielding pile row (σ_{sh}). The last is equal to a difference of the external pressure σ_{aa} from the zone II and a return silage pressure upon the shielding inclined plane of soil σ^1_{sil} in the zone I with considering of the distribution coefficient K_d (Fig. 3, a):

$$\sigma_{shp}= \sigma_{sil} + \sigma_{sh} \quad (5)$$

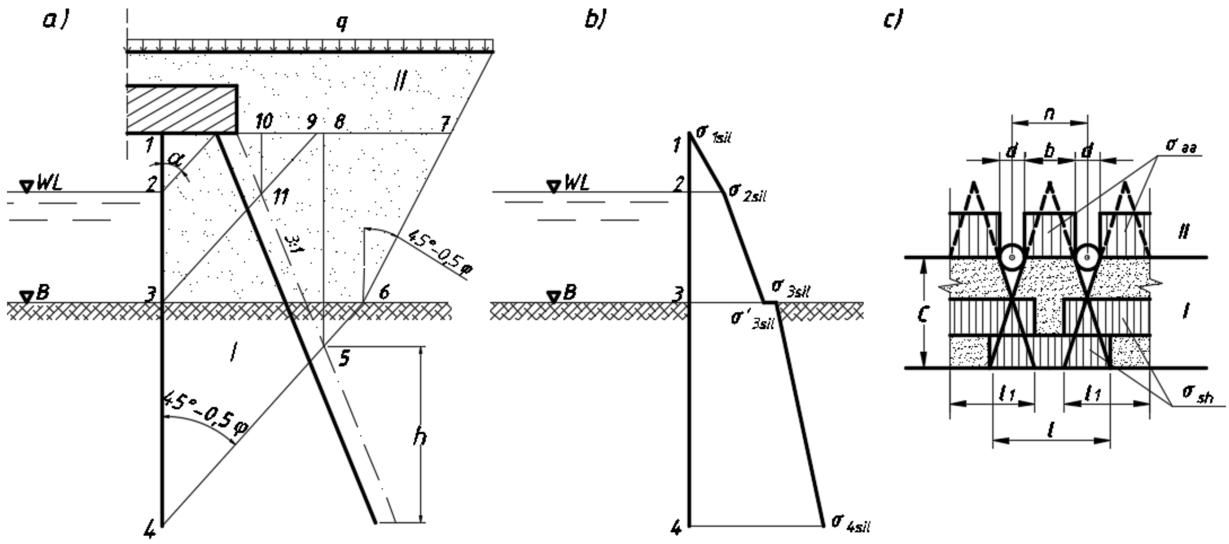

**Figure 3. Design diagrams for determination of lateral pressure in zone I:
a – simplified diagram; b –specified diagram defining intensity from a condition of the maximum plane of a collapse; c – determination of pressure of a pile row upon the sheet pile wall**

The lateral pressure in the silo of variable width. The lateral pressure in the point 2, determined the straight line at the angle 45° – 0.5φ to a vertical from the beginning of the shielding plane of the piles, is equal (Fig. 3, a):

$$\sigma_{2sil} = \gamma\, h_{1\text{-}2}\cdot\lambda_{a\text{-}1}, \tag{6}$$

where $h_{1\text{-}2}$ – a height between points 1 and 2 (Fig. 3, a).

The lateral pressure in the silo of variable width in the points 3 and 4 is defined by the following engineering reception with a margin of safety (Fig. 2, b). Presume that the active pressure in the point 3 is defined by a pressure of the incomplete wedge (prism) of failure 1, 3, 11, 10, and the pressure in the point 4 respectively the prism 1, 4, 5, 8. Taking into account this circumstance the lateral pressure 3 and 4 is equal (Fig. 3, a):

$$\sigma_{3sil} = \gamma_2\cdot h_{2\text{-}3}\cdot\lambda_{a1}; \tag{7}$$

$$\sigma^{1}_{3sil} = \gamma_2 h_{2\text{-}3}\cdot\lambda_{a2} \tag{8}$$

$$\sigma_{4sil} = \gamma_2\cdot h_{4\text{-}5}\cdot\lambda_{a2}, \tag{9}$$

where $h_{2\text{-}3}$ – a height between points 2 and 3 (Fig. 3, a); $h_{4\text{-}5}$ – a height between points 4 and 5 (Fig. 3, a).

Soil adhesion is not considered in the margin of safety. In case it appears that $\sigma_{3sil} < \sigma_{2sil}$, the value σ_{3sil} is determined by the straight line passing through the values σ_{3sil} and σ_{4sil}.

In determination, the additional pressure upon the sheet pile wall σ_{sh} from the shielding pile row in departmental norms [31] was used similar N.A. Smorodinsky's ideas (1937) about a distribution of the pressure upon the sheet pile wall and the pile is proportional to their rigidity. This offer was progressive at the beginning of origin of calculations of the pile constructions, but does not consider influence of distance of the sheet pile wall and also the cross section of piles and the step of the pile row along a line of a cordon. These factors have more essential impact on the sheet pile wall and piles than a ratio of their rigidity. The simplified decision for the definition σ_{sh} by the distribution coefficient K_d is given [32] below. The pressure upon the sheet pile wall from the zone II is calculated depending on the step of piles through K_d:

$$\sigma_{sh} = K_d\cdot(\sigma_{aa} - \sigma^{1}_{sil}); \tag{10}$$

$$K_d = (b/\mathrm{n})\cdot(2\cdot l_1/l), \tag{11}$$

where n – the step of the piles; $l_i = c_i\cdot\mathrm{tg}\,\varphi$ – a variable zone of the partial distribution of resultant load behind the sheet pile wall (c_i – a variable distance from the pile to the sheet pile wall); l = b +2·c_i·tgφ – a variable zone of distribution of the resultant load between the site b piles of the sheet pile wall; d – the diameter of the shielding pile (Fig. 3).

Results

Initial data of calculation grillage: the load on surface of the backfill q = 40 kN/m^2; the free height of the wall H = 12.5 m, R/C sheeting pile 50 x 50 cm, the cross section of the anchoring piles 40 x 40 cm, the step of piles n = 1.5 m, the incline of piles 3:1 (α= 18°25'), h_p = 2.5 m.

Design characteristics of foundation soil: banded clay γ_w= 10 kN/m^3, φ = 20°, c = 30 kPa; backfill soil: γ = 18 kN/m^3, γ_w = 10 kN/m^3, φ = 30°. The elastic modulus of the construction material E_{sh} = E_s= 2.1·10^{4}MPa. Deformation characteristics of backfill soil and the foundation in the form of the horizontal and vertical coefficients of subgrade resistance for the sheet pile wall and the piles were taken using normative documents.The calculation results are shown in Table 1 and Figures4–6.

Table 1. The results of calculation the quay using two methods

Name	The calculation by the classical method [31]	The calculation of the proposed method in the program SCAD[25]
Moment in the top anchorage of sheet piles, [kN·m]	-67.48	-374.35
Moment in the top anchorage of the piles, [kN·m]		
The first series of piles	-56.96	-223.48
The second series of piles	-36.36	275.48
The third series of piles	125.76	-118.1
Moment in the span of sheet piles, [kN·m]	-76.90	363.34
Moment in the span of piles, [kN·m]		
The first series of piles	-5.745	86.05
The second series of piles	6.03	76.40
The third series of piles	198.60	132.45
Moment in the bottom anchorage of sheet piles, [kN·m]	148.17	0.00
Moment in the bottom anchorage of piles, [kN·m]		
The first series of piles	45.47	-
The second series of piles	48.42	-
The third series of piles	-526.81	-
Longitudinal force in the vertical pile, [kN]		
The first series of piles	-177.64	-334.34
The second series of piles	-173.39	-279.65
Longitudinal force in the inclined pile, [kN]		
The third series of piles	327.18	6.5
Maximum vertical displacement of the structure, [mm]	-2.4526	11.6
Maximum horizontal displacement of the structure, [mm]	-8.13	-65.40

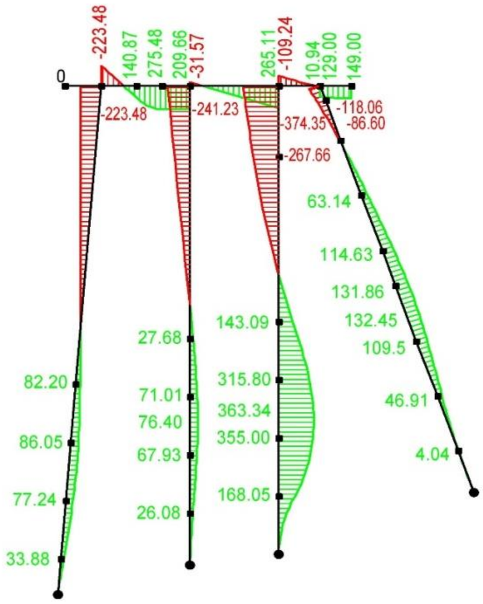

Figure 4. Diagram of bending moment in the pile grillage, kN·m [25]

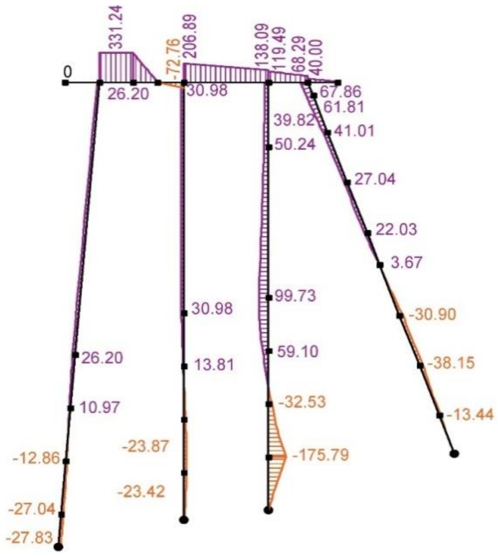

Figure 5. Diagram of shear forces in the pile grillage, kN [25]

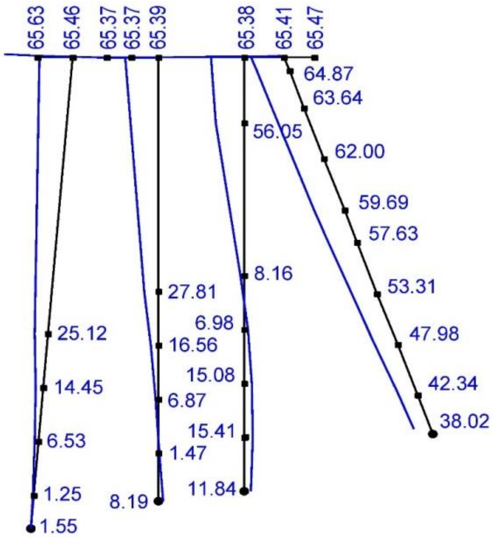

Figure 6. Diagram of displacements of the pile grillage, mm [25]

Discussion

The comparison of the calculation results by two methods revealed their lack of comparability quantities of internal forces in elements of the structure. It arose due to discrepancies of the design diagrams in the considered methods. Therefore, in the calculation, based on [31], the internal forces in the rigid anchored frame legs arise from the ground active pressure on them and from the entire structure rotation about the bottom fixed end. In the calculation, based on [25], the rigid anchorage of the frame legs is not used, but the reactive ground pressure on the inclined pile in the space between piles begins to prevent turn the whole structure. At the same time, the bending moment diagram in the pile can change a direction depending on the stiffness properties of ground.

The universal method with the variation of the horizontal and vertical coefficients of subgrade resistance (without them in filling) upward allows to receive more or less commensurable values of internal forces with departmental regulations. However, in this case, the real stiffness characteristics of foundation soil are distorted.

Conclusions

1. In the existing technical literature and normative documents each type of the port hydraulic structure has own calculation method. The calculation of the pile grillage quay as the frame with absolutely rigid beam on elastic supports-piles with the rigid bottom anchorage is the special solution (deformation-free in ground) of N. Gersevanov's classical method. The transformation of the manual calculation to the computer calculation reduced substantially complexity computing works, but part of calculation methods is based on provisions and assumptions, which were laid without computer.

2. Shows the practical implementation of the proposed early engineering universal method of the mooring grillage quay design. The calculation used stiffness characteristics of the framework beam, piles and soil. There is no soil in the classical method.

3. It is defined the engineering solution of the determination diagram of the lateral pressure of soil in the silo of variable width.

4. It is showed the engineering reception of the calculation of the redistribution pressure on the back sheet pile wall from the ground external pressure on the inclined pile row.

5. The comparative calculations of the mooring quay with different values of the stiffness characteristics of soil, which interacts with the elements of piles, showed their significant impact on the classical design diagram of construction. In the case of the flexible frame legs and the solid foundation soil (excluding the stiffness of backfill soil), internal forces are comparable with the method of Gersevanov, which can be considered as the special case of the proposed method. In other cases, in the proposed method the internal forces in the elements of quay change significantly upwards or downwards in comparison with the classical calculation method.

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Disaggregation of ultrafine powders in conditions of ultrasonic cavitation

Дезагрегация ультрадисперсных порошков в условиях ультразвуковой кавитации

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Key words: high-dispersive powders; additive; aggregates; disaggregation; ultrasound; cavitation; cement; buildings; construction

Ключевые слова: высокодисперсные порошки; добавка; агрегаты; дезагрегация; ультразвук; кавитация; цемент; здания; строительство

Abstract. Use of high-dispersive additives as structure modifiers is one of the most effective instruments of cement composites required technical exploitation characteristics increase nowadays. However, use of high-dispersive powders is complicated by the fact that their particles are consolidated into aggregates. Thus, the major advantage of high-dispersive powder, which is its possibility to form more bonds when their content is very low, appears to be unrealized. One of the promising directions of dispersed phase fine grinding is use of ultrasonic cavitation. Ultrasonic exposure does not always provide the efficiency required due to insufficient information about the effect of conditions defining ultrasonic radiation intensity on amount of aggregates damage. In this regard, it is necessary to define factors and conditions of ultrasonic exposure, which provide effective high-dispersive powders disaggregation. In article acoustic streaming in a fluid occurring due to cavitation and providing mass transfer intensification has been considered. The data about efficiency of cavitation bubble size and large-scale acoustic streaming effect on the distribution of powder particle size were obtained by calculation. The efficiency of high-dispersive powders disaggregation can be improved by using lowered hydrostatic pressure exposure.

Аннотация. Использование высокодисперсных добавок как модификаторов структуры, в настоящее время является одним из эффективных инструментов повышения требуемых технико-эксплуатационных характеристик цементных композитов. Однако использование высокодисперсных порошков осложнено тем, что их частицы консолидированы в агрегаты. Тем самым не реализованным оказывается основное преимущество высокодисперсного порошка – возможность образовывать большее количество контактов при очень небольшом его содержании. Одним из перспективных путей тонкого диспергирования дисперсной фазы является использование ультразвуковой кавитации. Обработка ультразвуком не всегда обеспечивает требуемую эффективность, что связано с недостаточной информацией о влиянии условий, определяющих интенсивность ультразвукового излучения, на степень разрушения агрегатов. В связи с этим необходимым является определение факторов и условий ультразвукового воздействия, обеспечивающих эффективную дезагрегацию высокодисперсных порошков. В статье рассмотрены акустические потоки в жидкости, возникающие в результате кавитации и обеспечивающие интенсификацию массопереноса. Расчетным путем получены данные об эффективности влияния размера кавитационных пузырьков и крупномасштабных акустических течений на дисперсный состав порошка. Установлено, что эффективность дезагрегации высокодисперсных порошков можно повысить при обработке в условиях пониженного гидростатического давления.

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Introduction

One of the promising directions of building material engineering development is hydration and structure formation of binding substance control by means of synthesized high-dispersive modifying agent introduction. Use of such powders allows one to change directionally the matrix structure of cement composites while their producing. Thus, the properties of cement composites can be regulated [1–5].

High-dispersive materials can be used as modifiers in building material engineering. These materials can be both naturally occurring as minerals (schungite, montmorillonite, smectites, palygorskite, chrysotile) and specially made powders of wide nomenclature nowadays [2, 6–8]. That became possible due to the instrument base development and accumulation of material synthesis knowledge.

However, there are two factors complicating the use of high-dispersive powders in the technological practice. Firstly, use of powders both synthetic and natural origin as modifiers is difficult because most of them are consolidated in sufficiently dense aggregates. Secondly, regardless of bond type, traditional disaggregation methods using mechanical impact are not effective with respect to objects of this class. If one cannot break the aggregates, the activate effect of nanocomponents introduced sharply decreases as the major advantage of high-dispersive powder, which is its possibility to form more bonds when its content is very low, appears to be unrealized.

One of the promising directions of dispersed phase fine grinding and disaggregation is use of ultrasonic cavitation. In this case, the destruction of material occurs due to the action of shock waves and liquid microjets. Although cavitation technology is widely used in industry nowadays, nevertheless, the influence of the factors mentioned above on the efficiency of destruction fine particles and aggregates is still insufficiently studied.

Purpose of work is to define factors and conditions of cavitation exposure, which provide effective high-dispersive powders disaggregation.

Methods

There are a number of proved enough conceptions of ultrasonic material dispersion mechanism, according to which the powder particle destruction occurs due to shock waves appearing in medium when cavitation bubble collapsing. Both shock wave and acoustic streaming occurring due to cavitation bubble microexplosion can be the cause of particles and aggregates destruction [10–12]. Taking a value of pressure occurred due to cavitation bubble micro explosion equals 10^2–10^3 MPa [12], and considering, theoretical oxides strength counted according to the Griffith`s formula is 10^4–10^5 MPa [13] and the center of collapsing cavitation bubble located at some distance from solid particle surface, one can conclude that the stress applied to the powder particle is two or three orders lower than its theoretical strength. Thus, in spite of real material strength being considerably lower than theoretical one due to structure imperfection the value of cavitation microexplosion energy is insufficient for destruction of particles characterized by low defect rate. Thereby, the main process determining mesh size distribution change is primarily disaggregation of binded by autoadhesion forces powder particles having considerably lower strength in comparison to uniform particles.

Another factor influencing the process of particles and aggregates destruction can be acoustic streaming occurring during cavitation. Powder particles and aggregates movement and collision occurs near cavitation bubble under the influence of acoustic streaming generated in liquids at the stage of cavitation bubble micro explosion. Large-scale acoustic streaming are considered [11, 12, 14] to be the most influencing on this process as they occur in the whole volume of the medium, compared to small-scale acoustic streaming.

Eckart and Rayleigh streaming are considered to be large-scale acoustic streaming. Eckart originates in free volume of medium, where the sound wave energy absorption occurs. The velocity of such streaming is proportional to the amplitude of oscillating velocity squared and may be determined from the expression given in [14]:

$$V = \begin{cases} V_0\left[\dfrac{1}{2}\left(1-\dfrac{r^2}{r_1^2}\right)-\left(1-\dfrac{1}{2}\dfrac{r_1^2}{r_0^2}\right)\left(1-\dfrac{r^2}{r_0^2}\right)-\ln\dfrac{r_1}{r_0}\right] & when\ 0<r<r_1 \\ -V_0\left[\left(1-\dfrac{1}{2}\dfrac{r_1^2}{r_0^2}\right)\left(1-\dfrac{r^2}{r_0^2}\right)+\ln\dfrac{r}{r_0}\right] & when\ r_1<r<r_0 \end{cases} \quad (1)$$

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where $V_0 = \dfrac{B r_1^2 \rho}{2\mu}$,

where ρ is medium density, r_0 is radius of the cylindrical tube, r_1 is the radius of the acoustic beam, μ is shear viscosity coefficient, *B* is constant.

A feature of Rayleigh streaming is their two-dimensionality and the independence of its velocity from medium viscosity [11, 14]. The latter can be defined by the formulas:

$$V_x = \frac{3V_0^2}{16c}\sin 2kx\left[1-\left(1-\frac{\eta}{\eta_1}\right)^2\right], \quad (2)$$

$$V_y = -\frac{3V_0^2}{16c}ky_1\cos 2kx\left[1-\left(1-\frac{\eta}{\eta_1}\right)-\left(1-\frac{\eta}{\eta_1}\right)^3\right], \quad (3)$$

where $k = \omega / c$, $\eta = y/\delta$, $\eta_1 = y_1/\delta$, δ is the thickness of the acoustic boundary layer: $\delta = \sqrt{2\nu/\omega}$, $\nu = \mu/\rho$ is kinematic viscosity coefficient.

It should be noted that the greater the losses of acoustic energy in the medium, the greater the intensity and velocity of both Eckart and Rayleigh streaming. It does not matter whether the mechanism of these losses associated with the medium viscosity, a chemical reaction, relaxation or caused by the medium nonuniformity (cavitation region, gas bubbles). The irreversibility of acoustic wave energy and pulse losses is only important [10].

The velocity of the corresponding streaming determines the rate of the dispersed phase. Using the equations (1–3) and the methods proposed in [15, 16] the estimates were made for the powder particles of 0.1; 0.01; 0.001 mm sizes. The results showed that the velocity *v* of liquid microjets, with their radius equal in order of magnitude to the minimum radius of the cavitation bubbles, is approximate to the velocity of their collapse and reaches values of the sound velocity in the fluid (1.5–2.0)*10^3 m/s. Since the particle collision process has short duration, one can neglect the energy losses caused by plastic deformation and equate the work of destruction to the magnitude of powder particles kinetic energy, which may be determined using the formula:

$$W_k = K\frac{mV^2}{2}Cv4\pi Rr, \quad (4)$$

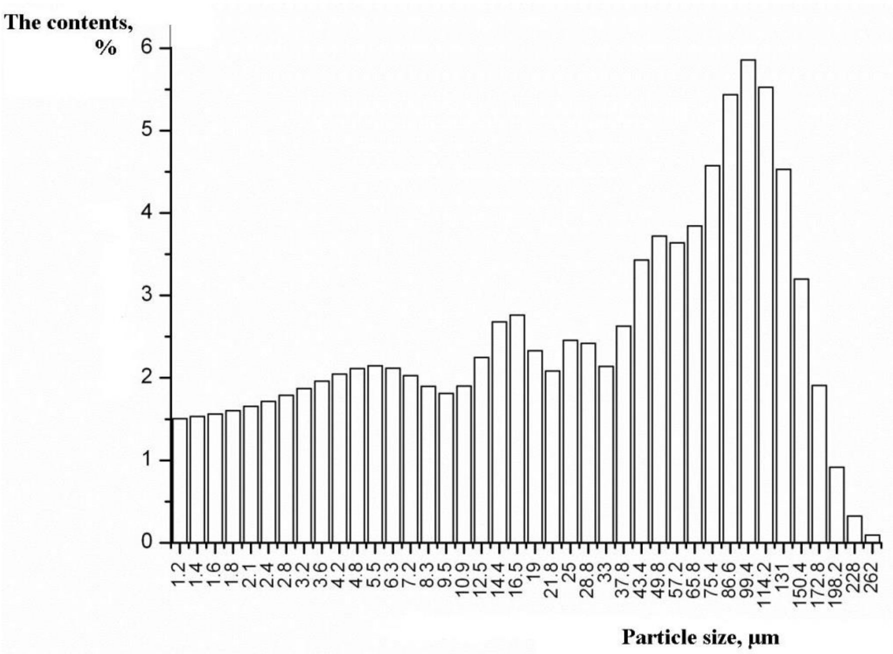
where *m* is mass of powder particles; *v* is velocity of the powder particles; *C* is the concentration; $4\pi Rr$ is cross section of stream.

Calculations made for the powder particles of 0.1; 0.01; 0.001 mm size showed that the magnitude of the kinetic energy was sufficient to destroy them in the process of collisions. Moreover, the stress appeared to be in the range of 10–10^3 MPa, which is comparable to the pressure occurring when the cavitation bubble collapses (10^2–10^3 MPa) [10, 15]. Therefore, one can assert that, in addition to the energy released during cavitation bubbles micro explosion, collisions factor of particles moving under the influence of acoustic streaming can also affect the disaggregation of high-dispersive powders.

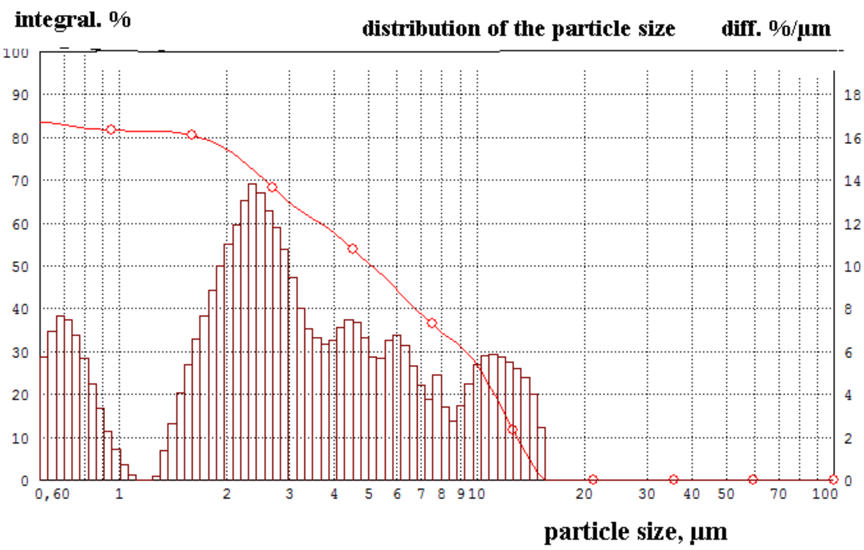
Thus, the efficiency of particle and their aggregates destruction in the conditions of cavitation exposure depends on the shock wave power and velocity of acoustic streaming occurring in the medium when cavitation bubble collapsing. This efficiency is determined by the conditions of ultrasonic influence. It should be noted that the analysis of the data and the results of researches [10, 17, 18] show that, depending on the particle size of the dispersed phase and ultrasonic exposure parameters (amplitude, frequency), one of the factors mentioned above will prevail. Thus, larger particles are destroyed due to cavitation micro explosions acoustic currents. Smaller particles are destroyed due to powerful shock waves, since their pressure pulse has the greatest destructive effect only in the event when the dispersed phase particles float on the surface of the cavitation bubble. That is only possible when the dimensions of the bubble are much larger than particle size [10, 19]. In our opinion, an aggregate of particles, with the size smaller than the pores in such aggregate, can be considered as the version of the latter ultrasonic dispersing scheme. In this case, the gas bubble size is determined by a pore size. Thereby, the parameters of the ultrasonic field should provide a collapse of gas bubbles, which are located in the pores of such aggregates. Thus, the problem reduces to the determination of the ultrasound exposure parameters that provide cavitation of gas bubbles having a size of $R_b < R_{agr}$.

Results and Discussion

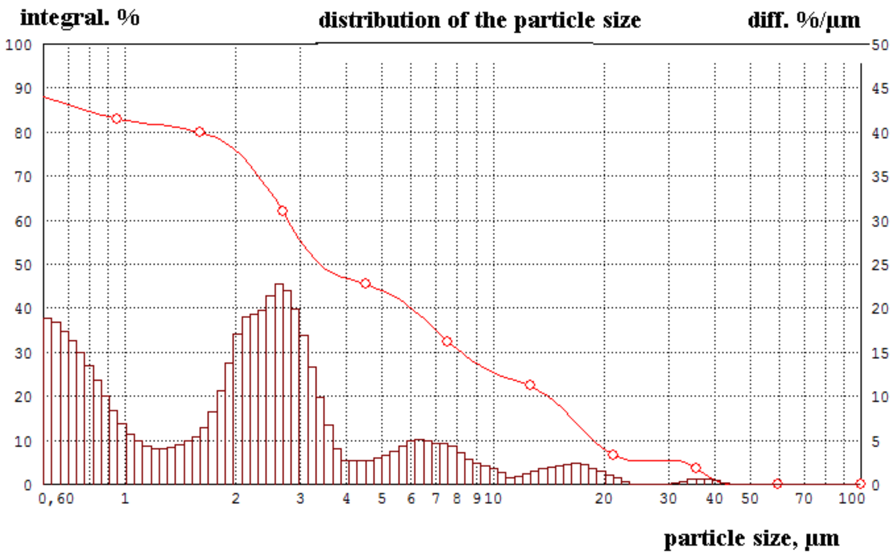
Figure 1 presents data on the changes in length depending on the size distribution of the ultrasonic treatment of calcium carbonate powder, taken as a model. Ultrasound exposure contributes to the destruction of units: this is particularly intense in the initial period of exposure. Further treatment for 15 minutes leads to additional dispersion: number of fractions, consisting of large aggregates is reduced. Thus increasing the average size of less than 1 micron fraction reaches a certain size limit and begins to vary in the range of 600 to 1000 nm due to the wave nature of ultrasound. Moreover, its number tends to a limit not exceeding 15–20 %.



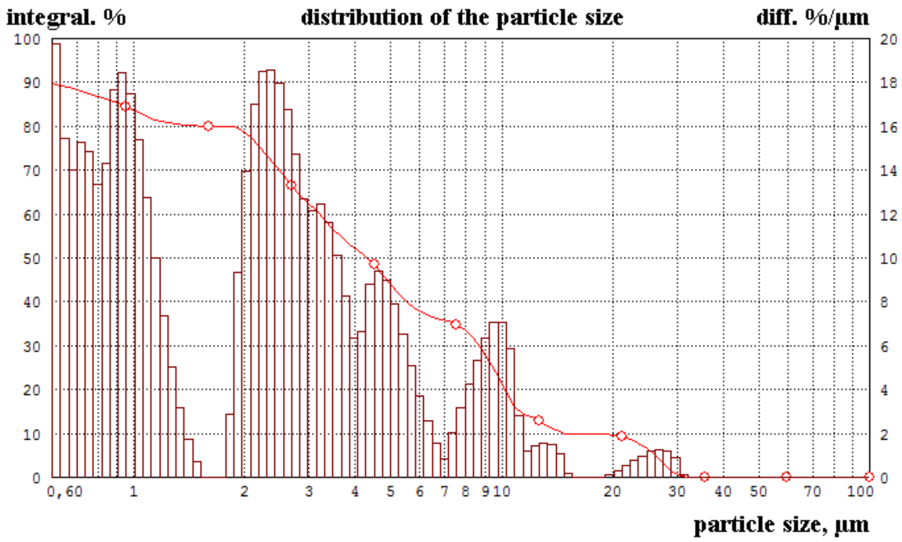
a)



b)



c)



d)

Figure 1. Results chalk granulometric composition depending on the sonication time: a) 0 min, b) 1 minute, c) 10 minutes and d) 15 minutes

To determine the ultrasonic exposure conditions providing the required size of a cavitation bubble, we take into account the following conditions. The gas bubble is located in the solution of gas in a liquid, occupying unlimited space and exposed to time-variant pressure i.e. exposed to ultrasound. Exposed by external ultrasonic pressure the bubble expands (contracts) when pressure dynamically decreasing (increasing). Thus, fluid static balance is disturbed causing the gas diffusion from fluid to bubble (from bubble to fluid). Therefore, the gas concentration in liquid (gas concentration in the bubble) changes. Gas concentration in a solution equal to C_0 is coordinate-independent in the initial time (t = 0). An exception applies to the bubble surface points where the concentration is equal to the saturation concentration C_S, which is pressure determined *p(t)* and hence time-dependent. Let us denote the radius of the bubble by R_n. Given that the occurring fluid movement is spherically symmetric, we introduce the spherical coordinate system and superpose its origin with the center of the bubble.

The experimental data [12] shows that the growth of the bubble caused by diffusion process occurs when the gas concentration in fluid is several times higher than the saturation concentration at the bubble surface. Thus, for the calculations, C_0 can be taken equal to:

$$C_0 = 1{,}5\ C_S$$

Having applied the well-known convection-diffusion equation of molecular physics theory, which allowed us to determine the gas concentration in any point in space at any time given we derived an equation:

$$\frac{\partial C}{\partial t}+v_r\frac{\partial C}{\partial r}=D(\frac{\partial^2 C}{\partial r^2}+\frac{2}{r}\frac{\partial C}{\partial r}), \tag{5}$$

where C is gas concentration in the fluid, D is diffusion coefficient, r is the distance from the center to the origin of the spherical coordinate system, v_r is the velocity of bubble boundaries movement.

According to Fick's law, gas stream to a bubble per unit time is defined as:

$$\frac{dm}{dt}=4\pi R^2 D(\frac{\partial C}{\partial t})_{r=R}, \tag{6}$$

In addition to relations written above, we have the equation of state

$$\mathrm{P}_g V_g=\frac{m_g}{M}RT, \tag{7}$$

where P_g, V_g are gas pressure and volume inside the bubble; M is molecular mass of the gas; B is the universal gas constant; T is gas temperature; m_g is gas mass in the bubble which can be determined from the equation:

$$m_g=\frac{4}{3}\boldsymbol{\pi R_b^3}\rho_g, \tag{8}$$

The gas density inside the bubble can be determined in accordance with the following formula:

$$\rho_g=\frac{M}{RT}P_g, \tag{9}$$

Making use of differential Nolting-Nepairas equation describing the evolution of the gas-filled bubble:

$$R_b R_{\ddot{b}}+\frac{3}{2}R_{\dot{b}}^2+\frac{2\sigma}{\rho R_b}-\frac{1}{\rho}\left(P_0+\frac{2\sigma}{R_0}\right)\frac{R_0^3}{R_b^3}=-\frac{1}{\rho}(P_0-P_m\sin(w\,t)), \tag{10}$$

the relationship between gas pressure inside the bubble and the parameters of the surrounding fluid movement can be determined

$$P_g=\rho\left(R_b R_b^{\cdot\cdot}+\frac{3}{2}R_{\dot{b}}^2\right)+\frac{2\sigma}{R_b}-P_g+P(t), \tag{11}$$

where ρ is the fluid density.

Having calculated time derivative (7) and using relations (6), (9), (11) the following differential equation can be derived:

$$D\left(\frac{\partial C}{\partial r}\right)_{r=R}=\frac{M\rho}{3RT}(R_b^2\,\dddot{R}_b+7\,R_b\dot{R}_b\ddot{R}_b+\frac{9}{2}\,\dot{R}+\frac{3}{\rho}\left[\frac{3}{4}\frac{\sigma}{R_b}+P(t)-P_g\right]\dot{R}_b+\frac{1}{\rho}\frac{d\,P(t)}{d\,t}R_b \tag{12}$$

The last equation, equation (4) and continuity equation

$$\frac{\partial\ v_r}{\partial\ r}+\frac{2v_r}{r}=0 \tag{13}$$

form a system of differential equations. Integration of these equations allows one to define the movement of a single gas bubble boundaries, caused by the action of inertial forces and diffusion in the fluid. The calculations showed that bubbles with an initial radius of 1 to 10 microns collapse at a frequency of 18–22 kHz. The bubbles with an initial radius of more than 10 microns do not collapse but execute complex oscillations. These oscillations occur when the natural frequency of the bubble is close to the frequency of ultrasonic transducer forced oscillations. The oscillation process for bubbles with an initial radius greater than 150 microns is not cavitational. Apparently, this is due to the fact that the initial radius of the bubbles becomes greater than resonance, which according to our calculations and according to the data

[19], for the frequency of 22 kHz is approximately 140 to 170 microns. Thus, bubbles with the initial radius of less than 10 microns are involved in cavitation. The bubbles with an initial radius greater than 10 microns execute complex oscillations and rise to the surface when reaching the resonant dimensions.

The calculations also show that the less is the hydrostatic pressure P_0, the less is the critical pressure P_{cr}. When local sound pressure exceeds the critical pressure, the bubble begins to rise sharply, the fluid collapses and cavitation occurs. Thus, the less is the pressure P_{cr}, the less is the critical radius R_{cr}, the so-called lower cavitation threshold. As a result, the range of cavitating bubbles expands radially, allowing a greater number of bubbles to collapse and thereby to intensify degassing, dispersion and destruction of the structure processes. Therefore, the hydrostatic pressure decrease (i.e. vacuum pumping) allows one to reduce the lower limit of bubble size (table 1). In this case, the occurrence of cavitation under vacuum requires ultrasound of less power [20].

Table 1. The radii of the bubbles corresponding to the resonance frequency for different values of the hydrostatic pressure and ultrasonic frequency (R_b 10^{-6} m)

P_0, Pa	**f , frequency, kHz**		
	18	22	44
1000	37	32	19
10000	68	57	30
100000	205	170	85

Conclusions

The effect of ultrasonic influence conditions on the cavitation bubble size and acoustic streaming caused by cavitation were considered. The obtained results allow us to arrive at the following conclusions:

1. The efficiency of particles and their aggregates destruction during the cavitation exposure is determined by the pressure and velocity of acoustic streaming occurring in medium by the explosion of a cavitation bubble.

2. Depending on the size of aggregates, both shock wave and factor of powder particles and aggregates movement and collision influenced by acoustic streaming can be the cause of aggregates destruction. The efficiency of high-dispersive powders disaggregation can be improved by using lowered hydrostatic pressure exposure.

3. The parameters of the ultrasonic field should provide a collapse of gas bubbles, which are located in the pores of such aggregates. The bubbles with an initial radius of 1 to 10 microns collapse at a frequency of 18–22 kHz.

The bubbles with an initial radius of more than 10 microns do not collapse but execute complex oscillations. The oscillation process for bubbles with an initial radius greater than 170 microns is not cavitational.

4. The value of cavitation microexplosion energy is usually insufficient for destruction of particles characterized by low defect rate. The efficiency of high-dispersive powders disaggregation can be improved by using lowered hydrostatic pressure exposure.

5. To realize the full potential inherent in the fine powders one should make the search for new combined methods of dispersion, for example, based on the use of ultrasound in combination with surfactants.

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Earned value management in project time control

Метод управления освоенным объёмом в контроле сроков проекта

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Key words: method of management; earned value; project time control; planned value; actual cost; earned value management

Ключевые слова: метод управления; освоенный объём; контроль сроков проекта; плановый объём; фактическая стоимость; метод управления освоенным объёмом

Abstract. The paper is devoted to earned value management, it describes the basic tools of this method, particular attention is directed to the indicators of the deviation of the time of the project. Also reasons for not using the method in practice are revealed. The main problem of the method of earned value management – the impossibility of its application to project time control, analyzed ways to solve this problem, proposed by W. Lipke, were identified. As an example, a project consisting of 8 works was reviewed. It has been reviewed three cases of possible cost sharing between critical and non-critical activities. It is revealed that the method proposed by Lipke, is not suitable for the project, where the cost of critical activities is low. The conclusions are about the possibility of using the method for the project time control.

Аннотация. Статья посвящена методу управления освоенным объёмом, в ней описаны основные инструменты данного метода, особое внимание направлено на показатели отклонения сроков проекта. Также выявлены причины отказа от использования метода на практике. Выявлена основная проблема метода управления освоенным объёмом – невозможность его применения для контроля сроков проекта, проанализированы пути решения этой проблемы, предложенные У. Липке. В качестве примера был рассмотрен проект, состоящий из 8 работ. Было проанализировано три случая возможного разделения затрат между критическими и некритическими работами. Выявлено, что метод, предложенный У. Липке, не подходит для проектов, где стоимость критических работ невелика. Выводы содержат информацию о возможности использования метода для контроля сроков проекта.

Introduction

Current practice of the implementation of construction projects requires giving attention to the timeliness of work completion and commissioning of the object [1–3]. There are a number of projects for which the prevention of disruption of timing is very important, such as the Olympic objects in Sochi was necessary to complete and put in commission before the Olympic Games, football stadiums needed to build before the World Cup in 2018, etc [1–5]. Commissioning of these objects into operation later approved date (deadline) not only leads to a drastic reduction in the efficiency of the project, but often defeats the purpose of the implementation of the projects and can lead to the collapse of the program, which includes the project [6–9].

Therefore, when working with important projects, it is necessary to pay attention not only to the preparation and optimization of the calendar plan of the building, but also the formation of an effective system of monitoring, control and management of the project [10–15].

Practice of construction projects shows that the time of the completion of some of them can not be failure and timeliness of their implementation is an important management problem. However, some of the building projects realized in the "background" and the time of their completion is not important for project participants or the main criterion for the success of the project is to minimize the costs to the

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detriment of the duration of the project [12–15]. But in this paper such projects are not considered. The main objects of study are the projects that have disruption of the schedule leads to serious catastrophic consequences.

However, joint control of the two most important parameters (time and cost) can be realized on the basis of the method of Earned Value Management. The basic principles of this method have been known and used in the industrialized countries in the late 1800s. In the early 1960s the US Air Force for the first time officially used the concept of Earned Value in the design and implementation of a unique project “Minute Man Missile”. Due to the success of the application of this method has been introduced in the practice of project management and was legislated in the USA as mandatory for the implementation of projects [16].

Currently, there is little need to use the method of Earned Value Management both Russian and foreign practice in project management [17]. These experts in the field of application of the method of Earned Value Management as a K. Flemming and D. Koppelman give, in their opinion, the reasons for non-use of the method [17]:

- Method of Earned Value Management is not always and not for all is clear and can be difficult at first acquaintance with him.
- Method of Earned Value Management originally was to be used only for large systems and projects, such in the USA only 1 % of the projects using this method, and, in general, these projects involve the major systems of government.
- Method of Earned Value Management describes the current state of the project in some detail (which is not always satisfactory), and the management does not want to know all the negative information [17].

Due to state of this method in Russia it conducted a survey among experts project management sphere. Respondents should have been to give reasons for not using the method. The leader of the responses was the reason associated with the complexity of the link accounting systems and calendar planning. Also, the following reasons have been put forward: a lack of information on the practical application of the method of Earned Value Management and the absence of good reasons to use this method in the control of the project [16, 17].

However, the main reason for not using the method of Earned Value Management in practice is the impossibility of direct application to time control of the project [18]. Customer, first of all, interested in the project to be completed on schedule, that is, key information is how many days the execution of works is behind schedule (or ahead of him), and by using this method it is possible to determine the backlog in duration only in monetary terms. Therefore it is required to develop practical methods and algorithms for the application of this method in the time control [18, 19].

Methods

The method of Earned Value Management controls two important parameters of the project. These parameters are time and cost. Forecast of final duration and cost of the project can be obtained on the identified trends [18].

The main indicators of the method of Earned Value Management are [18, 19]:

- AC – Actual Cost.
- PV – Planned Value.
- EV – Earned Value.

In Figure 1 you can see a graphic image of the method of Earned Value Management. The planned value graphically represents the S-curve. It is a curve of planned distribution of cost (value) within the schedule from the beginning to the end of the work (project) [18, 19]. AC curve shows the actual money spent of the execution of the work (project) from the beginning to the actual date. The third curve EV shows money that has to be spent if the actually performed value of work (the project) to pay according to the plan [18, 19].

The graph shows the deviation of the time, cost and duration. The deviation of the curve AC from PV has cost variance (CV), determined by the formula:

$$CV=EV-AC, \tag{1}$$

where CV – cost variance; EV – budgeted cost (earned value); AC – the actual cost.

If CV > 0, there is a budget saving, and if CV < 0 – cost overruns [20].

The deviation of the curve EV from PV has Schedule Variance (SV), but it is expressed in monetary units, that is, the difference between the cost of the executed and planned works, determined by the formula:

$$SV=EV-PV \quad (2)$$

where SV – schedule variance; EV – earned value; PV – planned value [20].

If SV > 0 – the works are ahead of plan, and if SV < 0 – the works are lagging behind [20].

These two deviations can be calculated using the formulas and seen graphically. But the deviation of the duration (TV – Time Variance) is determined only graphically (Fig. 1) [20, 21].

Also relative indicators are introduced. Cost Performance Index is calculated as the ratio of the cost of executed works (EV) to actual cost (AC) and Schedule Performance Index is calculated as the ratio of earned value to the value which had to be done according to the schedule. If the CPI > 1, there is a budget savings, and if the CPI < 1 – cost overruns. Similarly, if the SPI > 1 – the works are ahead of schedule, and if the SPI < 1 – the works are lagging behind [18–21].

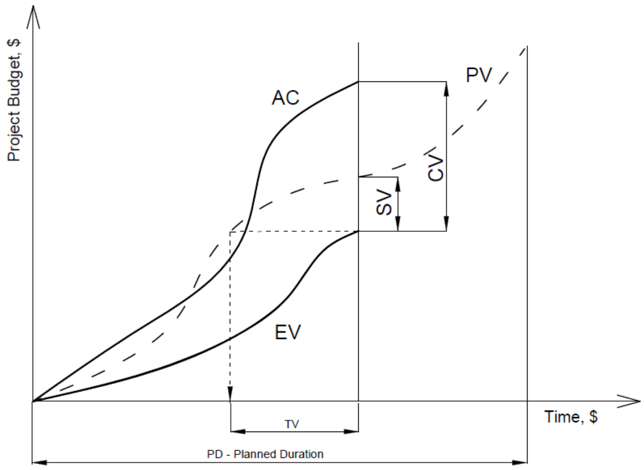

Figure 1. Graphical representation of Earned Value Management method

Also the Time Estimate at Completion (TEAC) may be determined based on the deviation in duration. Also the graph shows the planned parameters of the project: the planned budget of the project named Budget at Completion (BAC) and the planned duration of the project named Schedule at Completion (SAC) [19–22].

Results and Discussion

Compliance with deadlines is important in the implementation of construction projects. For some objects untimely completion of work could mean that the failure of the project. The method of Earned Value Management used for cost control can not be directly used to time control of the project. By this method to determine the backlog of the schedule can be graphically and in terms of money, but for the customer it is important to know that the backlog in days. The transition from the cost parameters to the time parameters is impossible because they have a fundamental difference: the final budget for the completion of the project consists of the cost of all work, and the project implementation time is determined by the critical path length, rather than the sum of the execution time of all the work [18–22]. Also, the time deviation indicators SV and SPI at the completion of the project with a delay do not indicate it [18–23].

This problem of the Earned Value Management method has been considered in the works of Walt Lipke, who developed the "Earned Schedule Concept", using time parameters [18, 19]. The essence of this method is to find the time for which the EV will be equal to PV [18, 19]. The indicator ES is entered and named Earned Schedule. It can be determined graphically (Fig. 2) and algebraically by the formula (3):

$$ES=C+I, \quad (3)$$

where ES – time for which the EV will be equal to PV; C – the number of time periods for which the EV ≥ PV; I – an increment, which is determined by linear interpolation of the period C + 1 by the formula (4):

$$I = {(EV-PV_c)}\Big/{(PV_{c+1}-PV_c)}, \qquad (4)$$

where I – the increment; EV – earned value; PV_c; PV_{c+1} – the planned values on the borders of the time period C + 1.

Then we can define indicators of the time deviation SV (t) and SPI (t), which will be determined by the formulas (5) and (6), respectively [18–21]:

$$SV(t)=ES-AT \qquad (5)$$

where SV (t) – the time deviation, months .; ES – time for which the EV will be equal to the PV, months; AT – actual time months.

$$SPI(t)={ES}\big/{AT} \qquad (6)$$

where SPI (t) – the Schedule Performance Index; ES – time for which the EV will be equal to the PV, months; AT – actual time months.

If ES ahead ET, the SV (t) – is positive, and If ES is behind AT, the SV (t) – is negative. SPI (t) > 1, when ES exceeds AT, and SPI (t) < 1, when ES is smaller AT [18–21].

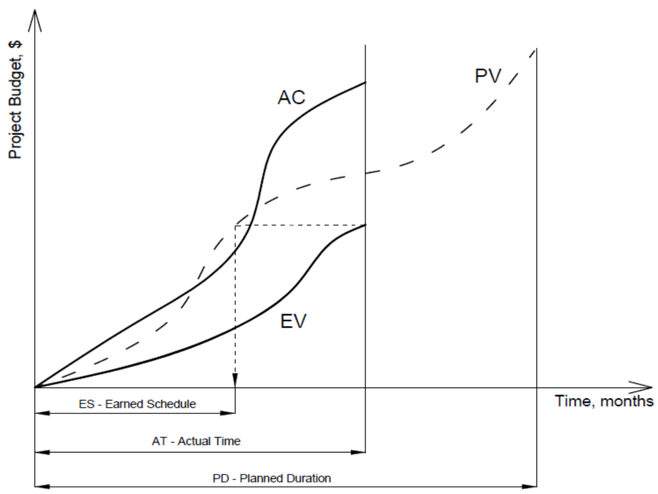

Figure 2. Graphical representation of the Earned Schedule Concept [23–25]

Let us try to test the theory of Walt Lipke by the example. Consider a project consisting of 8 works. (Fig. 3).

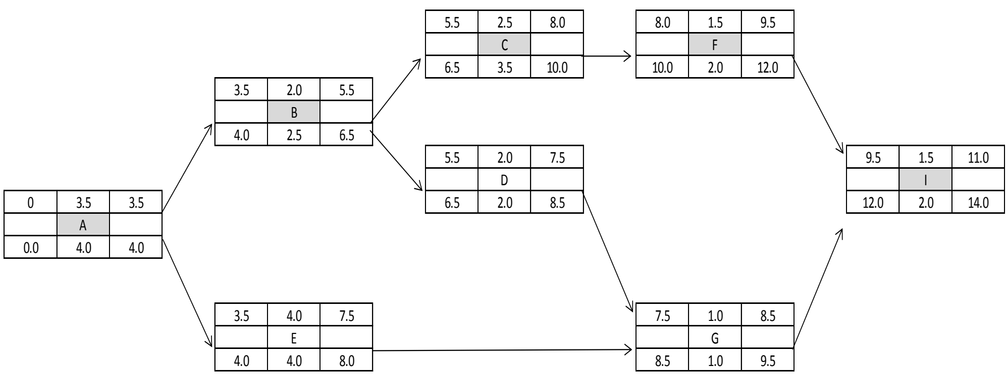

Figure 3. Project for an example

Parameters of the project work are shown in a Table 1. The project planned duration is 11 days, actual duration is 14 days, the critical path formed by the work A, B, C, F, I.

Table 1. Parameters of the project work

Work	Duration	Start	Completion
A	3.5	0	3.5
B	2	3.5	5.5
C	2.5	5.5	8
D	2	5.5	7.5
E	4	3.5	7.5
F	1.5	8	9.5
G	1	7.5	8.5
I	1.5	9.6	11

Consider three cases: 1 case where the critical activities constitute the main cost of the project (that is, critical activities are much more expensive than non-critical), 2 case where the value of the critical activities is a small part of the total cost (that is, the critical works are cheap compared to non-critical) and 3 case – the cost of critical and non-critical activities about the same [23–27].

In all three cases, the planned cost of the project is 1 million, all non-critical activities are done in a timely, but the timing of non-critical activities is failure. For each of the cases the actual date is 7 days. The main task is to calculate the time deviation SV (t), the Schedule Performance Index SPI (t) and forecast the timing of completion of project.

Case 1. The critical activities constitute the main cost of the project

Table 2. The distribution of PV by status dates for case 1

Work	Duration	PV	Days						
			1	2	3	4	5	6	7
A	3.5	100	28.57	57.14	85.71	100	100	100	100
B	2	300	0	0	0	75	225	300	300
C	2.5	250	0	0	0	0	0	50	150
D	2	30	0	0	0	0	0	7.5	22.5
E	4	40	0	0	0	5	15	25	35
F	1.5	150	0	0	0	0	0	0	0
G	1	30	0	0	0	0	0	0	0
I	1.5	100	0	0	0	0	0	0	0
Amount		1000	28.57	57.14	85.71	180	340	482.5	607.5

Table 3. The distribution of EV by status dates for case 1

Work	Duration	EV	Days						
			1	2	3	4	5	6	7
A	4	100	25	50	75	100	100	100	100
B	2.5	300	0	0	0	0	120	240	300
C	3.5	250	0	0	0	0	0	0	35.725
D	2	30	0	0	0	0	0	0	7,5
E	4	40	0	0	0	0	10	20	30
F	2	150	0	0	0	0	0	0	0
G	1	30	0	0	0	0	0	0	0
I	2	100	0	0	0	0	0	0	0
Amount		1000	25	50	75	100	230	360	473.225

Table 4. The forecasting of the completion date project on the basis of EVM indicators for case 1

AT	C	I	ES	SV	SV(t)	SPI	SPI(t)	IEAC(t)
1	0	0.88	0.88	-3.57	-0.12	0.88	0.88	12.57
2	1	0.75	1.75	-7.14	-0.25	0.88	0.88	12.57
3	2	0.63	2.63	-10.71	-0.37	0.88	0.88	12.57
4	3	0.15	3.15	-80.00	-0.85	0.79	0.56	19.80
5	4	0.31	4.31	-110.00	-0.69	0.86	0.68	16.26
6	5	0.14	5.14	-122.50	-0.86	0.86	0.75	14.74
7	5	0.50	5.50	-134.28	-1.50	0.79	0.78	14.12

After analyzing the indicators, we see that the forecast is close to the actual duration. This means that the method of “Earned Schedule Concept” can be applied in this case.

Case 2. The cost of the critical activities is a small part of the total cost

Table 5. The distribution of PV by status dates for case 2

Work	Duration	PV	Days						
			1	2	3	4	5	6	7
A	3.5	15	4.2855	8.571	12.8565	15	15	15	15
B	2	0	0	0	0	0	0	0	0
C	2.5	0	0	0	0	0	0	0	0
D	2	350	0	0	0	0	0	87.5	262.5
E	4	300	0	0	0	37.5	112.5	187.5	262.5
F	1.5	5	0	0	0	0	0	0	0
G	1	320	0	0	0	0	0	0	0
I	1.5	10	0	0	0	0	0	0	0
Amount		1000	4.2855	8.571	12.8565	52.5	127.5	290	540

Table 6. The distribution of EV by status dates for case 2

Work	Duration	EV	Days						
			1	2	3	4	5	6	7
A	4	15	3.75	7.5	11.25	15	15	15	15
B	2.5	0	0	0	0	0	0	0	0
C	3.5	0	0	0	0	0	0	0	0
D	2	350	0	0	0	0	0	0	87.5
E	4	300	0	0	0	0	75	150	225
F	2	5	0	0	0	0	0	0	0
G	1	320	0	0	0	0	0	0	0
I	2	10	0	0	0	0	0	0	0
Amount		1000	3.75	7.5	11.25	15	90	165	327.5

Table 7. The forecasting of the completion date project on the basis of EVM indicators for case 2

AT	C	I	ES	SV	SV(t)	SPI	SPI(t)	IEAC(t)
1	0	0.88	0.88	-0.54	-0.12	0.88	0.88	12.57
2	1	0.75	1.75	-1.07	-0.25	0.88	0.88	12.57
3	2	0.63	2.63	-1.61	-0.37	0.88	0.88	12.57
4	3	0,05	3.05	-37.50	-0.95	0.76	0.29	38.50
5	4	0.50	4.50	-37.50	-0.50	0.90	0.71	15.58
6	5	0.23	5.23	-125.00	-0.77	0.87	0.57	19.33
7	6	0.15	6.15	-212.50	-0.85	0.88	0.61	18.14

As a result of the calculation obtained that the forecast exceeds the actual duration of 4 days that is a false result. This means that the method of “Earned Schedule Concept” can not be applied in this case [26, 27].

Case 3. The cost of critical and non-critical activities about the same

Table 8. The distribution of PV by status dates for case 3

Work	Duration	PV	Days						
			1	2	3	4	5	6	7
A	3.5	125	35.7125	71.425	107.1375	125	125	125	125
B	2	125	0	0	0	31.25	93.75	125	125
C	2.5	125	0	0	0	0	0	25	75
D	2	125	0	0	0	0	0	31.25	93.75
E	4	130	0	0	0	16.25	48.75	81.25	113.75
F	1.5	125	0	0	0	0	0	0	0
G	1	125	0	0	0	0	0	0	0
I	1.5	120	0	0	0	0	0	0	0
Amount		1000	35.7125	71.425	107.1375	172.5	267.5	387.5	532.5

Table 9. The distribution of EV by status dates for case 3

Work	Duration	EV	Days						
			1	2	3	4	5	6	7
A	4	125	31.25	62.5	93.75	125	125	125	125
B	2.5	125	0	0	0	0	50	100	125
C	3.5	125	0	0	0	0	0	0	17.8625
D	2	125	0	0	0	0	0	0	31.25
E	4	130	0	0	0	0	32.5	65	97.5
F	2	125	0	0	0	0	0	0	0
G	1	125	0	0	0	0	0	0	0
I	2	120	0	0	0	0	0	0	0
Amount		1000	31.25	62.5	93.75	125	207.5	290	396.6125

Table 10. The forecasting of the completion date project on the basis of EVM indicators for case 3

AT	C	I	ES	SV	SV(t)	SPI	SPI(t)	IEAC(t)
1	0	0.88	0.88	-4.46	-0.12	0.88	0.88	12.57
2	1	0.75	1.75	-8.93	-0.25	0.88	0.88	12.57
3	2	0.63	2.63	-13.39	-0.37	0.88	0.88	12.57
4	3	0.27	3.27	-47.50	-0.73	0.82	0.72	15.18
5	4	0.37	4.37	-60.00	-0.63	0.87	0.78	14.18
6	5	0.19	5.19	-97.50	-0.81	0.86	0.75	14.70
7	6	0.06	6.06	-135.89	-0.94	0.87	0.74	14.77

We see that the forecast is close to the actual duration. This is an acceptable result. This means that the method of “Earned Schedule Concept” can be applied in this case [26, 27].

Conclusions

Earned Value Management method used to control the cost of the project work, but it is equally important to control the time of the work execution. But for time control this method can not be applied directly. This is because the time control should be realized using time parameters, and Earned Value Management method uses cost parameters. This problem is trying to decide by applying the “Earned Schedule Concept” developed by Walt Lipke. But if critical works are inexpensive, this method can give a false result. As a conclusion, we can say that require the development of practical algorithms and methods of using this method to time control of the project.

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The performance of composites from vegetable raw materials with changes in temperature and humidity

Показатели композитов из растительного сырья при изменениях температуры и влажности

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Key words: strength; static bending; cyclic test; composite board materials

Ключевые слова: прочность; статический изгиб; циклические испытания; композиционные плитные материалы

Abstract. The composite board materials with woodchip fillers and nonprocessable flax and cotton processing waste based on the thermosetting binder matrix can be used as construction material for thermal insulation purposes. Research methods of temperature and humidity change influence on adhesive-bonded board materials' qualities are reviewed. The results of strength and loss of mass behaviour of board materials after the cyclic tests of soaking/freezing/thawing/drying are discribed. Board of unrecoverable waste spinning flax and cotton have the remaining strength is more than 60 % after ten cycles. The experimental data confirm the existence of long-term resistance of the composites to variable temperature and humidity effects.

Аннотация. Композиционные плитные материалы с наполнителями из древесной стружки и невозвратных отходов переработки льна и хлопка на основе матрицы из термореактивного связующего могут использоваться в качестве строительного материала теплоизоляционного назначения. Рассмотрено влияние температуры и влажности окружающей среды на показатели композиционных плитных материалов. Приведены результаты динамики прочности и потери массы плитных материалов после циклических испытаний «замачивание – замораживание – оттаивание – сушка». Волокнистые плиты из невозвратных отходов прядения льна и хлопка имеют остаточную прочность более 60 % после десяти циклов испытаний. Экспериментальные данные подтверждают наличие длительной стойкости композитов к переменным температурно-влажностным воздействиям.

Introduction

When using board products for construction purposes, the stability of their quality characteristics is of importance that determines their long-term performance. One of the major signs of material aging is strength reduction. The environment with variable temperatures and humidity has an attenuation effect on the strength of the construction materials.

Stresses arise in the adhesive material and the filler due to changes in temperature and humidity in the composite products with vegetable fillers based on the matrix from thermosetting binders. A parallel flow of further structuring processes is quite possible thus resulting in an increase in the stiffness of glued joints and stress concentrations as well as polymer's hydrolytic degradation.

Humidity stresses that develop in the adhesive layer of the glue joint will affect the polymer and the glued material in a different way. The stresses that pass from the glue contact area will emerge on the border of the rigid cross-linked polymer and vegetable filler with a higher relaxation action. The magnitude of the stresses also will be influenced by the vegetable filler's structure (the wooden one or

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from annual plant's waste) i.e. fractional size, porosity, availability of mineral, extraneous or fat and wax substances.

Under the operating conditions, the construction materials including three-layered structures (building boards, etc) [1] are affected by variable factors such as heating, humidification, freezing, etc. According to B.S. Batalin, changes in the operational properties of thermal insulation materials "may occur due to ongoing photo-oxidative and thermo-oxidative processes, which result in changes in the molecular mass and molecular-mass distribution. Apart from that, the reason for the changes in the operational properties may become structural changes that occur over time under the influence of a relatively low temperature. The successful use of any polymeric material under different conditions depends on its ability to preserve its performance characteristics, i.e. on its long service life [2]. Therefore, in order to predict the aging of materials during operation, they use field tests or accelerated cyclic exposure methods.

The field test methods are more informative and precise as for the practical application [3–10]. There exist grapho-analytical methods for comparing field tests in the atmosphere with control laboratory samples of the materials that greatly reduce labour intensiveness and time of study [11–13]. According to the data provided by A.S. Freydin, the use of grapho-analitical methods in predicting the use of these materials in construction for a period of thirty years (10^9 c) will require conducting field tests for 5 10^7 c (i.e. 1.5 years.) However, the calculation analogue methods [14] used for the predicting are theoretically substantiated for the materials in rubberlike state, and to a lesser degree, are applicable to the board materials on thermosetting matrix. It is difficult to use the methods of random/degradation functions to predict the strength of composites from vegetable raw materials based on thermosetting binders that adequately describe the systems from wood chips and mineral binders (such as cement bonded particle boards.) [1, 15–17].

It should be noted that the complexity and duration of full-scale testing reduces the efficiency of decision-making on the choice of technological impact on the material. The strength reduction of materials during operation will be a consequence of residual technological, temperature or humidity stresses as well as of a more intensive influence of alternating loads.

It has been proven that the composite components react differently to variations in temperature and humidity. The lignocellulosic filler will remain rather stable in these conditions for a long time. According to the data provided by E. Pokrovskaya, "the infrared spectra of the wood samples display no chemical changes in the xyloid substance in the course of time. There is only a quantity change of the components; the lignin-carbohydrate complex in the conditions of variable humidity … displays stability in the course of time" [18].

Thus, J. Zhu notes that the sensitivity of lignocellulosic materials to humidity poses a problem as before, whereas the properties of composite materials made of flax fibers are more dependent on the type of binder (thermoplastic, thermosetting materials or biomaterials) [19].

The use of flax and cotton fibers as fillers ensures high strength characteristics of composite materials, where the strength of the composite is inversely proportional to the share of lignocellulosic filler [20, 21].

The strength of composite with vegetable filler is determined by the strength of the thermosetting binder matrix.

According to the data provided by V.M.Khrulev, the reduction in glued joint strength is directly proportional to the speed of swelling [22].

If the material is exposed to alternating impacts, the relaxation efficiency of the glued joint is weakened, at the same time the adhesive bonds will be the weakest [23, 11].

The low temperature can particularly affect the relaxation capacity of glued joints. If a temperature is -8 °C or still lower, the ice is formed in the pores of the vegetable raw material-based composites including the wood. The material rigidity will sharply increase, and in the joints of two dissimilar materials (polymer and vegetable filler) that have a different thermal expansion coefficient, thermal stresses will occur. At a low temperature, densely cross-linked polymers behave as brittle bodies; they can even be destroyed by a relatively small deformation [20]. Because of a short soaking – freezing – thawing – drying period, the stresses do not have time to relax, a devastating impact on the material increases as compared to the natural conditions. In consideration of the foregoing, the accelerated cyclic tests will be more preferable.

Susoeva I.V., Vahnina T.N., Titunin A.A., Asatkina J.A. The performance of composites from vegetable raw materials with changes in temperature and humidity. *Magazine of Civil Engineering*. 2017. No. 3. Pp. 39–50. doi: 10.18720/MCE.71.5.

V.M. Khrulev [25–27] made a considerable contribution into the concept of the structure formation of wooden composite materials used for construction purposes. He proposed a cyclic test mode to assess the weather resistance of laminated wood and board materials where the samples should be kept in water for five hours and then are subject to drying at 70 °C within twenty-four hours. For the progress of his own and his colleagues' research, they developed Russian State Standard GOST 17580 "Wooden laminated structures. Method to determinate stability of glued joints against cyclic temperature-and humidity influences" (the wording of 1972). This method is applicable to the assessment of structural construction adhesives [14]. In [28] comprises testing of materials in boiling water. Later, the exposure to temperature changes and environmental humidity factors were modeled in the form of cyclic tests in standard procedures [29–32].

Scientists performed research on the influence of factors on physical-mechanical properties of materials using self-developed [33, 35, 36, 38–43] and the standard [34, 37] procedures. According to H.D. Dibaba, the cyclic test helps to evaluate the deformation, fatigue and creep properties of a material in polymers and polymer-composites [39].

It should be noted that thermal insulating materials serving as engineering structure elements are also exposed to temperature and humidity changes. However, they must retain their performance characteristics with temperature and humidity changes due to weather conditions in relation to the conditions of use.

For insulation panels, unlike the structural water resistant ones, the strength and swelling in thickness after the cyclic tests is not specified. In practice, any change in temperature corresponds to a humidity change of the environment air. But for all that, as a result of ongoing sorption-desorption processes of water vapour, the low density material having some open pores periodically absorbs and produces moisture. This phenomenon significantly affects the vapour permeability of heat insulation materials [44]. These processes entail the development of temperature/humidity stresses. If the stresses do not have time to decompress, and their intensity is higher than the adhesion bond strength, it will result in reduction of the composite strength and in deterioration of shape stability and thermal properties.

Accordingly, in the course of work, the decision has been made to use a standard cyclic test method of the material [29]. The results of cyclic tests will permit to assess the resistance of the material to temperature and humidity changes while in the course of service.

Experimental Part

In the laboratory of forest harvesting/woodworking department of the Kostroma State University (Kostroma, Russia), some chipboards and composite fibre board based on fillers from nonprocessable waste of cotton and flax fibre production based on the matrix from synthetic and non-organic binders have been tested for their resistance to temperature and humidity effects [45]. As a binder alternative for the boards were chosen: liquid glass $Na_2O(SiO_2)_n$, (module n = 1.6…3.75), phenol-formaldehyde resin and aluminium-chromium-phosphate binder $CrAl_3(H_2PO_4)_n$, (n = 8.8…9.6). The heat insulation composite material had a medium density of 375 kg/m^3, the consumption of binders was 40 % from the mass of filler. The material samples were dried at 80°C to reach 8±1 % humidity.

The cyclic temperature and humidity tests were made under [29]. One cycle of sample temperature/humidity exposure included the following procedures: the immersion of samples into water at 20 °C for twenty hours, freezing of wet samples at -20 °C for six hours, thawing at 20 °C for sixteen hours and heating at 60 °C for six hours.

The results of physical and mechanical performance determination of the materials after each of the ten test cycles are shown in Tables 1–5. Table 6 presents the dependences of residual strength of composite board materials (У) on the number of tests (x). As a rule, the reduction of strength is expressed with a function of varying complexity beginning from a simple exponential [11]. In research, we construct dependences ranging from the exponential to the fourth degree polynomials. Values of approximation validation are shown for each model.

Table 1. Behaviour of chipboard* in cyclic tests

Cycles	Loss of mass, Δm, %	Bending strength, σ_i, MPa	Swelling in thickness, P_s, %	Residual mass, %	Residual strength, %
Prior to tests (control)	- / -	25.5/22.97	9.4/24.5	-	-
1	5.38/2.95	21.09/11.39	10.7/24.8	94.62/97.05	82.5/49.6
2	16.82/10.63	18.27/6.50	12.2/25.7	83.18/89.37	71.5/28.3
3	26.3/ broke	16.02/ broke	12.9/ broke	73.7/-	62.9/0
4	28.07/ -	13.20/ -	14.2/ -	71.93/-	51.6/0
5	36.71/ -	10.47/ -	15.31/ -	63.29/-	41.0/0
6	41.82/ -	7.95/ -	17.91/ -	58.18/-	31.3/0
7	45.98/ -	6.00/ -	19.81/ -	54.02/-	23.5/0
8	46.25/-	4.72/-	22.16/-	53.75/-	18.5/0
9	46.87/-	4.28/-	25.38/-	53.13/-	16.8/0
10	47.14/-	3.51/-	28.56/-	52.86/-	13.7/0

***** Above the line are the values for chipboard with phenol-formaldehyde binder; under the line – with urea-formaldehyde binder (carbamide-formaldehyde resin)

Table 2. Behaviour of fibre boards** with phenol-formaldehyde resin in cyclic tests

Cycles	Loss of mass, Δm, %	Bending strength, σ_i, MPa	Swelling in thickness, P_s, %	Residual mass, %	Residual strength, %
Prior to tests (control)	- / -	0.49/0.55	3.36/8.2	-	-
1	7.42/8.61	0.46/0.53	1.82/4.32	92.58/91.39	0.94/0.96
2	22.11/26.21	0.46/0.53	1.96/4.88	77.89/73.79	0.94/0.96
3	36.82/41.63	0.44/0.52	2.07/5.27	63.18/58.37	0.89/0.94
4	38.64/44.52	0.43/0.51	2.23/5.58	61.36/55.48	0.87/0.93
5	51.01/59.45	0.41/0.49	2.31/5.93	48.99/40.55	0.83/0.89
6	58.48/66.04	0.41/0.47	2.38/6.84	41.52/33.96	0.83/0.85
7	64.37/72.44	0.40/0.47	2.91/7.32	35.63/27.56	0.81/0.85
8	64.75/73.08	0.38/0.41	3.22/8.30	35.25/26.92	0.77/0.74
9	65.42/73.47	0.31/0.32	3.48/8.76	34.58/26.53	0.63/0.58
10	65.61/74.59	0.15/0.26	3.62/9.29	34.39/25.41	0.31/0.47

****** Above the line for boards from cotton; under the line for boards from flax.

Table 3. Behaviour of fibre boards with carbamide-formaldehyde resin in cyclic tests***

Cycles	Loss of mass, Δm, %	Bending strength, σ_i, MPa	Swelling in thickness, P_s, %	Residual mass, %	Residual strength, %
Prior to tests (control)	- / -	0.38/0.49	3.26/6.9	-	-
1	7.56/8.79	0.37/0.47	1.91/3.7	92.44/91.21	0.97/0.96
2	18.9/21.97	0.37/0.47	2.44/4.25	81.1/78.03	0.97/0.96
3	35.91/41.74	0.35/0.47	2.71/4.63	64.1/58.26	0.92/0.96
4	42.54/49.63	0.32/0.46	2.94/4.89	57.46/50.37	0.84/0.94
5	51.26/58.34	0.27/0.43	3.08/5.16	48.74/41.66	0.71/0.88
6	58.12/65.75	0.22/0.43	3.22/5.54	41.88/34.25	0.58/0.88
7	63.41/71.68	0.22/0.42	3.39/5.91	36.59/28.32	0.58/0.85
8	68.29/76.13	0.20/0.41	3.44/6.68	31.71/23.87	0.52/0.83
9	71.63/78.41	0.18/0.35	3.51/7.03	28.37/21.59	0.47/0.71
10	73.55/80.80	0.11/0.30	3.58/7.61	26.45/19.2	0.29/0.61

******* Above the line for boards from cotton; under the line for boards from flax.

Table 4 Behaviour of fibre boards with liquid glass in cyclic tests***

Cycles	Loss of mass, Δm, %	Bending strength, σ_i, MPa	Swelling in thickness, P_s, %	Residual mass, %	Residual strength, %
Prior to tests (control)	- / -	0.71/0.88	5.68/9.16	-	-
1	7.71/9.12	0.70/0.87	2.98/4.52	92.29/90.88	0.98/0.99
2	16.23/20.54	0.67/0.85	3.41/5.06	83.77/79.46	0.94/0.96
3	31.62/39.67	0.62/0.81	3.92/5.71	68.38/60.33	0.87/0.92
4	40.24/43.75	0.60/0.77	4.15/6.34	59.76/56.25	0.84/0.87
5	49.13/51.62	0.57/0.76	4.73/7.02	50.87/48.38	0.80/0.86
6	53.97/56.73	0.55/0.73	5.01/7.69	46.03/43.27	0.77/0.83
7	59.64/62.12	0.51/0.72	5.38/8.21	40.36/37.88	0.72/0.82
8	64.82/65.53	0.49/0.69	5.84/9.05	35.18/34.47	0.69/0.78
9	68.32/69.41	0.44/0.48	6.04/10.11	31.68/30.59	0.62/0.54
10	70.69/70.75	0.38/0.41	6.35/10.78	29.31/29.25	0.53/0.46

******* Above the line for boards from cotton; under the line for boards from flax.

Table 5. Behaviour of fibre boards with aluminium-chromium-phosphate in cyclic tests***

Cycles	Loss of mass, Δm, %	Bending strength, σ_i, MPa	Swelling in thickness, P_s, %	Residual mass, %	Residual strength, %
Prior to tests (control)	- / -	0.69/0.81	3.07/3.72	-	-
1	6.12/ 7.45	0.68/0.79	1.82/1.96	93.88/92.55	0.98/0.97
2	14.28/15.61	0.66/0.78	2.01/2.15	85.72/84.39	0.95/0.96
3	30.42/34.72	0.65/0.76	2.83/2.94	69.58/65.28	0.94/0.94
4	38.16/41.26	0.64/0.75	3.06/3.27	61.84/58.74	0.93/0.92
5	45.17/49.24	0.64/0.73	3.53/3.86	54.83/50.76	0.93/0.90
6	51.83/53.62	0.58/0.69	3.94/4.18	48.17/46.38	0.84/0.85
7	58.27/60.78	0.49/0.61	4.14/4.69	41.73/39.22	0.71/0.75
8	66.71/69.28	0.41/0.52	4.85/5.12	33.29/30.72	0.59/0.64
9	70.29/73.68	0.33/0.46	5.12/6.59	29.71/26.32	0.48/0.57
10	74.22/76.48	0.28/0.39	5.47/7.62	25.78/23.52	0.40/0.48

******* Above the line for boards from cotton; under the line for boards from flax.

Table 6. Dependence of residual strength of composite board materials from the number of test cycles

Composite material	Dependence of boards' residual strength У, % from number of cycles x	Approximation accuracy R^2
Chipboard with phenolformaldehyde resin	$У = 111.28e^{-0.212x}$	0.9899
Fibre board from cotton with phenolformaldehyde resin	$У = 1.1873e^{-0.083x}$	0.5757
	$У = -0.0102x^2 + 0.0603x + 0.8443$	0.8741
	$У = -0.0031x^3 + 0.0412x^2 - 0.1771x + 0.902$	0.9679
Fibre board from flax with phenolformaldehyde resin	$У = 1.1693e^{-0.069x}$	0.7573
	$У = -0.0088x^2 + 0.0461x + 0.8443$	0.9739
	$У = -0.001x^3 + 0.0082x^2 - 0.0323x + 0.9903$	0.9866
Fibre board from cotton with urea formaldehyde binder	$У = 1.2591e^{-0.122x}$	0.9098
	$У = -0.0016x^2 - 0.0584x + 1.0677$	0.9661
	$У = -0.0004x^3 - 0.0082x^2 - 0.028x + 1.0333$	0.9671
Fibre board from flax with urea formaldehyde binder	$У = 1.081e^{-0.044x}$	0.7904
	$У = -0.0055x^2 + 0.0247x + 0.932$	0.961
	$У = -0.0007x^3 + 0.0069x^2 - 0.0323x + 0.9963$	0.9751

Composite material	Dependence of boards' residual strength У, % from number of cycles x	Approximation accuracy R^2
Fibre board from cotton with liquid glass	$У = 1.0699e^{-0.061x}$	0.9542
	$У = - 0.0459x + 1.0287$	0.9812
	$У = - 0.0013x^2 – 0.0318x + 1.0003$	0.9862
	$У = - 0.0008x^3 + 0.0117x^2 – 0.0915x + 1.0677$	0.9969
Fibre board from flax with liquid glass	$У = 1.166e^{-0.073x}$	0.7624
	$У = - 0.0521x + 1.0893$	0.83
	$У = - 0.0069x^2 + 0.0242x + 0.9368$	0.9241
	$У = - 0.0019x^3 + 0.0237x^2 – 0.1173x + 1.0963$	0.9638
Fibre board from cotton with aluminium-chromium-phosphate	$У = 1.2697e^{-0.097x}$	0.8359
	$У = - 0.0667x + 1.142$	0.8783
	$У = - 0.0091x^2 + 0.0329x + 0.9428$	0.9817
	$У = 0.0005x^3 – 0.0166x^2 + 0.0679x + 0.9033$	0.9833
	$У = - 0.0006x^4 – 0.0128x^3 + 0.0807x^2 – 0.1981x + 1.1108$	0.9977
Fibre board from flax with aluminium-chromium-phosphate	$У = 1.182e^{-0.076x}$	0.8627
	$У = - 0.0558x + 1.1047$	0.902
	$У = - 0.0069x^2 + 0.0205x + 0.9522$	0.9912
	$У = 0.0002x^3 – 0.109x^2 + 0.039x + 0.9313$	0.9918

In Figures 1, 2 the dependence of residual strength at a static bending, % (the results with the calculate confidence interval) for plates from the waste of linen and cotton after cycles of temperature and humidity exposure, in Figure 3 – dependence of this indicator for chipboard, tiles made of waste linen and cotton with phenol-formaldehyde binder.

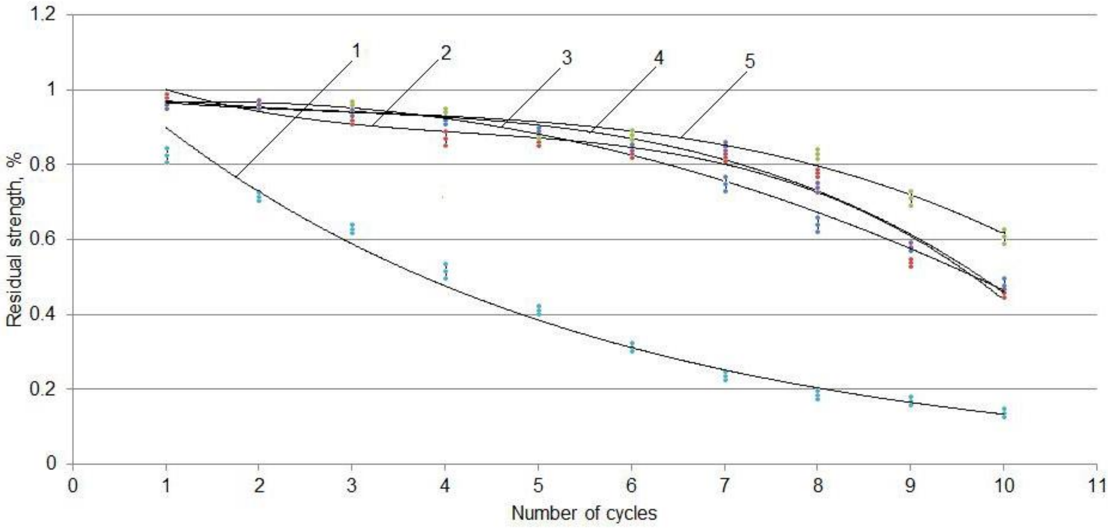


Figure 1. Limit of residual strength for boards from flax waste at static bending, % after cycles to temperature and humidity exposure: 1 – chipboard with phenol-formaldehyde resin; 2 – fibre board from flax with liquid glass; 3 – fibre board from flax with aluminium-chromium-phosphate; 4 – fibre board from flax with phenol-formaldehyde resin; 5 – fibre board from flax with urea-formaldehyde binder (carbamide-formaldehyde resin*).*

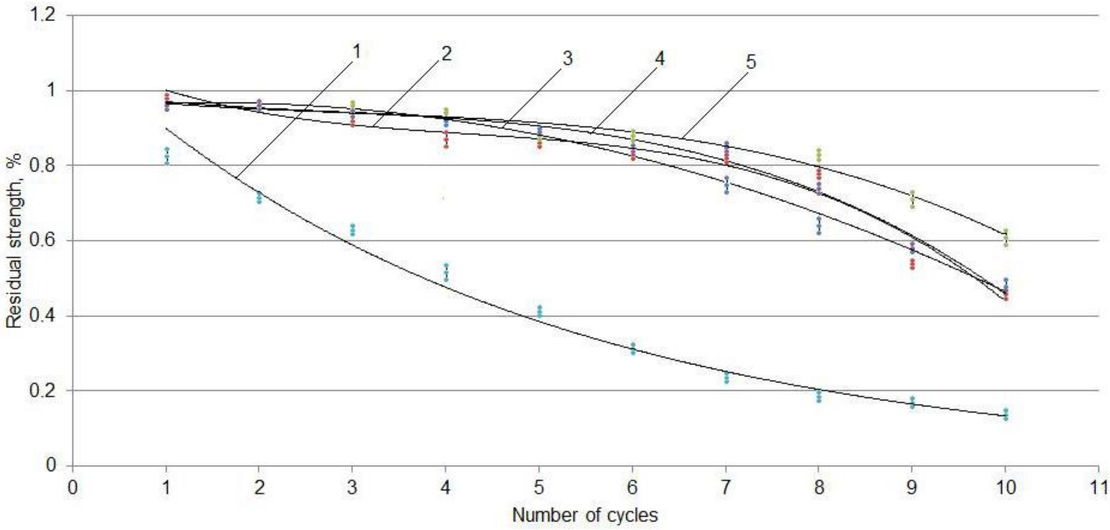


Figure 2. Limit of residual strength for boards from cotton waste at static bending; % after cycles to temperature and humidity exposure: 1 – chipboard with phenol-formaldehyde resin; 2 – fibre board from cotton with phenol-formaldehyde resin; 3 – fibre board from cotton with urea-formaldehyde binder; 4 – fibre board from cotton with liquid glass; 5 – fibre board from cotton with aluminium-chromium-phosphate.

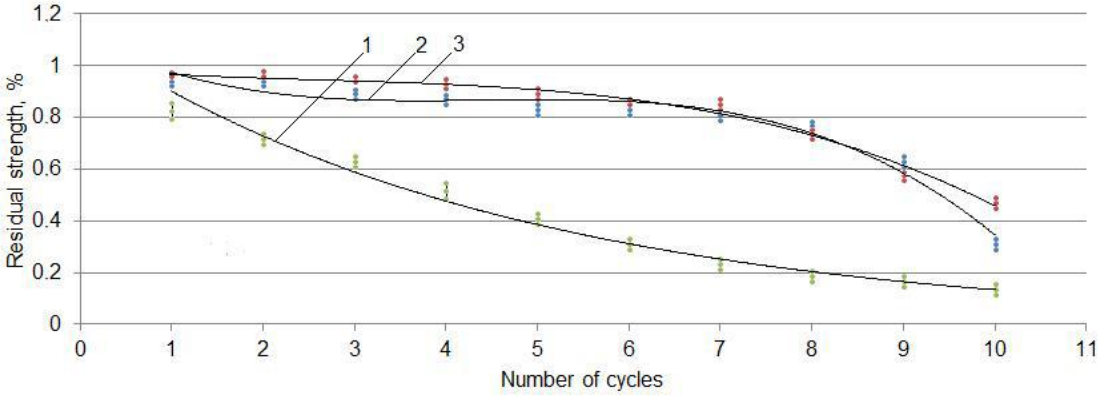


Figure 3. Limit of residual strength for boards at static bending, % after cycles to temperature and humidity exposure: 1 – chipboard with phenol-formaldehyde resin; 2 – fibre board from cotton with phenol-formaldehyde resin; 3 – fibre board from flax with phenol-formaldehyde resin.

Review of Results

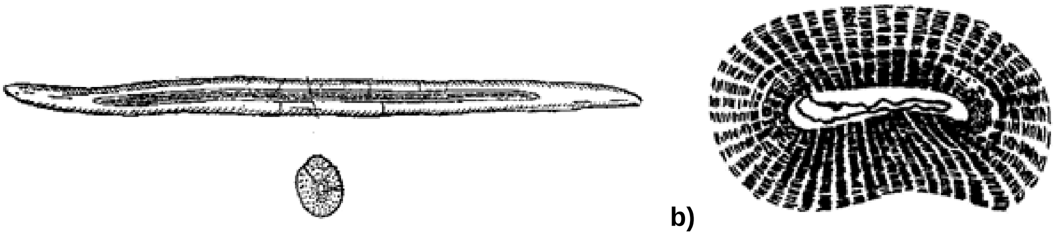
Subsequent to the results of experimental tests, it has been determined that the fibre board has a much higher resistance to cyclic temperature and humidity exposure than the chipboard.

The composites with amino-formaldehyde binder demonstrate the least resistance to temperature and humidity effects assuming that the initial strength at static bending of fibre board can be comparable for all four types of binder. This is comparable with the results of the research A.S. Freidin [11], according to which the composites of solid wood with urea-formaldehyde binder have destructively after six cycles of testing.

A low hydrolytic stability of the amino-formaldehyde binder results in a significant loss of strength of the chipboard immediately after the second test cycle (Table 1); the fibre board from cotton with amino-formaldehyde binder will have a strength under 60 % after the sixth cycle, however, the board with flax binder will have a residual strength above 60 % even after ten test cycles. The conversion of a number of test cycles into real composites' lifetime can be possible after the comparison of strength changes during field and cyclic tests.

The significance of differences is explicitly expressed in the use of one binder and three different fillers (Figure 3). Comparing a composite made by dry method, i.e. chipboard (hot pressing at 180 °C) with fibre composites made by wet method (drying at 80 ºC) when one and the same binder is used (phenol-formaldehyde), it becomes obvious that even incomplete hardening of phenol binder provides the fibre board with a higher dimensional stability (Table 1-3) and a higher residual strength. The reason for these differences lies in the filler's structure. The basic mechanical tissue of wood (libriform and flax fibres) (Figure 4a) have a similar highly elongated spindle shape with closed pointed ends.

However, elementary flax fibre differs from wood chips both in structure and size. The elementary flax fibres have a medium length of 10–24mm; a libriform fibre's length is about 1 mm. Their cross dimension is comparable (11–20 microns.) Apart from the libriform fibre, the wood chips contain water conducting elements, vessels with a diameter about 200 microns whose volume occupies 10…55 % [46]. The wood chips have a higher conducting function than the flax fibre. However, the flax fibre contains twice as much cellulose than wood. The layered fibre wall structure is a consequence of gradual cellulose deposition (with intervals) on the fibre walls



a) b)

Figure 4. Section of fibre: a – longitudinal section of flax fibre; b – cross-section of cotton fibre

A considerably higher orientation of the structural elements relative to the axis in the flax fibre as compared to the cotton fibre, can partially explain a higher strength of flax and its lower capability of elongating due to tension.

The elementary cotton fibres as well as those of flax have a layered structure (Figure 4b) due to a gradual layer-by-layer deposition of cellulose on fibre walls in the form of daily concentric layers. As the fibres ripen, the remainder of the protoplasm in the channel will dry up, and the fibre will get flattened. However, the outer diameter of fibres remains unchanged, and the diameter of channel due to the wall thickening decreases; the strength of the fibres and their elasticity increase, and the sorption properties improve.

It is possible to predict the residual strength of composite boards at static bending by means of regression models (Table 6). All the dependencies have an approximation adequacy of 0.7. An increase in polynomial dependence degree above two results in an increase in the prediction result by 0.06…9.4 % depending on the composite type.

Conclusions

The composites from cotton wastes have a lower cross-breaking strength at static bending than those from flax residue, and a higher loss of strength after the test cycles. This pattern is observed both for liquid phenolic-formaldehyde resin thermosetting binder-based boards, and for the materials based on non-organic binders, e.g. liquid glass and aluminium-chromium-phosphate.

The composites from nonprocessable flax fibre and flax residue display a high stability of the shape after the cyclic tests; a loss of mass over 50 % occurs after 5…6 test cycles. A higher loss of mass here than that of the chipboard can be explained by a removal of the dust fraction (together with water) that do not have chemical or hydrogen bonds in the composite structure where a large number of hydrogen bonds ensures swelling in thickness after ten cycles but not more than 6.4 % for the boards with cotton filler and 10.8 % for the boards with flax residue filler.

The aluminium-chromium-phosphate control samples have a higher strength at static bending as compared to the boards fabricated with phenol-formaldehyde liquid resin and liquid glass. However, after the first test cycles, higher values of residual strength of the composites based on flax and cotton fillers are displayed by the matrices from non-organic binders. A considerable decrease in the residual strength occurs after the fifth cycle for the boards from cotton filler based on amino-formaldehyde binder; for the other composites the strength will be substantially reduced after 8–9 test cycles.

Therefore, it has been proven that the composite board materials made from nonprocessable flax and cotton fibre production residue based on thermosetting and non-organic binders appear to have a high resistance capability to cyclic temperature and humidity influences. The acquired experimental data make it possible to recommend such composite materials for use as heat insulation components of building structures.

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The thermo-stressed state in massive concrete structures

Термонапряженное состояние массивных бетонных конструкций

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Key words: the building period; massive concrete and reinforced concrete structures; cement setting temperature; thermal stressed state; thermal cracking resistance; one-dimensional structural model

Ключевые слова: строительный период; массивные бетонные конструкции; железобетонные конструкции; экзотермия цемента; термонапряженное состояние; одномерная расчетная схема

Abstract. This article examines the justification for using one-dimensional structural models in the analysis of the thermo-stressed state in massive concrete and reinforced concrete structures during the building period. The paper presents calculation results for the thermo-stressed state in massive foundation slabs with different planform dimension ratios. Special attention is paid to presence/absence of thermal insulation on the sides of the slab during the mixture pouring process. Generally, one-dimensional structural model is suitable in cases when ratio h/l < 0.17, h stands for the minimal plane dimension in a construction such as slab. The research indicates the existence of zones near the sides of slabs, where values of the tensile stress exceed the values that were obtained with use of one-dimensional structural model. This excess may account for 9.5 %.

Аннотация. В настоящей работе рассматривается обоснованность использования одномерных расчетных схем при анализе термонапряженного состояния массивных бетонных и железобетонных конструкций зданий и сооружений в строительный период. Приведены результаты расчетного исследования термонапряженного состояния массивной фундаментной плиты с различным соотношением плановых размеров. Особенное внимание уделено наличию/отсутствию теплоизоляции на торцах при укладки бетонной смеси. Определено, что в общем случае одномерная расчетная схема применима при соотношении h/l < 0.17, где h–меньший из плановых размеров конструкции типа плиты. Проведенное исследование показывает наличие зон (в приторцевых участках), в которых растягивающие напряжения несколько превосходят значения, полученные по одномерной задаче. Превышение может составить 9.5 %.

Introduction

Structural calculation methods involve the use of structural models that are made with certain assumptions and simplifications that greatly facilitate the calculation. There is a limit for the use of each structural model after which it becomes invalid. Calculation with an incorrectly chosen structural model cannot be valid, even when using the most accurate methods.

Most of the industrial and civil constructions (especially nuclear power plants [NPP] and high-rise buildings) use large sized reinforced concrete slabs (many times longer than they are thick) for the foundation, for massive walls, and as floor slabs.

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During the building period, non-stationary and non-uniform temperature fields [1-7] appear in the concrete as a result of cement setting and heat exchange with the environment, consequently leading to the formation of thermal stresses [8–16]. Uneven temperature fields are the cause of tensile stresses (and consequently of extension strains) on the surface of the foundation, which are capable of generating dangerous cracking [17–23].

The problem of evaluating the fracture resistance of the concrete and the reinforced concrete blocks during the building period is rather complicated from an engineering point of view [24–30]. Strictly speaking, the foundation (wall, slab) is a 3D object, so the problem should be solved in a 3D formulation. Also, the large dimensions of the massive constructions necessitate the use of a large amount of finite elements in such a design model. The need to consider the influence of temperature on the thermal characteristics and deformability of the concrete (physical nonlinearity of the problem) significantly complicates the calculations. It is also important that the solution of the thermo-elastic problem will be extremely approximate; it is necessary to consider deformations due to concrete creep [31–33].

In the case when the plane dimensions of the slabs considerably exceed their thickness, a one-dimensional structural model can be applied for the central part of the slab with a sufficient degree of accuracy when tension and temperature are functions of a single spatial coordinate – the vertical one [14, 28–31].

The transition to this model is valid for a certain ratio between the thickness of the structure and its plane dimensions. In [10–12] there is an approximate numerical value for this ratio $\frac{h}{l} \leq \frac{1}{3} ... \frac{1}{4}$ where l- the smallest dimension of the block; h- thickness of the array of blocks.

The one-dimensional model has the following advantages:

1. it is a solution to the wide range of problems that arise in modern practice during the erection of massive concrete structures;
2. the method is comparatively simple ;
3. the implementation of the algorithms is simpler and consequently shortens calculations.

There are the disadvantages of this model as well: ratio $\frac{h}{l} \leq \frac{1}{3} ... \frac{1}{4}$ often are not satisfied and heat flow from the side surfaces becomes significant. In such situations, it is necessary to use 2D and 3D structural models.

For the foregoing reasons, finding the precise meaning of ratio $\frac{h}{l}$ is the vital task. Since calculation with an incorrectly chosen structural model cannot be valid, even when using the most accurate methods.

The purposes of this article are an elaboration of the numerical ratio $\frac{h}{l}$, which allows a change to one-dimensional structural model for the calculation of thermal-stressed state of concrete massifs (such as slabs, ceiling panels or walls) during the construction period, and an assessment of observational errors in case of the changing 2D and 3D models to the one-dimensional structural model.

Thermo-stressed state is meticulously surveyed on the ends (2D model) of foundation slabs (walls, ceiling panels), considering the heat sink from the side surfaces. As initial data (thermophysical and stress-related characteristics of concrete, cement heat radiation) the results or research, obtained in laboratory “Polytech-SKiM-Test” in CUBS department by Professor Y.G. Barabanschikov were accepted.

Statement of the problem

To define thermal stresses in a construction a concrete slab was analyzed. This slab does not have restraints in deformations and it is laid as one block and has height in a range of [0.5…2.5] m.

The process of concreting takes place in summer period.

In a slab’s cross-section the problem is two-dimensional, since stresses and temperatures are functions of the two spatial coordinates – vertical and horizontal.

It is required to analyze the thermal stressed state of foundation slab considering that the heat flow is two-dimensional, to assess influence of the heat flow from the ends with different ratio $\frac{h}{l}$, to elaborate the ratio $\frac{h}{l}$ wherein a structural model can be considered as one-dimensional.

This paper demonstrates calculation of the foundation slab's thermal stressed state with the help of TERM software [14] developed by the Institute of Civil Engineering at the St. Petersburg State Polytechnic University. This software calculates nonstationary fields of temperature and thermal stresses in slabs. In order to estimate the cracking resistance of the foundation slab, we would use the deformation criterion suggested by P.I. Vasiliev [24–28]. An essential feature of the TERM software is the consideration of temperature influence on thermophysical and stress-related concrete characteristics, which is vital for problems of the construction period.

The results of analysis of the thermal stressed state of foundation slab allow us to elaborate the $\frac{h}{l}$ ratio, which permits change to one-dimensional structural model. The assessment of the possible measurement errors is provided. The dangerous zones in the ends of the slab are surveyed.

Initial data.

1. Technological conditions of concrete mixture pouring:
a. ambient temperature (temperature of ambient air): 15^0C;
b. concrete mix temperature: 15^0C;

2. Conditions of heat transfer on the surface: third type boundary conditions. Heat transfer according to the Newton's law:

$$\frac{\partial \mathrm{T}}{\partial \mathrm{n}} = \frac{\beta}{\lambda}\left(\mathrm{T_{cp}} - \mathrm{T_{cp}}\right) \tag{1}$$

3. Geometry and plan dimensions: slab thickness equals to 1; 1.5; 2 m;

4. Thermal and physical characteristics of the concrete: thermal conductivity= 2.67 W/m·^{0}C, thermal capacity c= 1.0 kJ/kg·^{0}C;

5. Stress-related characteristics:
a. According to N.A. Malinin, the instantaneous elastic deformation modulus of concrete follows the equation [13]:

$$E(t) = E_{max}\left(1 - e^{\alpha t^{\gamma}}\right), \tag{2}$$

where E_{max} = 38000 MPa is the limit value of the concrete deformation [24]. Functional dependency parameters are $\propto = -0.37$, $\gamma = 0.32$, and t stands for the current time;

b. The heat dissipation process follows the I.D. Zaporozhets equation [2]:

$$\mathrm{Q}(\tau) = \mathrm{Q}_{\max}\left[1 - \left(1 + A_T \tau\right)^{-\frac{1}{m-1}}\right] \tag{3}$$

The parameters of heat dissipation process were defined experimentally. The results of research which was conducted by professor Barabanschiov Y.G. were used as initial data for heat dissipation process: the maximum heat dissipation Q_{max}=1.66×10^5 kJ/м^3; heat dissipation's rate of increase coefficient at 20°C A_{20} = 1×10^5 s^{-1}; 1/(m-1) = 0.833.

For accurate results, sizes of a finite element (in this problem 8-knot isoparametric finite elements are used) were not changed during the calculations. The structural model is represented at Figure 1.

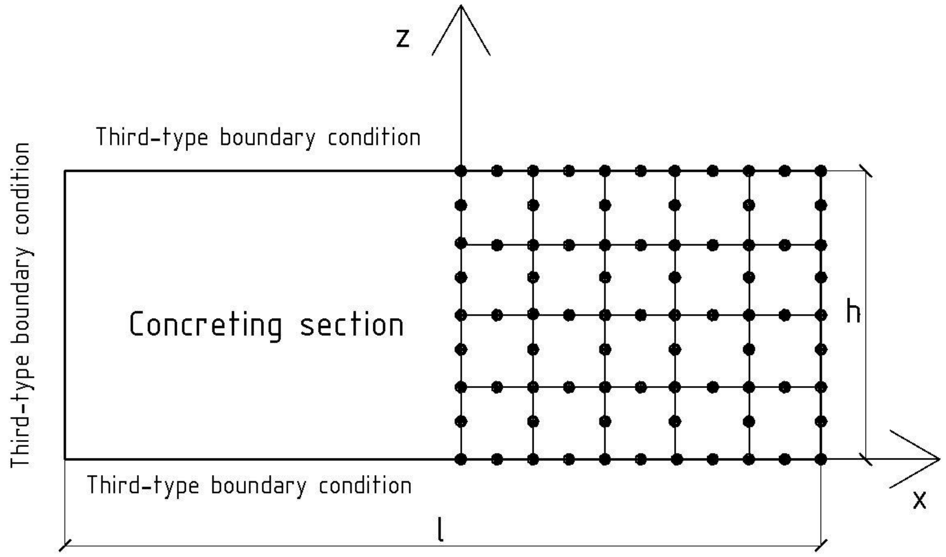

Figure 1. The structural model

Analysis of the thermo-stressed state

First experiment.

The following situation was modeled: for the structural model (Figure 1), the heat transfer from the ends is prohibited by artificial input of infinitesimal reduced heat transfer coefficient βred. The point of time where the maximum of exothermic heat of the plate takes place is explored (first and second day). There is an extension on the surface in this moment, while in the center – compression. Tensile stresses on the surface of the slab are dangerous and may cause crack formation.

As an example, we analyze the thermal stress of the slab, cross section of which is equal to the size 1.5 x 11 m. The temperature fields and thermal stress fields will look (on the 2nd day after laying the mixture) as shown in Figures 2–3.

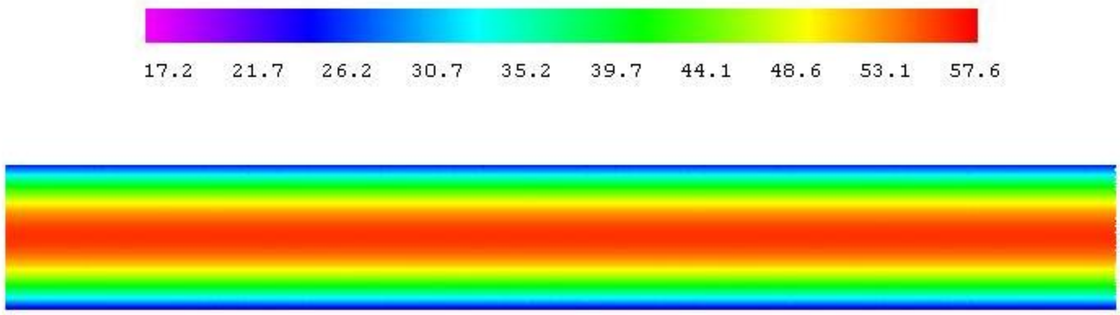

Figure 2. The temperature field on second day after pouring the concrete mixture (0C)

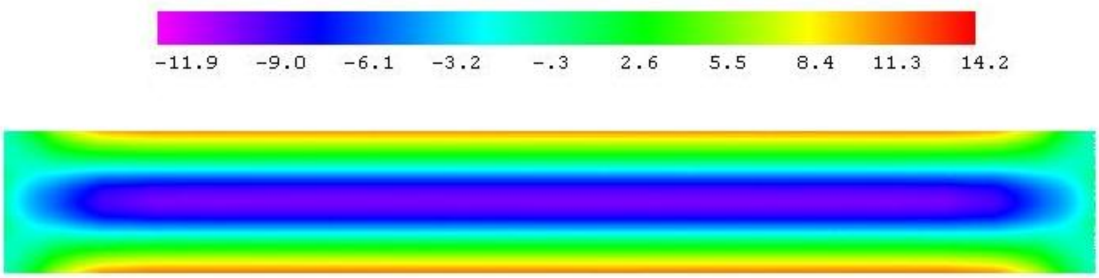

Figure 3. The stress field on second day after pouring the concrete mixture (kg/cm2)

Analysis of temperature fields (Figure 3) shows that temperature field is one-dimensional: the temperature's value varies along the Z-axis (height boards) only. The problem connected with stresses is still two-dimensional.

The diagrams of thermal stress (the X-axis) on the surface of the slab, as shown in Figure 4, decrease monotonically from the center to the sides of the slab. The values of the X-axis on the chart – the number of cross-section of the slab, considering that number 1 is a central cross-section.

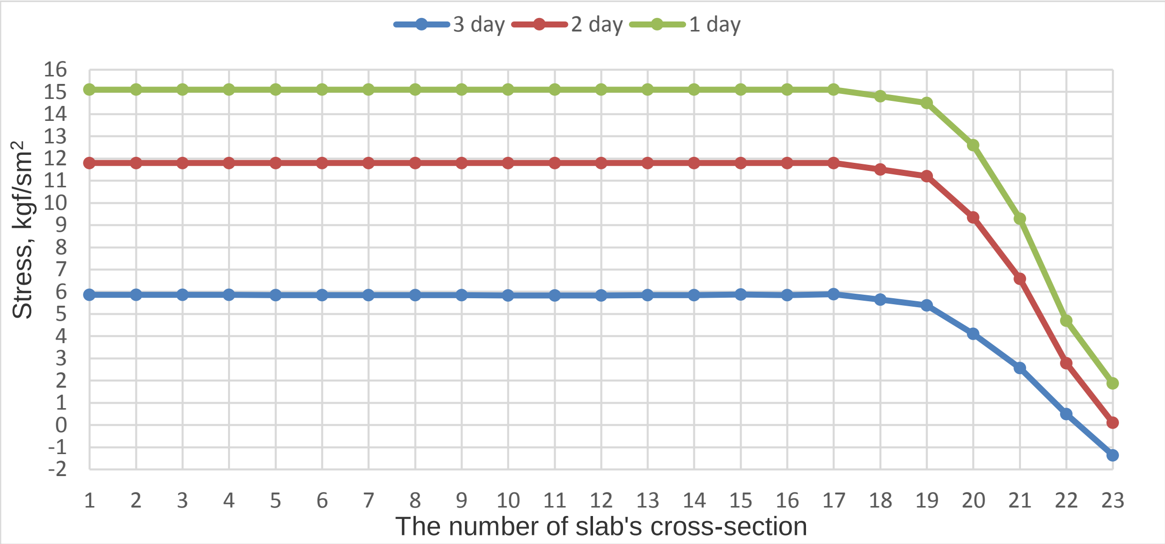

Figure 4. The diagrams of thermal stresses of the surface of the slab

At the same time it is possible to identify two areas inside the cross section (Figure 5) end region A (the amount of such areas - the number of the ends, which equals to 2) and the central region B over the area along the X coordinate are: zone B – 8.370 m, zone A – 2.630 m, or in relative units (relative to the length of the slab), respectively, 76.1 % and 23.9 %.

In the zone B stresses vary are relative only to the Z coordinate and it is suitable to apply the one-dimensional structural model. Obviously, in the zone A we can't obtain a sufficiently accurate solution without using 2D structural model.

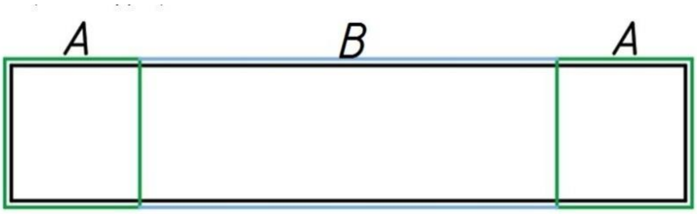

Figure 5. Zones of the slab

Second experiment.

In case of heat flow from the sides of the slab (that is what happens during the real building process), the diagrams of tensile stresses of the surface of the slab are different.

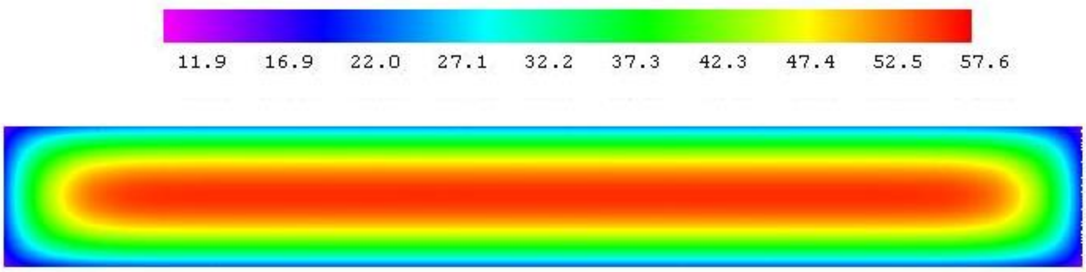

Figure 6. The temperature field on second day after pouring the concrete mixture (0C)

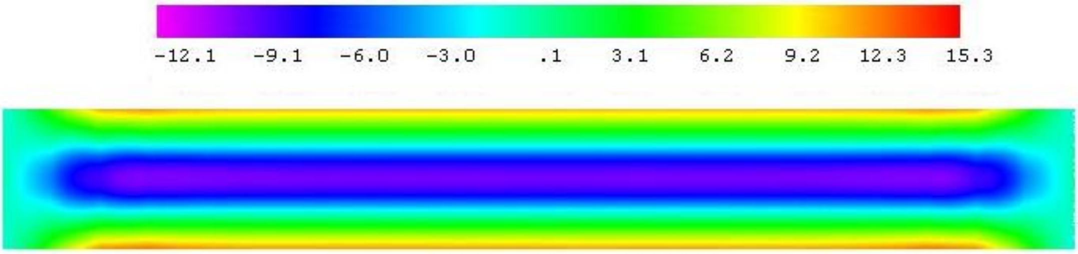

Figure 7. The stress field on second day after pouring the concrete mixture (kg/cm2)

The temperature field is non-uniform (2D problem). The diagrams of thermal stresses (X-axis) of the surface of the slab, according to the figure 7 decrease monotonously (they have maximum) – from the center of the slab to it's sides. As it shown on the Figure 9, in the cross-section of the slab there are 3 zones A, B, C. C – a new zone (mostly because zone B became shorter). C – is a zone of enhanced tensile stresses on the surface of the slab.

The length of zones along the X-axis equals to: zone B – 6.935 m, zone C – 2.152 m, zone A – 1.913 m. In relative units (relative to the length of the slab), respectively, 63 %, 19.6 % and 17.4 %.

Thermal stresses in zone C exceed stresses in zone B for first 3 days (when there are tensile stresses on the surface) from 7.63 % to 14.85 %

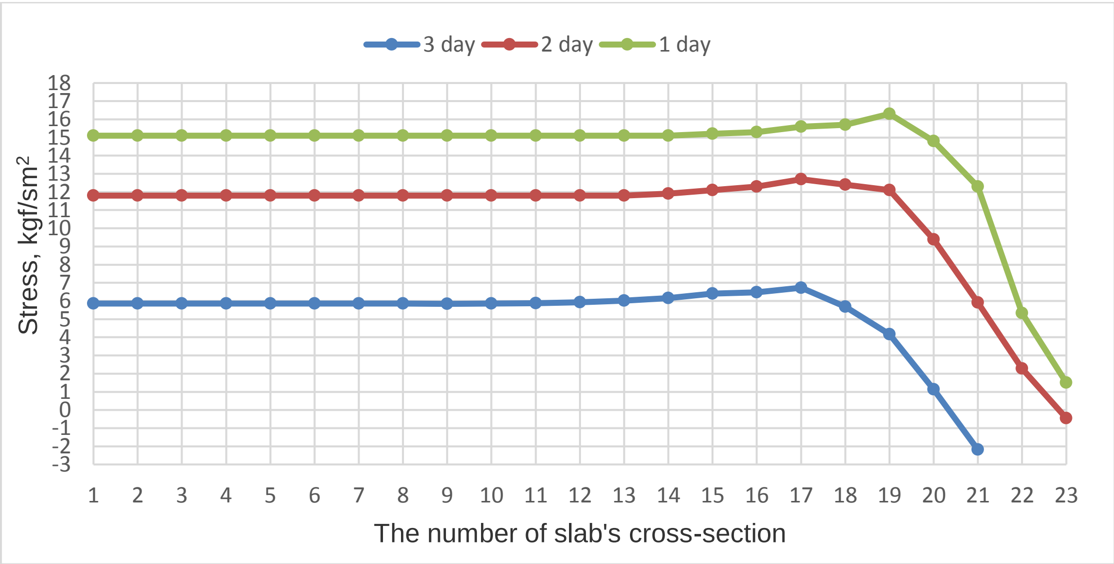

Figure 8. The diagrams of thermal stresses of the surface of the slab

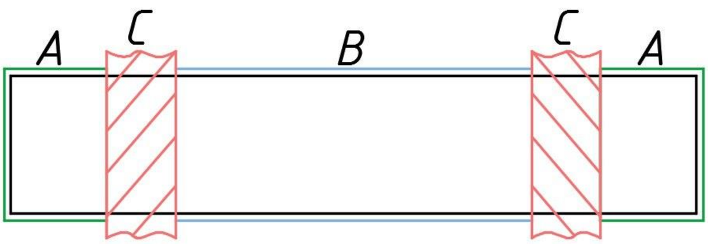

Figure 9. Zones of the slab

It is known that thicknesses of slabs which are used in industrial and civil constructions (especially in NPP and high-rise buildings) have range of 0.5-0.3 m. First and second numerical experiments were conducted for thicknesses 0.5-0.3 m and ratio $\frac{h}{l}$ was changing. There were 100 numerical experiments conducted.

For convenience the results of the experiments are represented in the table, part of which is listed below. The first and second columns are description of initial data for the experiment: the sizes of the cross-section and calculated ratio $\frac{\mathrm{h}}{\mathrm{l}}$. The third column is the number of days after the beginning of the experiment, when the mixture was poured. The fourth column is the length of zone B in percentage from the whole length. The fifth column the length of zone A, %. The sixth column is the maximal excess of thermal stress values in zone C above the values in zone B.

Table 1. The results of the numerical experiment

Slab's cross-section sizes h x l, м	$\frac{h}{l}$	Days	The length of zone B, %	The length of zone A, %	The maximal excess in zone C, %
1x3	0.33	1	19.2	46.2	8.7
		2	15.4	46.2	
1x4	0.25	1	38.2	44.7	9.4
		2	29.4	44.7	
1x5	0.2	1	50.0	38.1	7.0
		2	33.3	38.1	
1x6	0.17	1	56.0	32.0	7.0
		2	44.0	32.0	
1.5x4.5	0.33	1	26.3	42.1	7.2
		2	21.1	42.1	
1.5x6	0.25	1	36.0	36.0	8.5
		2	32.0	36.0	
1.5x7.5	0.2	1	51.6	25.8	7.9
		2	41.9	25.8	
1.5x9	0.17	1	56.8	21.6	8.5
		2	51.4	21.6	
2x6	0.33	1	28.0	36.0	9.3
		2	18.0	36.0	
2x8	0.25	1	42.4	30.3	9.2
		2	33.3	30.3	
2x10	0.2	1	53.7	21.9	9.2
		2	43.9	21.9	
2x12	0.17	1	59.2	19.4	9.2
		2	51.0	19.4	

Discussion

There is a limit for each structural model when it becomes impossible to use. The application of this research is that more accurate value of $\frac{\mathrm{h}}{\mathrm{l}}$ ratio was found and this ratio allows the use of one-dimensional structural models for calculations of massive concrete and reinforced concrete structures during the building period. In this research of the thermo-stressed state in massive foundation slabs with different planform dimension ratios the elaborated ratio $\frac{\mathrm{h}}{\mathrm{l}}$ was found allowing the use of one-dimensional structural model for the central part of the slab with a sufficient degree of accuracy when tension and temperature are functions of a single spatial coordinate – the vertical one: $T(z,\tau) = f(z)$; $\sigma(z,\tau) = f(z)$.

The transition to this model is valid for ratio $\frac{\mathrm{h}}{\mathrm{l}} < 0.17$. This value does not equal to the value that was used by other authors in articles [10–12, 14, 28–31]. They used the ratio in the range of $\frac{h}{l} \leq \frac{1}{3} \ldots \frac{1}{4}$.

The elaborated ratio $\frac{\mathrm{h}}{\mathrm{l}}$ decreases the number of observational errors in calculations. It also diminishes the calculation time and intensity. This ratio can be used to solve the wide range of problems that arise in modern practice during the erection of massive concrete structures.

Conclusions

The results of the conducted experiments allow us to make following conclusions:

1. Generally, one-dimensional structural model is suitable in cases when ratio $\frac{h}{l} < 0.17$, h stands for the minimal plane dimension in a construction such as slab;

2. The calculation results of thermo-stressed state with use of one-dimensional structural model should be used carefully in assessment of crack resistance of building blocks. This research indicates the existence of zones near the sides of slabs, where values of the tensile stress exceed the values that were obtained with use of one-dimensional structural model. This excess may account for 9.5 %;

3. In a real practice, ratio $\frac{h}{l}$ should be found individually, since intensity and amount of the heat dissipation significantly depend on exothermic characteristics of applicable cement and the amount of cement in the concrete mixture. Listed calculation results are based on constant concrete composition. The amount of cement and it's characteristics are averaged. There is also an influence on $\frac{h}{l}$ from β_{red} of the sides of the slabs and there are different unaccounted factors.

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Deformations of the periodic truss with diagonal lattice

Деформации периодической фермы с раскосной решеткой

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Key words: induction method; lattice truss; deflection; construction; structural mechanics; kinematic variability

Ключевые слова: метод индукции; решетчатая ферма; прогиб; конструкции; строительная механика; кинематическая изменяемость

Abstract. A scheme for statically determinate flat lattice-type truss is investigated. The truss has two supports and uniformly loaded at the upper zone. Analytical dependencies of the deflection of the structure and displacement of the movable support from its size, load and number of panels are obtained. It is shown that if the number of panels in multiples of three, the truss cinematically altered, as is clear from the zero determinant of the system of equilibrium equations. The relevant diagram of the possible speeds of the nodes was defined. The system of computer mathematics Maple with induction method was used for output results. This approach was previously proposed and developed by the author in some problems related to the pivotal plane and spatial trusses. It was discovered intermittent character of the dependence of deflection on the number of panels. Asymptotic properties of the solution were found. The features of the solution allow us to optimize the size of the structure.

Аннотация. Рассмотрена схема плоской статически определимой фермы решетчатого типа. Ферма имеет две опоры и загружена равномерной нагрузкой по нижнему или верхнему поясу. Найдены аналитические зависимости прогиба конструкции и смещения подвижной опоры от ее размеров, нагрузки и числа панелей. Показано, что если число панелей кратно трем, то ферма кинематически изменяема, что явствует из равенства нулю определителя системы уравнений равновесия. Найдена соответствующая схема возможных скоростей узлов. Для вывода результатов использована система компьютерной математики Maple и метод индукции, ранее предложенный и развитый автором в ряде задач о стержневых плоских и пространственных фермах. Обнаружен скачкообразный характер зависимости прогиба от числа панелей. Найдены асимптотические свойства решения. Выявленные особенности решения позволяют оптимизировать размеры конструкции.

Introduction

Regular (or structurally scalable) trusses represent a large class of rod systems, to which analytical methods of calculation based on the method of induction may be applicable. Flat trusses tend to have one natural parameter scale — the number of panels. In some problems, for example, in cantilever truss, there is a second parameter scaling (number of panels on the console). The problem of finding and calculating the new statically determinate rod structures of the regular type (periodic) studied in [1, 2]. Some questions of numerical methods for calculation, optimization, and fracture of beam structures considered in [3-8]. Examples of analytical and numerically-analytical solutions of problems of elasticity feature in the system of computer mathematics Maple [9] are given in [10, 11]. The exact solution for the deflection of the lattice trusses by the method of induction received and analyzed in [12–19]. Overview of analytical solutions for flat trusses, obtained in the system Maple can be found in [20, 21].

Design scheme

The truss contains the upper horizontal zone of rods, vertical struts and the inclined struts (Fig. 1). The truss with n panel contains $m = 4n + 12$ rods, including three support bar, and $s = 2n + 9$ joints with three joints (hinges) at the ends of the support rods. Given that for each node in the method of cutting of knots one can write two equations of equilibrium in projections, the system of equilibrium equations is closed, the design is statically determinate.

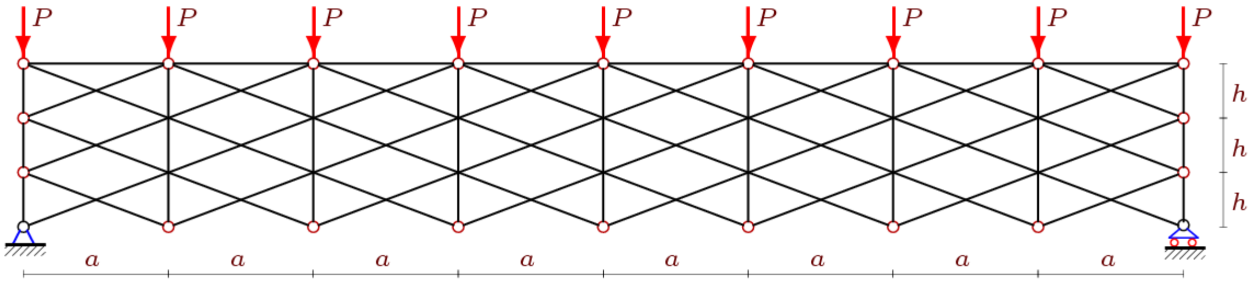

Figure 1. Truss, $n=8$

Let us introduce a coordinate system with the origin at the left fixed support to specify the geometry of the truss. For making the equilibrium equations of the nodes and finding the guides of the cosines of the effort, it will take the coordinates of hinges:

$$x_i = x_{i+n+1} = (i-1)a, \quad y_i = 0, \; y_{i+n+1} = 3h, \; i = 1,...,n+1,$$
$$x_{i+2+2n} = 0, \; x_{i+2+4n} = an, \; y_{i+2+2n} = y_{i+4+2n} = (3-i)h, \; i = 1,2.$$

The fixed bearing is modeled by two rigid bars that are attached to node 1: $x_{s-2} = 0, y_{s-2} = -h, \; x_{s-1} = -h, y_{s-1} = 0,$ a movable rod with one end pivot at the point $x_s = x_{n+1}, \; y_s = -h.$ While the length of the support rods is taken arbitrarily since according to the problem, they are rigid and their length does not influence the solution. The lattice structure of the truss is set the same way as is specified in discrete math graph conditional vectors $\bar{q}_i$, $i = 1,...,m$. The vectors of the diagonals of the lattice ($i = 1,...,n-2$), for example, have the following form: $\bar{q}_i = [i, i+n+4]$, $\bar{q}_{i+n-2} = [i+3, i+n+1]$. The rack is coded by the vectors: $\bar{q}_{i+3n+10} = [i+n+2, i+1]$, $i = 1,...,n-1$. Left and right hinge correspond to the vectors $\bar{q}_{m-2} = [1, s-2]$, $\bar{q}_{m-1} = [1, s-1]$, $\bar{q}_m = [n+1, s]$.

Solution

To determine the stresses in the bars program [12–16] based on the method of cutting of knots is used. The equilibrium equations of all nodes are organized in the general system of equilibrium equations:

$$G\bar{S} = \bar{T} \qquad (1)$$

with the matrix G of the guides of the cosines of effort calculated at the given coordinates. Here $\bar{S}$ is a vector of stresses in the bars, $\bar{T}$ a vector of given loads. For loads uniformly distributed on the upper zone, this vector has the form: $T_{2i} = -P, \; i = n+2,...,2n+2.$ Other components of the vector are equal to zero. A solution of a system of linear equations (1) finds in the symbolic form using Maple: $\bar{S} = G^{-1}\bar{T}$ where G^{-1} is the inverse matrix. The results of the program are analytical expressions for the stresses in the bars farm. For the computation of deflection use the formula of Maxwell-Mohr:

$$\Delta = \sum_{j=1}^{m-3} \frac{S_j N_j l_j}{EF}, \qquad (2)$$

where *E* is the modulus of elasticity of cores *F* – the cross-sectional area of the rods (the same as for the whole structure), l_j and S_j the length of the *j*-th core and force in it from the action of a given load; N_j — efforts from a single vertical force applied at mid-span. The summation is conducted on all rods of the truss, except hinges rods, which are assumed to be rigid. For the derivation of deflection for an arbitrary number of panels required to first obtain a sequence of solutions for a truss with different numbers of panels, and then by induction to generalize the solution to the arbitrary case. However, in the process of counting it was found that the determinant of the matrix G of the system (1) becomes zero if the number of panels in multiples of three. Obviously, this corresponds to a kinematic degeneration of the structure. Indeed, kinematic analysis for a truss with *n*=3 gave the following consistent characteristic of lattice schemes [14–16], the picture of possible velocity joints:

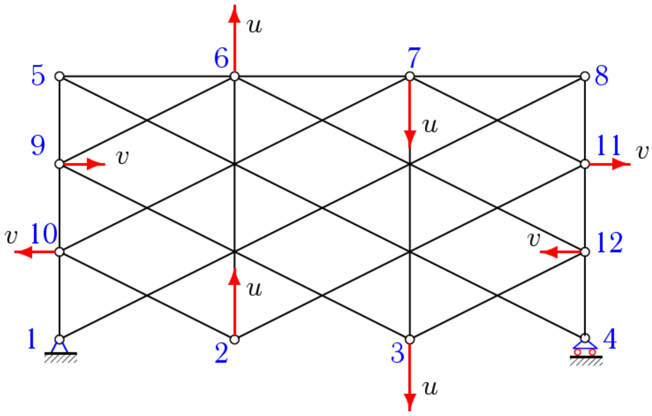

Figure 2. Scheme of possible velocity of joints

Rods with hinges 2–10, 3–12 commit instantaneous rotation around the poles 1 and 4, rods 6–9, 7–10 and 7–11, 6–12 – around corner points 5 and 8. Terminals 9–10, 11–12, 6–7 rotate around their mid-points. Rods 2–6 and 7–3 move forward with instantaneous speed *u*, the terminals 1–8 and 5–4 are fixed. Between the speeds there is the ratio $u/a = v/h$, following from a consideration of the provisions of the instant centers of velocity.

The same distribution of speeds obtained for other trusses in which the number of panels in multiples of three. In [16] discovered the kinematic degeneration and a diagram of possible speeds for stud truss with an odd number of panels. There are kinematically modified lattice in the statically indeterminate structures [15] also.

To obtain a sequence of solutions of trusses with the number of panels not multiple of three, we distinguish two cases. In the first case, the number of panels will be set by formula $n = 1+3k,\ k = 1,2,....$, in the second place $n = 2+3k,\ k = 1,2,.....$ Induction for these cases gives the general formula:

$$\Delta = P\frac{A_k a^3 + H_k h^3 + C_k c^3}{2h^2 EF}, \qquad (3)$$

where $c = \sqrt{a^2 + h^2}$. Two solutions differ only in the expression for the coefficients. In the first case the coefficients have the form:

$$A_k = \left(30k^4 + 40k^3 - 2\left((-1)^k + 15\right)k^2 + \left(20(-1)^k - 36\right)k - 7(-1)^k + 7\right)/32,$$

$$H_k = \left(10k^4 + 8\left(23(-1)^k - 6\right)k^3 + 6\left(7(-1)^k - 11\right)k^2 + 4\left(29(-1)^{k+1} + 37\right)k + 79(-1)^k + 113\right)/32,$$

$$C_k = \left(10k^4 + 8\left(23(-1)^k - 6\right)k^3 + 2\left(45(-1)^k - 17\right)k^2 + 4\left(11(-1)^{k+1} + 3\right)k + 9(-1)^{k+1} + 9\right)/64.$$

In the second ($n = 2+3k,\ k = 1,2,....$):

$$A_k = \left(30k^4 + 40k^3 + 2\left(3(-1)^k - 19\right)k^2 + 4\left((-1)^k + 5\right)k - 15(-1)^k + 15\right)/32,$$

$$H_k = \left(10k^4 + 136k^3 + 6\left(9(-1)^{k+1} + 5\right)k^2 + 4\left((-1)^k + 7\right)k + 47(-1)^k + 145\right)/32,$$

$$C_k = \left(10k^4 + 136k^3 + 2\left(27(-1)^{k+1} + 55\right)k^2 + 4\left(3(-1)^{k+1} + 11\right)k + 9(-1)^{k+1} + 9\right)/64.$$

The coefficient in the first case received in Maple system is a generalization of a sequence 0, 20, 97, 302, 712, 1446, 2625, 4412, 6976,..., 105372. Therefore, it is necessary to calculate a sequence of

16 trusses to get the pattern. For finding the general term of the sequence, the special operator rgf_findrecur from the Maple package genfunc is used and returns a recurrence equation:

$$A_k = 2A_{k-1} + 2A_{k-2} - 6A_{k-3} + 6A_{k-5} - 2A_{k-6} - 2A_{k-7} + A_{k-8}. \qquad (4)$$

A feature of the operator rgf_findrecur is that it only works with an even number sequence. The solution of equation (3) for the eight initial data 0, 20, 97, 302, 712, 1446, 2625, 4412 found operator rsolve. Verification of the obtained solution can be performed in the numerical solution that does not have significant limitations to run time. Similarly, but somewhat more complicated, for 18 trusses and with the equation of higher order other factors decision are defined.

To determine the *horizontal* displacement of the movable support is necessary to obtain analytical expressions of the forces in the bars from the action of a single force applied to this pole [22]. The solution according to the formula (2) takes the form

$$\Delta_h = P\frac{A_k a^3 + H_k h^3 + C_k c^3}{ahEF}.$$

If $n = 1+3k,\ k = 1, 2, ...$ we have the following coefficients

$$A_k = k(3k^2 + 3k - 2)/2,\ H_k = k^3 + 9k^2 + 5k + 2,\ C_k = k(k^2 + 9k + 2)/2.$$

If $n = 2+3k,\ k = 1, 2, ...$we have

$$A_k = (3k^3 + 6k^2 + k - 2)/2,\ H_k = k^3 - 6k^2 - 6k + 1,\ C_k = (k^3 - 6k^2 - 13k - 6)/2.$$

Note that the problem of horizontal offset of the support the degree of the polynomials that specify the coefficients is smaller than for the case of mid span deflection.

Analysis

Graphically, the dependence of the dimensionless relative deflection $\Delta' = \Delta EF/(PL)$ where $L=na$ from the number of panels k for a given panel length and height is displayed by points in figure 3. In this structure, this dependency is by definition discrete and non-monotonic, due to the "flashing" of the component species $(-1)^k$ in the coefficients of the solution. By the methods of Maple (operator, limit), you one can find out the extent of this growth:

$$\lim_{k\to\infty} \Delta'/k^3 = \frac{5(6a^3 + 2h^3 + c^3)}{192ah^2}. \qquad (5)$$

Noticeable jumps in the values of the deflection is due in part to the fact that the point at which deflection is determined, is, or exactly in the middle of the span for even values of n, or near the geometric center of the span. When the number of panels is small, the deflection may even have a different sign for trusses with different from each other at the one panel. With the increasing number of panels, this effect disappears, however, the difference in deflection similar in a set of trusses can achieve up to four times. The last conclusion shows the possibility of optimizing the stiffness of the truss by the rational choice of a number of panels.

The results of computing the relative displacement of the support $\Delta'_h = \Delta_h EF/(PL)$ also show an abrupt change in the result of which depends mainly on the parity of the number of panels. The growth rate in this case is less and determined by the limit

$$\lim_{k\to\infty} \Delta'_h / k^2 = \frac{3a^3 + 2h^3 + c^3}{6a^2 h}.$$

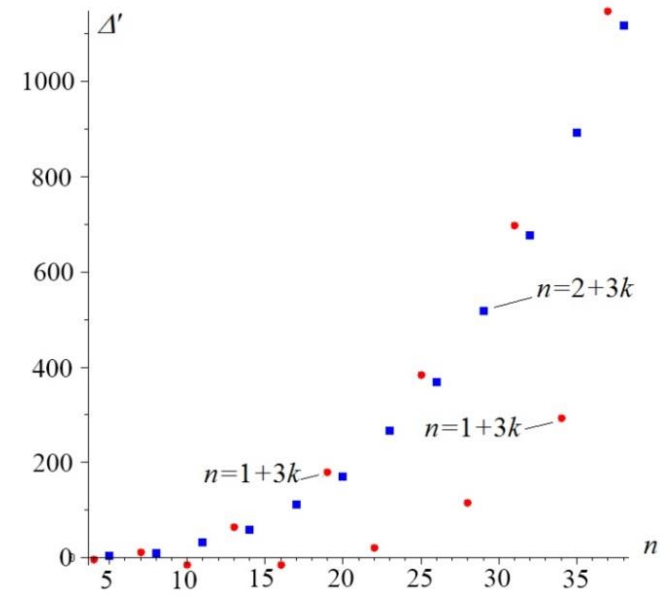

Figure 3. The Deflection, $a = 3\mathrm{m}, h = 2\mathrm{m}$

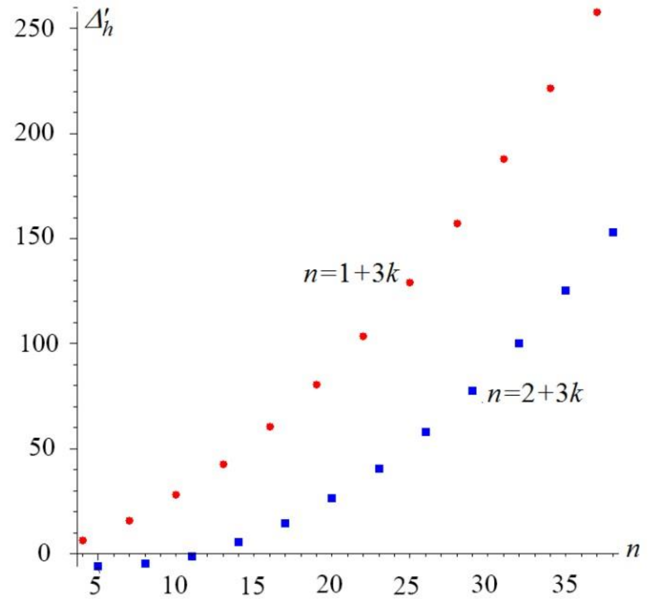

Figure 4. Offset of the support, $a = 3\mathrm{m}, h = 2\mathrm{m}$

The solution to (3) when any load is easily generalized to the case when the stiffness of the rods of the zones and grids are different. If to express the stiffness of the groups of rods of length *a*, *h* and *c* respectively using some given rigidity $EF_j = EF_0 / \mu_j$, $j = 1,...,3$, the result would be

$$\Delta = P\frac{A_k\mu_1a^3 + H_k\mu_2h^3 + C_k\mu_3c^3}{2h^2EF_0}.$$

Results and Discussion

The formulas for calculation of the deflection and displacement of the support trusses with an arbitrary number of panels are obtained. The main advantage and at the same time the main difficulty here is the dependence on the number of panels obtained by the method of induction. Induction method proposed and developed by the authors [12–15]. It should be noted that in the courses of structural mechanics the same formulas for estimating the deflection in the domestic and foreign literature are not found. The only well-known formula V.K. Kachurina [23, c. 310] for the optimal height of the truss selected for rigidity, is very approximate, intended for almost all of the girders and not take into account any particular structure of the lattice or the number of panels. Another alternative discussed in the article the approach is a fundamental theory of periodic lattices L.S. Rybakov [24], applicable to a wide class of structures. However, this theory analytically implements the finite element method, can't give a simple formula, closed form solutions, suitable for analysis and implementation. The method of induction, together with a system of symbolic mathematics allows not only to obtain a compact closed solution but also to identify in some cases, the threat of its features. In this calculation, it has been shown the kinematic system degeneration when the number of panels in multiples of three. Similar features were found in [14–16]. The analytical form of the solution allowed to reveal the asymptotic behavior of it. Comparative asymptotics of analytic solutions is dedicated in [20]. Special note is the object of study. This truss with complex diagonal bars not previously been investigated. The scheme of this statically determinate truss was developed by the authors.

Conclusions

The proposed scheme of the lattice girder is not quite normal. Bottom chord is not straightforward. It is more correct to say that it is not in traditional form. One of the purposes of introducing this scheme was to add architectural expression to the truss, in order to break the habitual way of the trusses with triangular lattice and rectilinear lines. By the way, it was discovered one important constructive feature of the design. It turned out that when the number of panels is three-fold, truss becomes instantaneously changeable mechanism. It is confirmed by the diagram of possible speeds. However, despite this feature, the purpose – output compact and convenient for practice of the formula, is achieved. A sequence of numbers is generated in which numbers of multiples of three are eliminated and formulas for deflection are derived for it and asymptotic estimates are obtained, which are necessary mainly for the comparative characterization of various lattice schemes.

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Both sided irradiated track membrane in local water supply

Двусторонне облученные трековые мембраны в локальном водоснабжении

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Key words: track membrane; water treatment; local water supply; buildings; filtration process; productivity; efficiency; spectrophotometry; spectroturbidimetry; civil engineering

Ключевые слова: трековая мембрана; очистка воды; локальное водоснабжение; фильтрация; производительность; эффективность; спектрофотометрия; спектротурбидиметрия; гражданское строительство

Abstract. Membrane filtration is one of the main methods for individual and local water supply systems. Track-etched membranes are among the types of membranes that allow to obtain high quality purified water due to their high selectivity. Ensuring the durability of track-etched membranes is one of the topical issues on their use. Comparative research of the natural water filtration process using standard 12-μm track-etched membrane and new, more durable 20-μm irradiated on both sides was performed. Both membranes were manufactured by NPF TreM, St. Petersburg. The pore diameter of membranes was 0.20–0.205 μm, the pore density was 2.3×10^8 cm^{-2} and 1.5×10^8 cm^{-2}, respectively. The research was conducted with natural water from the pond in the park, "Malinovka" (St. Petersburg, Kosygin pr.). The filtration was run in the dead-end model. Raw water and filtrate samples were analyzed by spectrophotometry and spectroturbidimetry using KFK-3.01 photoelectrocolorimeter and SF-56 spectrophotometer. The dispersion analysis of nature water was performed by dynamic light scattering on Zetatrac laser analyze. The experimental data showed the same dependences of the productivity of dead-end filtration of pond water samples upon the volume of passed water and close values of turbidity and color in filtrate samples for both membranes. The results allowed us to recommend the 20-μm-thick membrane irradiated on both sides with a beam of argon ions having a range shorter than the film thickness for natural water purification for local water supply of individual buildings.

Аннотация. Мембранная фильтрация является одним из основных методов, применяемых в индивидуальном и маломасштабном водопользовании. С помощью трековых мембран, обладающих высокой селективностью, можно получать очищенную воду высокого качества. Но при их использовании необходимо обеспечивать прочность трековых мембран, что является актуальной проблемой. В работе проведено сравнительное изучение процесса фильтрации природной воды с использованием стандартной трековой мембраны толщиной 12 мкм и новой, более прочной, мембраны толщиной 20 мкм, облученной с обеих сторон. Обе мембраны изготовлены НПФ «Трем» (Санкт-Петербург). Диаметр пор обеих мембран составлял 0,20–0,205 мкм, плотность пор – $2{,}3 \times 10^8$ cm^{-2} and $1{,}5 \times 10^8$ cm^{-2}, соответственно. В экспериментах использовали природную воду из пруда парка «Малиновка» (Санкт-Петербург, пр. Косыгина). Фильтрацию проводили в тупиковом режиме. Образцы природной воды и фильтрата анализировали с помощью методов спектрофотометрии и спектротурбидиметрии на фотоэлектроколориметре КФК-3.01 и спектрофотометре СФ-56. Дисперсионный анализ природной воды проводили методом динамического светорассеяния на лазерном анализаторе Zetatrac. Для сравниваемых мембран были получены одинаковые зависимости производительности процесса фильтрации от объема пропущенной природной воды в тупиковом режиме, а также близкие

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значения мутности и цветности в пробах фильтрата. Полученные результаты позволили рекомендовать трековую мембрану толщиной 20 мкм, облученную с обеих сторон пучком ионов аргона с длиной пробега меньшей, чем толщина пленки, для очистки природной воды в питьевых целях.

Introduction

The problem of providing the population with pure drinking water is still remains actual for cities, towns, villages and individual consumers. The technologies of water treatment are becoming more unified for big cities. In the same time water treatment technologies for the individual consumers, small farms could strongly differ depending on the water quality in the water source, the presence of available materials (natural filtering materials, sorbents, membranes [1] or reagents),utilization of these materials, strength characteristics, and so on. Experts of water treatment and students from different universities have been engaging this problem. For example, teams of students were given different methods of water purification: membrane filtration, membrane filtration coupled with an activated carbon adsorption, ultra violet disinfection and etc [2]. The students evaluated the treatment methodologies in terms of their cost, ease-of-use, energy requirements, efficacy (on indicators of color, turbidity, and odor) and time of treatment (they also determined the time to purify the minimum amount of water needed to human a day according to the WHO recommendations).

The above implies that the membrane filtration is one of the main methods for individual water purification and local water supply. Membrane technology also dominates among other water purification technologies owing to its energy efficiency [3].

Track membranes are among the types of membranes that allow obtaining high quality purified water [4–6]. Track (nuclear) membranes are made from 12–23-μm-thick polymer films by bombarding with high-energy heavy-ion beams [7-12]. The diameter of these pores can vary in the range from 0.05 to 5 microns, depending on etching conditions.

Track Membranes are characterized by high selectivity [13, 14], but they have smaller filtration productivity than other types of membranes.

Before recommending membrane for use in the filter element, it is necessary to make certain of their strength characteristics. Therefore, durability of the membrane determines the economic efficiency of purification technology [6].

Track membrane with improved strength characteristics was obtained in [15] by realization energy-efficient variant of irradiation 20-μm-thickpolyethylene terephthalate film. Irradiation was done on both sides of a film with a beam of 53.4 MeV Ar^{+8} ions (this energy is not sufficient for the formation of a through track).

The aim of this work was to study productivity and efficiency of natural water purification process using 20-μm-thicktrack membrane produced by both side irradiation, and standard 12-μm-thick track membrane to recommend its application in the local water supply of individual buildings.

Methods and Materials

Experiments were performed on track membranes with a thickness 12 μm irradiated on one side and 20 μm irradiated on both sides (manufactured by NPF TreM, St. Petersburg). The pore diameter of membranes was 0.20–0.205 μm, the pore density was 2.3×10^8 cm^{-2} and 1.5×10^8 cm^{-2}, respectively.

The research was conducted with natural water from the pond in the park, "Malinovka" (St. Petersburg, Kosyginpr.). A preliminary defining of the relationship between the productivity of the filtration process and pressure were run with distilled water.

The filtration in this research was run in the dead-end model. In this work the cell with a filtration area of 25.5 cm^2 (manufactured by the Research-and-Production Association for Analytical Instrumentation of the RussianAcademy of Sciences, St. Petersburg) was used. The volume of this cell was 200 cm^3.

Raw water and filtrate samples were analyzed by spectrophotometry and spectroturbidimetry (KFK-3.01 photoelectrocolorimeter, Zagorskiy Optical and Mechanical Plant (ZOMZ), SF-56 spectrophotometer, OOO LOMO SPEKTR, St. Petersburg). The dispersion analysis of nature water was perfomed by dynamic light scatteringonZetatrac laser analyzer (Microtrac Inc., USA).

The electrical conductivity of the water samples was measured using a portable Multi-Range Conduction / TDS meter HI 8733N (HANNA Instruments, Germany), pH – by ionomer I-500 (OOO Aquilon, Russia).

Filtration productivity G, cm/(c*bar), was determined by the formula (1):

$$G = \frac{V}{t \cdot P \cdot S}, \quad (1)$$

where V – volume of the sample, cm^3; t – sampling time, s; P – pressure, bar; S – filtration area, sm^2.

According to the spectroturbidimetrymethod, the average particle size *d*, nm, was determined by the formula (2)

$$d = \frac{\alpha \cdot \lambda}{\pi \cdot \mu_0} \quad (2)$$

where α – relative particle size (determined by the wave exponent *n*); μ_0 –refractive index of the dispersion medium; λ – average wavelength, nm.

The average concentration ν,cm^{-3}, of the average particle size was determined by formula (3)

$$\nu = \frac{1.26 \cdot 10^{17} \cdot \tau}{(\lambda')^2 \cdot K(\alpha, m) \cdot \alpha^2} \quad (3)$$

where $\lambda' = \frac{\lambda}{\mu_0}$, in angstrom; $\tau = \frac{2.303 \cdot D}{\ell}$ (τ – turbidity; *D* –average optical density; ℓ – optical path length; m – relative refractive index; $K(\alpha,m)$ – coefficient of scattering.

Results and Discussion

While selecting an object of the research, comparison of the characteristics of water samples from the two ponds of St. Petersburg was made. Rivers and brooks flowing within the city were characterized by high pollution degree [16], so they have not been explored.

Some indicators of water samples from a pond Olginsky (st. Jacques Duclos) and pond of park Malinovka (pr. Kosygin) are presented in Table 1.

Table 1. Indicators of water quality of the pond Olginsky and thepond of park "Malinovka"

Indicator	PondOlginsky	Pondofpark "Malinovka"
Color,degrees	11	51
Turbidity, mg/L	0.6	4.0
pH	8.0	7.7
Specific electrical conductivity, µS/cm	200	590

Based on the obtained data we chose the water from the pond of park "Malinovka" because of its color and tubidity exceed the allowable values for drinking water [17].

For the sample filtered through roll filter paper, the spectrum in coordinates (lgD–lgλ) was constructed (Fig. 1).Linear region in the wavelength range 500–700 nm was selected on this spectrum. The trendline, tangent of the angle of which was equal to 3.5, was drawnfor it. This is the wave exponent value. Using the relationship between the characteristic features of the spektroturbidimetric method [18] values of the average particle size and average concentration were calculated according to the formulas (2) and (3).Since the value of wave exponential slightly exceeded its maximum (n_{max} = 3.4) average particle size and average particle concentration were evaluative: d < 140 nm, ν > 2 x 10^9 cm^{-3}.

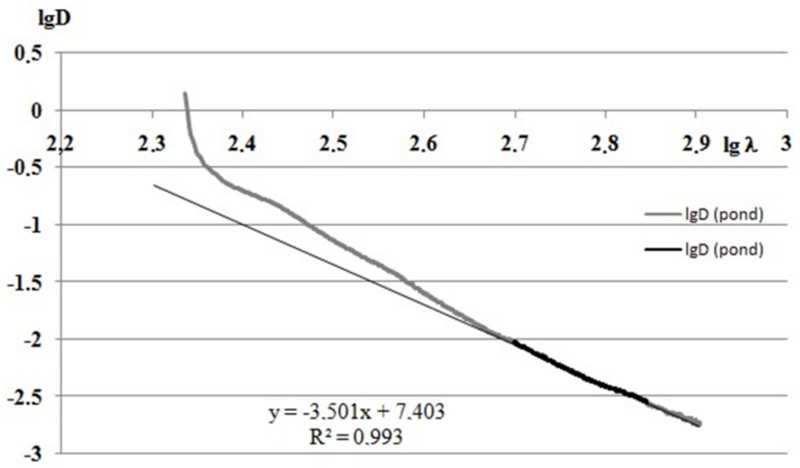

Figure 1. Dependence of the optical density logarithm on the wavelength logarithm for water sample from a pond of park Malinovka

For determining the particle size distribution dynamic light scattering method was used.The measurements were performed on the analyzer Zetatrac [19], which showed ample opportunities during the study of aggregation of polydisperse systems [20].The data were represented as scattered light intensity and particle number distributions over particle sizes.The results are shown in Figure 2 and Table 2.

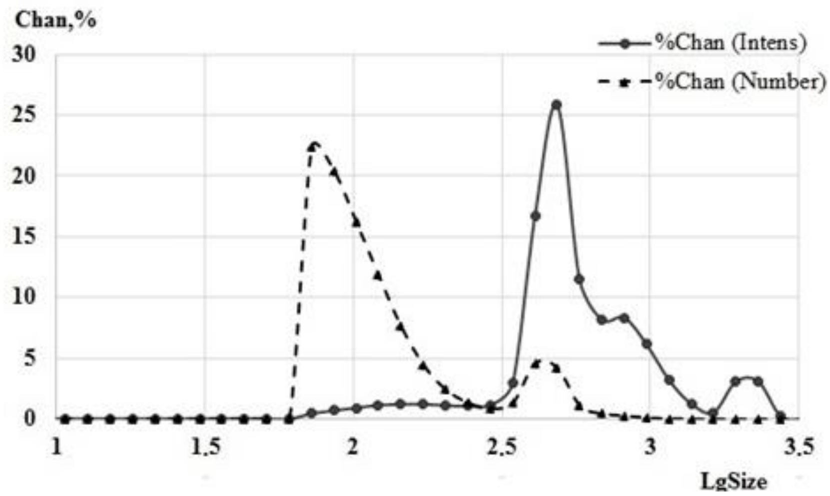

Figure 2. Scattered intensity and particle number distributions over particle sizes for water sample from a pond of park Malinovka

Table 2. Peaks Summary

Distribution	Size, nm	Vol, %	Width, nm	Loading Index
Scatteredintensity	1943	6.7	445	0.014
	459	93.3	390	
Number	410	12.5	138.4	
	86.7	87.5	59.6	

These results demonstrate that the water sample mostly contains suspended impurities smaller than 100 nm. However, a small number of larger impurities (above 400 nm) determines the main contribution to light scattering.

Microfiltration experiments were carried out in dead-end mode on the investigated membranes (12-μm-thick irradiated on one side and 20-μm-thick irradiated on both sides).

Dependences of the productivity of dead-end filtration of distilled and pond water samples through two track-etched membranesare presented at Figure 3.

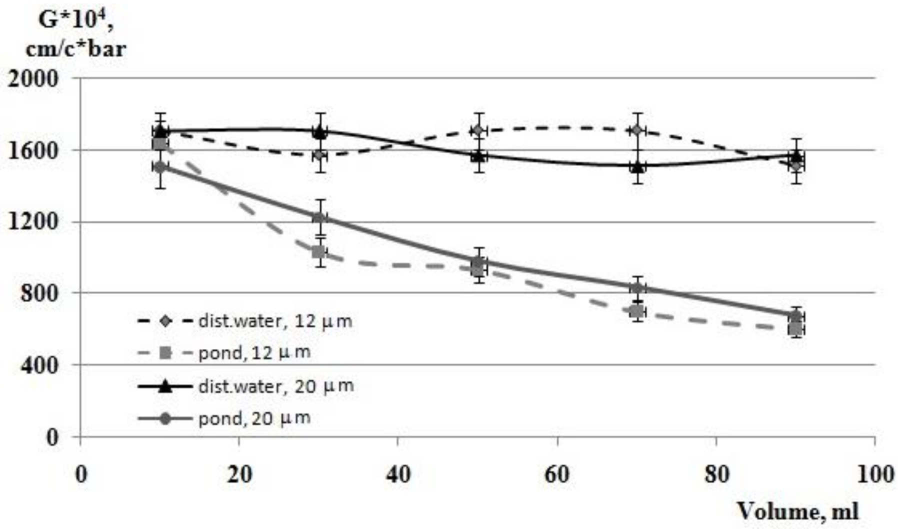

Figure 3. Dependences of the productivity of dead-end filtration of distilled and pond water samples through 12-μm-thick membrane irradiated on one side and 20-μm-thick membrane irradiated on both sides.

The results shows the same productivity properties of studied membranes, despite the high porenumber density of the 12-μm-thick membrane irradiated from one side, and almost twice-greater length of the pores in the 20-μm-thick membrane irradiated from both sides. Apparently, the transport properties of the membrane irradiated from both sides is formed with a layer of "connectivity" in the middle of the film (12–14 nm) [15].

Fig. 4 and 5 show the results of comparison of purification effectiveness (in terms of color and turbidity) of pond water by filtration in a dead-end mode on two investigated membranes.

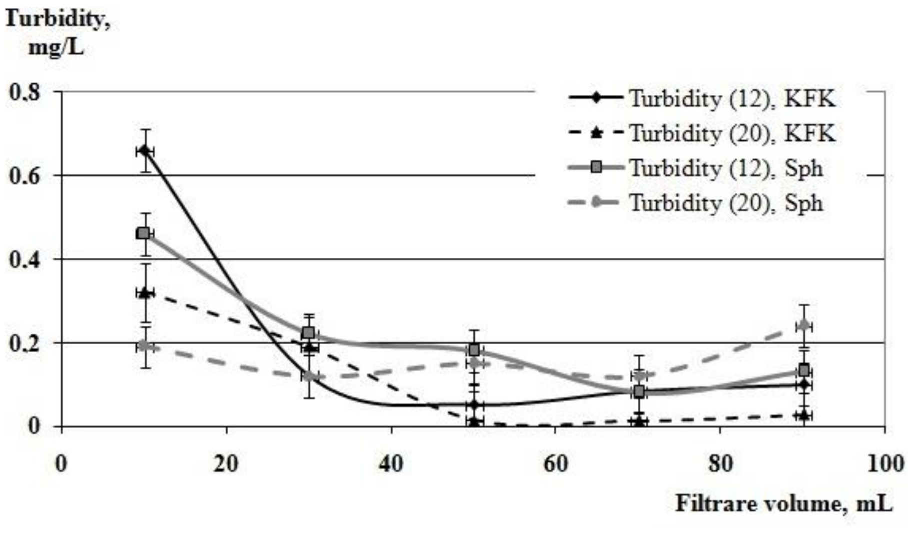

Figure 4. Dependences of turbidity in filtrate samples on filtration volume when optical density was measured using photoelectrocolorimeter (KFK) and spectrophotometer (Sph). 12–12-μm-thick membrane; 20–20-μm-thick membrane

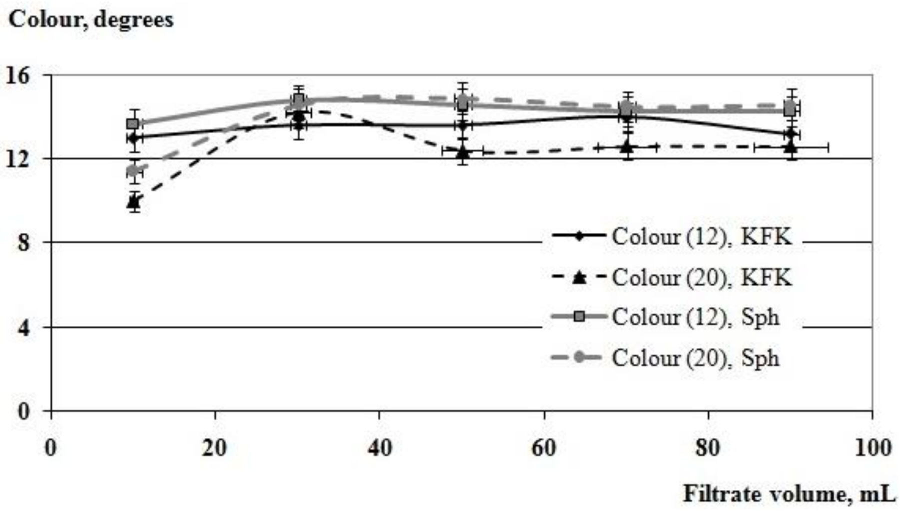

Figure 5. Dependences of colour in filtrate samples on filtration volume when optical density was measured using photoelectrocolorimeter (KFK) and spectrophotometer (Sph). 12–12-μm-thick membrane; 20–20-μm-thick membrane

Results show almost equal values of turbidity and color in the filtrate samples obtained by using the 12- and 20-μm-thick membranes. It is also seen that both membranes provide reduction of turbidity and color of water from the pond to values less than maximum permissible concentration (MAC) for drinking water (beginning from the second sample 1.3–1.4 times by color indicator, 7–10 times for turbidity).

It should be noted that theproductivity of filtration of pond water decreases slightly compared to the productivity of distilled water filtration. To reveal the cause of the drop in performance of pond water filtration process, curves for the dependence of the squared inverse productivity upon the overall filtration time were plotted. Comparison of the curves with the theoretical relationships [21] suggested the adsorption on the pore surface and pore clogging (as in [22]). This is confirmed by the relation of pore sizes (0.2 μm) and suspended impurities from pond water (Fig.2, Table 2).

Determination of suspended particle size in the filtrate samples and the size distribution by the methods used in the work was not possible, because the concentration of suspended impurities was low than sensitivity of the analyzers.

Conclusions

Comparative study of the natural water filtration process using a track membrane produced by implementing energy-efficient variant of irradiation of 20-μm-thick PET film on both sides, and a standard 12-μm-thick membrane irradiated from one side having the same pore diameter of 0.2 microns, has shown:

– similarproductivity of the filtering process;

– the same removal efficiency of suspended impurities with an average size of 90 nm and above.

The results obtained allow us to recommend the 20-μm-thick membrane obtained with a beam of argon ions having a range shorter than the film thickness for natural water purification for local water supply in such objects of civil engineering as individual buildings.

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Приглашает специалистов проектных и строительных организаций,
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- ***Технология и контроль качества строительства***
- ***Основы проектирования зданий и сооружений***
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- ***Управление строительной организацией***
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- ***Проектирование, монтаж и эксплуатация систем водоснабжения и водоотведения***
- ***Автоматизация проектных работ с использованием AutoCAD***
- ***Электроснабжение и электрооборудование объектов***

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- ***Проектирование и расчет железобетонных конструкций***
- ***Проектирование и расчет металлических конструкций***
- ***Проектирование зданий и сооружений с использованием AutoCAD***
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