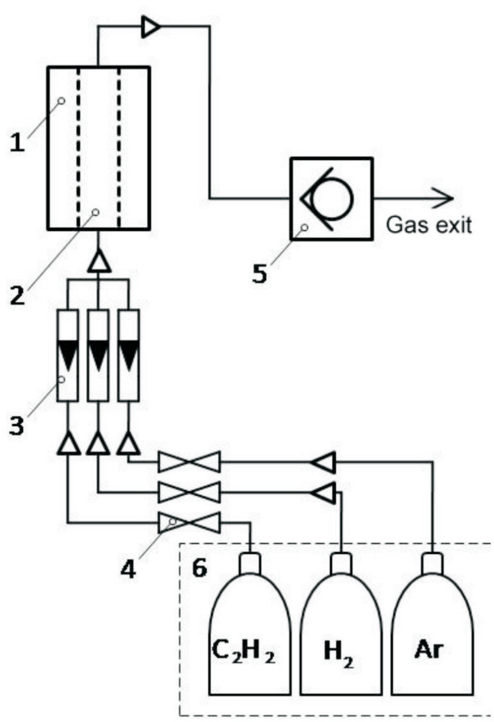
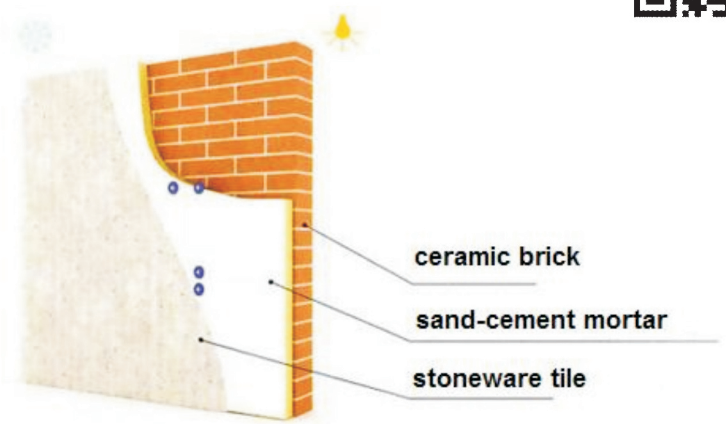
ISSN 2071-4726
2071-0305

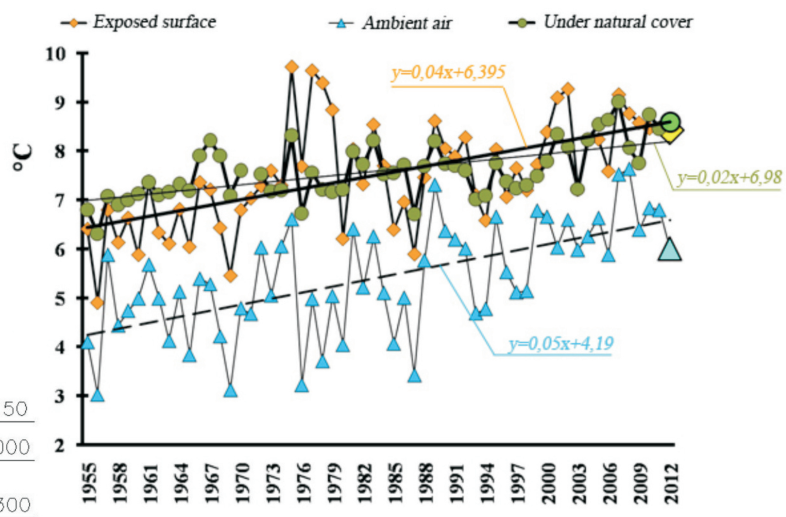
Инженерно-строительный журнал

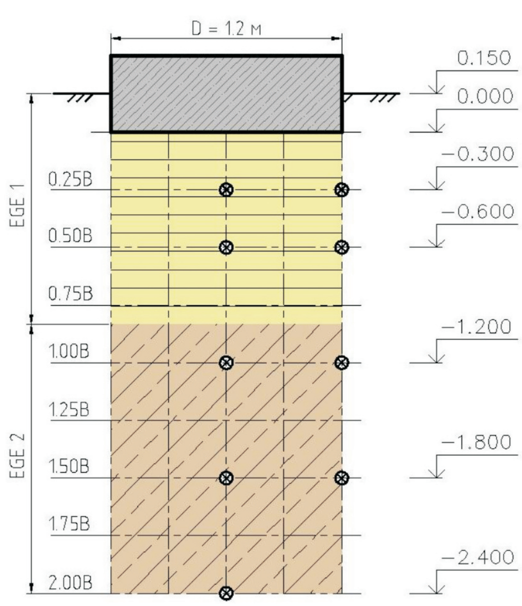
НАУЧНОЕ ИЗДАНИЕ

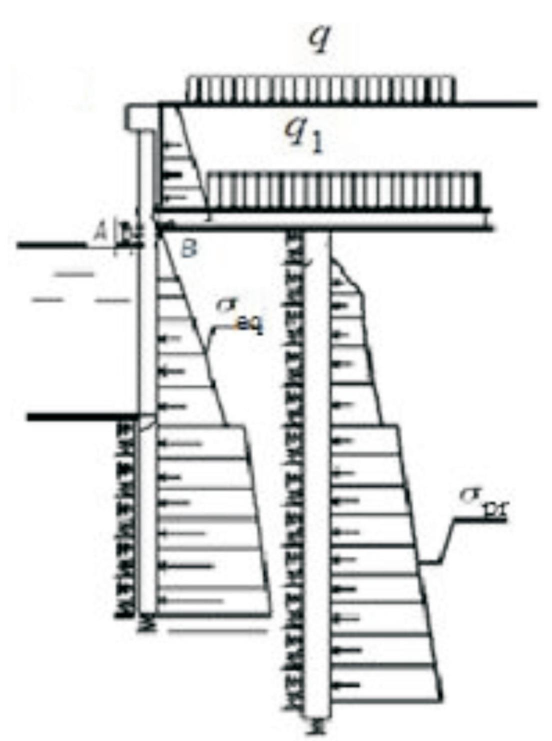
№4(72) 2017

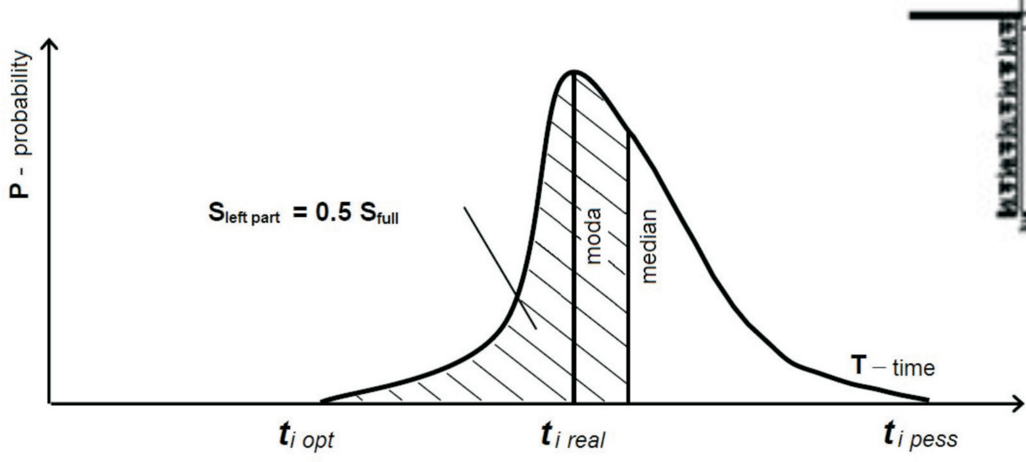












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Инженерно-строительный журнал
НАУЧНОЕ ИЗДАНИЕ

ISSN 2071-4726

Свидетельство о государственной регистрации: ПИ №ФС77-38070, выдано Роскомнадзором

Специализированный научный журнал. Выходит с 09.2008.

Включен в Перечень ведущих периодических изданий ВАК РФ

Периодичность: 8 раз в год

Учредитель и издатель:

Санкт-Петербургский политехнический университет Петра Великого

Адрес редакции:

195251, СПб, ул. Политехническая, д. 29, Гидрокорпус-2, ауд. 227А

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Содержание

На обложке: иллюстрации авторов к статьям номера

Установочный тираж 1000 экз.
Подписано в печать 13.09.2017. Формат 60x84/8, усл. печ. л. 10,5. Заказ № 2104
Отпечатано в типографии СПбПУ. СПб, ул. Политехническая, д. 29

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http://www.engstroy.spbstu.ru – full-text open-access version in Internet. It is updated immediately with each new issue.

Magazine of Civil Engineering
SCHOLAR JOURNAL

ISSN 2071-4726

Peer-reviewed scientific journal

Start date: 2008/09

8 issues per year

Publisher:

Peter the Great St. Petersburg Polytechnic University

Indexing:

Scopus, Russian Science Citation Index (WoS), Compendex, DOAJ, EBSCO, Google Academia, Index Copernicus, ProQuest, Ulrich's Serials Analysis System

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On the cover: authors' illustrations

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Web: http://www.engstroy.spbstu.ru/eng/index.html

doi: 10.18720/MCE.72.1

Shielded retaining wall in the form of substandard rack-mount frame

Экранированный больверк в виде нестандартной стоечной рамы

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Key words: berth; shielded retaining wall; variable foundation modulus; silage pressure; incomplete sliding wedge

Ключевые слова: причальная набережная; экранированный больверк; переменный коэффициент постели; силосное давление грунта; неполная призма обрушения

Abstract. The article gives a practical implementation of the engineering method for calculation of different retaining walls types. The main provision of the method is to provide the construction calculation model as a combination of construction elements with the respective conditions of their fixing. The calculation model of shielded retaining wall (covered type of sheet pile wharf) is proposed in the form of substandard rack-mount frame where resilient ground attachment is accepted instead of rack lower anchorage. This model uses stiffness characteristics of the ground in the form of variable foundation modulus. The article considers the definition of lateral earth pressure on the wall while staying in sliding wedge of pile bent. The article gives an engineering solution for definition of diagrams of lateral pressure on the front wall with allowance for nonlinear influence of pile bent location, piles pitch and cross section.

Аннотация. Приведена практическая реализация, предложенного ранее инженерного универсального метода расчета причальных набережных. Основное положение метода заключается в представлении расчетной схемы сооружения в виде комбинации элементов конструкции с соответствующими условиями их закрепления. Расчетная схема экранированного больверка предложена в виде нестандартной стоечной рамы, где вместо нижней жесткой заделки стоек, принимается упругая грунтовая заделка. При этом используются жесткостные характеристики грунта, в виде переменных коэффициентов постели. Рассмотрен вопрос определения бокового давления грунта на стенку при нахождении в призме обрушения свайного ряда. Получено инженерное решение определения эпюры бокового давления на лицевую стенку с учетом нелинейного влияния расположения свайного ряда, шага и сечения свай.

Introduction

The calculation of water transport hydrotechnical constructions interacting with the ground belongs to one of the mixed elastic-plastic tasks of continuous body. There are a significant number of software packages (Geomechanica, Plaxis, Msheet, LIRA, etc.). They allow by a numerical method on a PC to calculate the stress-strain state of the construction in two and three dimensional statement taking into account elastic and extreme areas that previously was impossible. The calculation of berthing constructions in software packages using the model of a continuous body allowed to obtain a more complete picture of construction work in the ground. However, the calculation of these constructions in programs which use the model of a continuous medium designed for structural materials is not strict because it gives an approximate picture of the pile foundation behaviour in discrete soil ground

Due to the complexity of consideration of many factors affecting the structures in the port hydraulic engineering various engineering solutions [1–7] are widely used including methods considering foundation modulus (hypothesis of Fuss-Winkler) and not taking into account the distribution capacity of the ground. Some of the works used abroad [8–17] should be noted. These papers consider the tabulated solutions to the classical Blum-Lohmeyer method of calculation [8]. Abroad in the last 10–15 years the calculation of berthing facilities is performed in the program Plaxis using the finite element method for the continuum model. This model designed for construction materials is not strict for soils. It gives an approximate picture of pile foundation behaviour in discrete soil ground.

Using software packages based on the model of continuous body reference to the tasks of port hydraulic engineering requires consideration of several factors: difference in compression and tension ground resistance, variable degree of distribution capacity of the ground adjacent to the deformed wall up to its complete disappearance, for example, because of buttress piers end displacement leading to significant transformation of initial outline of pressure diagrams on height and so on [18]. The using of vertical thin layer of ground with reduced strength characteristics (interface) and so on at the contact point between the ground and wall describes their connection in the model of continuous body corresponding to the full adhesion of two areas incorrectly. It is not accident when publishing tasks associated with anchored enclose in programs that implement a continuous body the diagram of contact lateral earth pressure on the wall is not provided, and the obtained contour curves of the horizontal stress of ground behind the wall is more or less similar to the Coulomb diagram [19–23].

The model of discrete (grainy) environment revealing the physical mechanism of intergrain interaction more corresponds to the behaviour of backfill ground behind the wall than the model of continuous body based on a phenomenological approach [24].

Shielding elements which take a part of lateral pressure are widely used in deep-water sheet pile wharf, so that allows us to use a sheet piles with lower bearing capacity [1]. In addition, both task of improving the method of covered type of sheet pile wharf calculation and task of construction units' lateral pressure determination are relevant.

The purpose of work is to refine the shielding (insulation) effect of front wall due to the pile row in shielded retaining wall. In technical literature the effect of piles' step and their distance to the front wall on the shielding effect is not considered [1, 6, etc.]. Moreover, this effect is taken into account indirectly through the stiffness characteristics of pile foundation elements while determining their bending moments [6]. The authors set the task of proposing a direct method for determining the pressure on the wall from the soil wedge cut out by a pile row. To solve this problem a scheme for the distribution of the resulting lateral pressure in the form of a strip load between the piles acting on the front wall is considered. In this case, the piles' step and the distance to the wall are taken into account.

Materials and Methods

It is commonly believed that ground vaults are formed between the piles of longitudinal row on account of ground friction on piles (Fig. 1a) [6]. For simplicity they are replaced by a scalloped surface (Fig. 1b). These vaults take up lateral pressure of the ground located behind them and transmit it to the piles.

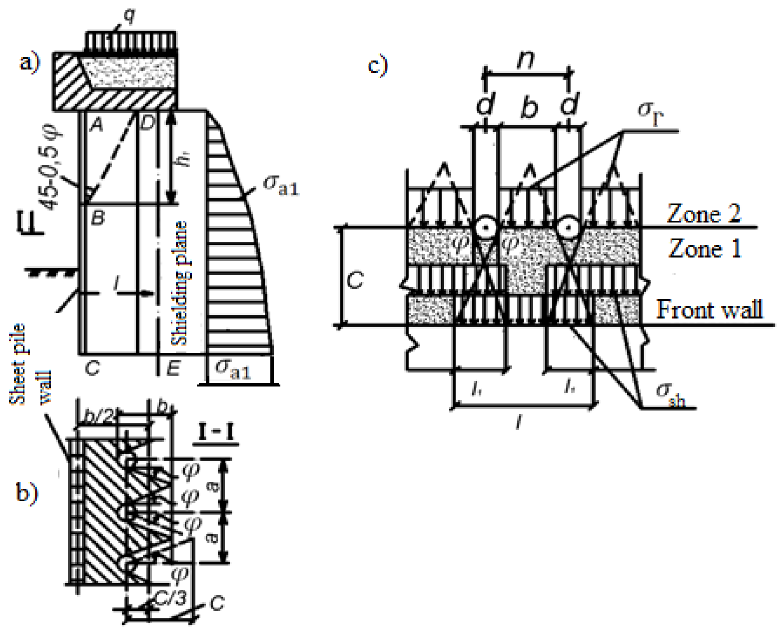

Figure 1. Composition schemes of diagrams of silage ground pressure on the front all (a) and (b) additional shielding load pressure behind pile row (c)

Only the pressure of the ground located between the wall and the ground vaults of piles' longitudinal row fully acts on the sheet pile wall. For calculations vaulted surface is replaced by a conventional vertical shielding plane shielding DE (Fig. 1a). The position of the shielding plane is usually determined by graphical constructions (Figs. 1a and b) [6].

Shielding plane is accepted at a distance c/3 from the axis of longitudinal pile row, where c is the scallop height in this case. The diagram of backfill ground pressure on sheet pile wall is constructed considering the position of shielding plane.

Ground filling between sheet pile wall and shielding plane is in conditions similar to the state of granular material in a flat silage. In the graphical calculation silage pressure is calculated by drawing (at an angle of 45 ° – 05 φ to the vertical) from crossing point D of shielding plane with foundation grill a line DB of ground break to its intersection with the calculated wall plane (Fig. 1a.).The ground pressure at the wall segment AB rises linearly. The ordinate of pressure diagram at point B.

$$\sigma_{a1} = \gamma \cdot h_1 \cdot \lambda_a \tag{1}$$

Below point B the intensity of ground pressure remains constant σ_{a1}.

$$\sigma_{sil} = \gamma h\, \lambda(\delta),\ 0 < h < h_1; \quad \sigma_{sil} = const \ \text{ if } \ h \geq h_1 = l\mathrm{tg}(45° - 0.5\varphi°)\ . \tag{2}$$

The authors believe that the replacement of pile row by solid shielded wall is a particular case of pressure at sufficiently flexible front wall and a small pitch distance. In such wall deflection implements an active pressure with the manifestation of ground vaults when the step between piles is $n \leq 3d$. In technical literature the value of bending moment in the wall and pile row distributes according to their rigidity [6]. Such decision is incorrect because it does not consider the actual ground pressure on the construction elements.

Shielding effect occurs not in all types of grounds. Practice shows that shielding effect of basement in weak grounds is poorly effective. As weak grounds because of their properties closed to the viscous liquid it causes a conventional flawover (dulling) of piles by soil ground. In weak clayey grounds it is recommended not to take into account Shielding effect.

It is assumed that the intensity of lateral pressure on sheet piles consists of silage pressure σ_{sil} in zone 1 and additional pressure from the shielding pile row σ_{sh} (Fig. 1c) [25]. It is equal to the difference between external pressure in zone 2 and reverse silage pressure on shielding ground plane in zone 1. The value of this difference in turn depends on the pile step and distance C to the front wall (Fig. 1c).

$$\sigma_{sp} = \sigma_{sil} + \sigma_{sh}\ . \tag{3}$$

The authors propose to determine the pressure on sheet piles from zone 2 considering the influence of pile step and the distance C to the front wall by means of distribution coefficient K_d = 0÷1.

$$\sigma_{sh} = K_d\, \sigma_r = (n - d)/n)(2l_1/l)\sigma_r\,, \tag{4}$$

where $\sigma_r = (\sigma - \sigma_{sil})$ – resulting pressure in a plane passing through the shielding piles; $\sigma = (q + \gamma h)\lambda_a$ – lateral pressure on pile row (Fig. 1*c*); *n* – piles' step; $l = b + 2c{\cdot}tg\varphi$ – the area of distribution of resulting load between piles in section b on the sheet pile wall; $l_1 = c{\cdot}\mathrm{tg}\varphi$ – the area of distribution of resulting load behind piles on the sheet pile wall, (Fig. 1*c*); *d* – diameter of shielding pile.

The influence of pile row step. If $d \to n$ we have $b \to 0$ multiplier $K_p \to 0$ the pressure is completely taken up by pile row, so $\sigma_{sh} \to 0$ (Fig.1c). If $d \to 0$ silage pressure tends to zero $\sigma_{sil} \to 0$, so $\sigma_{sp} \to \sigma$;

The influence of pile row distance from wall. In case $c \to 0$ silage pressure tends to zero, $\sigma_{sh} \to 0$. If $c \to \infty$, $l \to \infty$ respectively, so $\sigma_{sh} \to 0$;

The nonlinear influence of zone 1 width and piles' step at the value of silage pressure intensity. Vertical and inclined piles slope sliding wedge of the ground behind the wall and take upon itself a part of lateral pressure of the ground (shield) decreasing the ground pressure on the front wall. However, the intensity of shielding of triangular sliding wedge`s section is uneven from its width. That is why it is nonlinear.

Consider the character of the phenomena occurring in the ground while slotting sliding wedge by pile row which accepted for simplicity continuous. If pile row is absent, sliding wedge CGE which creates lateral earth pressure CGD impacting on the wall (Fig. 2a). The appearance of pile row in sliding wedge decreases sliding wedge by the amount ABE and accepts pressure diagram ABD. This led to a decrease of diagrams of pressure on the wall by the amount ABD respectively.

Expressing the resultant of shielded pressure on the wall E_a in the form of a nonlinear function of the distance to pile row:

$$E_{eq} = E_a (x/L)^a , \tag{3}$$

where L – length of sliding wedge; a ≤1 – exponent.

If x = 0 shielded pressure on the wall E_{sh} = 0, that means that the pressure is completely taken up by pile row, but if $x \geq L$ the influence of pile row at the front wall is absent. The dependence of shielded pressure resultant from distance between pile row and the wall x in case n = 0.5 in relative magnitudes is given in Figure 2b.

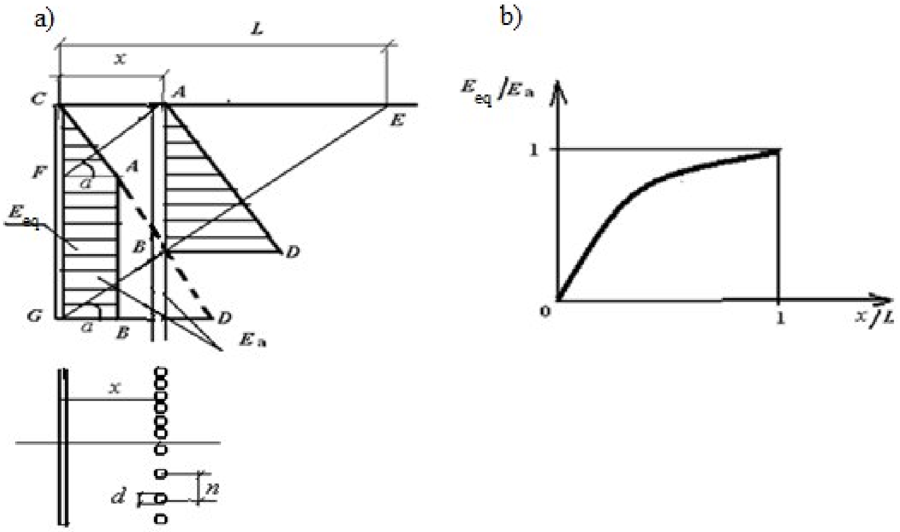

Figure 2. (a) Scheme for accounting of shielding; (b) The dependence of shielded pressure resultant from distance between pile row and the wall

Example 1. Determine the resultant of shielded pressure on the wall if x/L is equal 0.1; 0.2; 0.3.

Using the expression for E_{eq} if n =0.5 we obtain E_{eq1} = 0.31E_a; E_{eq2} = 0.45E_a; E_{eq3} = 0.54E_a, respectively. So the optimal arrangement of shielding pile row should not exceed 0.1 x/L.

Consider pile row with diameter d driven into the ground with step n. In this case the resultant of shielded pressure on the wall E_{sh} is represented as a function of distance to pile row which also depends on the pile diameter and distance between them (Fig. 2a).

$$E_{sh} = E_a (x/L)^a + [E_a - E_a (x/L)^n] \, [(n - d)/n]^m , \tag{6}$$

where E_a – resultant of the active pressure excluding shielding; $m \leq 1$ – exponent.

Exponents a and m require experimental researches.

If d = 0 the shielding effect is absent, so $E_{sh} = E_a$, but if $d = n$ there is continuous pile row, respectively.

The first term of equation for E_{sh} is a value of the resultant of silage pressure on the sheet pile wall depending on silage width *x.* The second term of the equation gives the component of redistributed pressure on the wall from the pressure on pile row considering reverse silage pressure.

Preselection of sheet pile type and driving depth of covered type of sheet pile wharf is accepted as for conventional sheet pile wharf. According to Russian Construction Norms and Regulations SNiP 2.02.03-85 the driving depth of the piles is specified and the lateral pressure on sheet pile wharf elements is determined according to the Table 1.

The calculation model of covered type of sheet pile wharf is a rack-mount cantilever frame. In the frame the rear pillar with relieving platform is fastened to rear part of the front wall by hinged support (Fig. 3a) or fastened to rigid supports respectively (Fig. 3b). Additionally the frame front part has hinged support displaying the device of anchor nodes (Fig. 3).

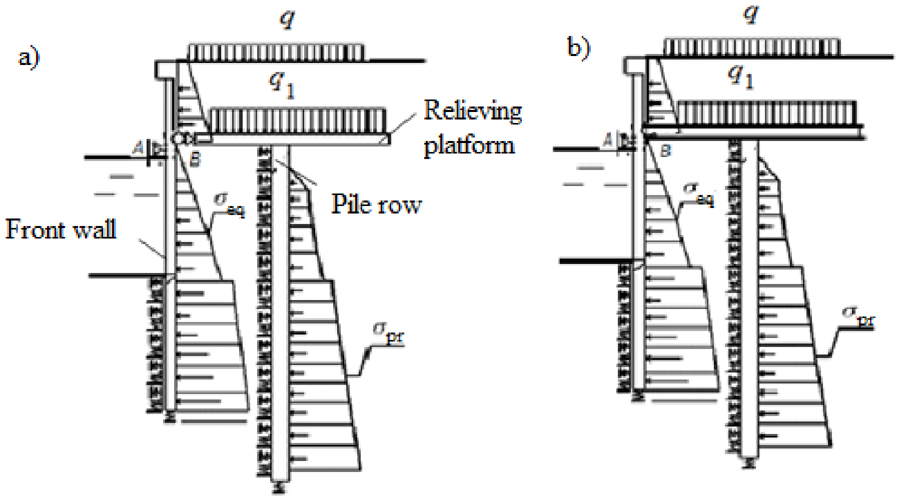

Figure 3. The calculation model of covered type of sheet pile wharf is in the form of: (a) Rack-mount frame with hinged anchor support A in its outer part and hinged support B in its inner part of front pillar to the top of frame; (b) Rack-mount frame with hinged anchor support A in its outer part and rigid supports B in its inner part of front pillar to the top of frame

Foundation modulus for sheet pile wall and piles. The front wall is partially dipped into foundation soil, pile row is totally in it. Lower ends of rack-mount frame have a hinged sliding supports. Ground behavior is described by variable foundation modulus:

$$K_f = y\ K\ K_a \cdot K_{pl} \text{ – for sheet pile wall;} \tag{7}$$

$$K_f = a \cdot K_{fr}\ K \cdot y/\gamma_c \text{ – for piles,} \tag{8}$$

where y – the depth of the considered point; K (kN/m^4) – coefficient of proportionality [7], K_{fr} – friability coefficient (using for piles which submerge into backfill ground); K_a – anisotropy coefficient: for granular soil is 1 and for clayey soil – 0.7 ÷ 0.8, K_{pl} = 0.6 ÷ 0.8 – coefficient considering the plastic properties of foundation in front of the wall for upper ground layer 1÷2 m; a = 0.8 – coefficient taking into account the influence of neighboring piles; γ_c = 3 – coefficient of work conditions. The iteration method implemented in the SCAD-Cross can be used to clarify K_f [26].

The main provisions of suggested calculation use the idea of engineering multi-purpose method in the form of different combinations of construction members [17]. This combination taking into account different conditions of element fixing uses local and structural strains. The calculation model of covered type of sheet pile wharf is accepted in the form of substandard rack-mount cantilever frame. The rear pillar of frame with relieving platform hingedly fastened to rear part of the front wall. In its turn the front part of frame also has hinged support. This support corresponds to anchoring of wall.

Initial data in form loads on the covered type of sheet pile wharf (Fig. 3):

A. Payload on the territory q;

B. The load from the backfill ground q_1.

C. Dead load of relieving platform;

D. The resulting horizontal load of ground on pile row and on the front wall.

To solve this task for the covered type of sheet pile wharf in the SCAD program it is necessary:

1. Introduce the elements of construction, select nodes (according to coordinates), set the connections at the ends of the front wall and piles, assign necessary rigidness to elements.

2. Introduce initial data in the program in the form of loads on the covered type of sheet pile wharf

3. Introduce in each element parts the schemes needed according to supporting conditions of the horizontal foundation modulus (ground stiffness) amount.

4. Set up a basic load combination and make a linear analysis.

5. In graphical analysis print out the results in the form of: bending moment diagram M, shearing force diagram Q, axial force diagram N, reactive soil pressure R and the scheme of construction deformation.

Example 2 given below is performed in accordance with the recommendations of the work [27].

Table 1. Lateral pressure on the shielded pile row and the front wall

Silage pressure on the pile row (The diagram from the left, (Fig. 4a))	**Resulting pressure on the pile row (See the diagram right to solid line), (Fig. 4a)**	**Resulting pressure on the front wall (Fig. 4b)**
$\sigma_{sil,1} = 0$; $\sigma_{sil,2} = 0$; $\sigma_{sil,3} = \gamma_1 h_2 \lambda_{a,1}$; $\sigma_{sil,4} = (\gamma_1 h_2 + \gamma_2 h_3)\, \lambda_{a,1}$; $\sigma_{sil,5} = \sigma_{sil,4}$; $\grave{\sigma}_{sil,5} = \sigma_{sil,5}\, \lambda_{a,2}/\lambda_{a,1} - \sigma_{ac}$; $\sigma_{sil,6} = \grave{\sigma}_{sil,5}$; Angle a = (45°-0.5φ_1) ; σ_{sh} – pressure on the pile row by backfill soil ; σ_{sil} – reverse silage pressure on the pile ro from water side.	1. $\sigma_{sh,1} = 0$; 2. $\sigma_{sh,2} = 0$; 3. $\sigma_{sh,3} = \gamma_1 h_2 \lambda_{a,1} - \sigma_{sil} = 0$; 4. $\sigma_{sh,3}` = [q + \gamma_1 (h_1 + h_2) + \gamma_2\, h`_3]\, \lambda_{a,1} - \sigma`_{sil,3}$ ; 5. $\sigma_{sh,5} = [q + \gamma_1 (h_1 + h_2) + \gamma_2 (h`_3 + h_4)] \lambda_{a,1} - \sigma_{sil}$; 6. $\grave{\sigma}_{sh,5} = \sigma_{sh,5}\, \lambda_{a,2}/\lambda_{a,1} - \sigma_{ac} - \sigma_{sil,5}$; $\sigma_{ac} = 2c \cdot \mathrm{tg}\,(45° - 0.5\varphi_2)$ – friction value; 7. $\sigma_{sh,6} = (\grave{\sigma}_{sh,5} + \gamma_2 h_5) \lambda_{a,2}$; $\lambda_{a,i}$ – coefficient of active earth pressure considering the force of friction on the wall;	1. $\sigma_{sh,1} = q_1 \lambda_{a,1}$; 2. $\sigma_{sh,2} = (q_1 + \gamma_1 h_1) \lambda_{a,1}$; $\grave{\sigma}_{э,2} = 0$; 3. $\sigma_3 = \sigma_{sil,3} + K_{al} \sigma_{sh,3}$; 4. $\sigma_3` = \grave{\sigma}_{sil,3`} + K_{al} \sigma_{sh,3`}$; 5. $\sigma_5 = \sigma_{sil,5} + K_{al} \sigma_{sh,3}$; 6. $\grave{\sigma}_5 = \sigma_{sil,5} + K_{al}(\sigma_{sh,5} - -\sigma_{ca})$; 7. $\sigma_{,6} = \sigma_{sil,6} + K_{al}(\sigma_{sh,6} - \sigma_{ca})$; K_{al} – coefficient of distribution of additional load on the front wall from a pile row.

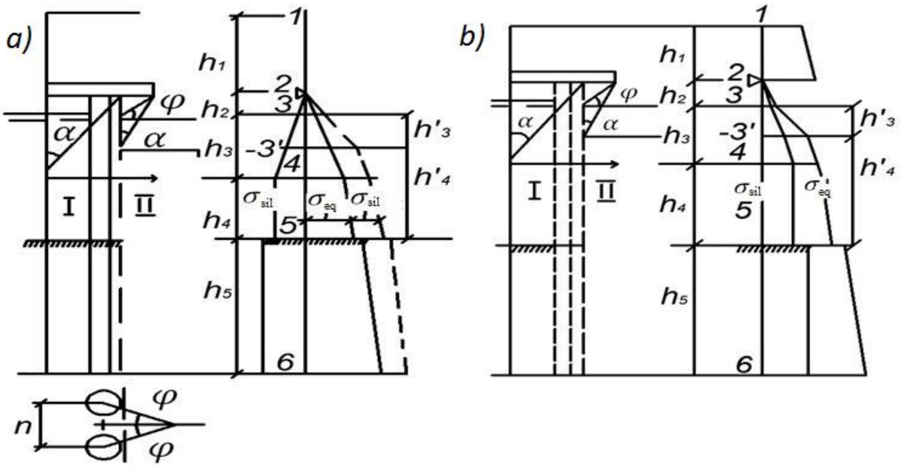

Figure 6. Scheme of loads on pile row: a) Silage pressure on the pile row (see the diagram from the left) and resulting pressure on the pile row (see the diagram right to solid line); b) Resulting pressure on the front wall

Results

Example 2. Calculate the covered type of sheet pile wharf with height of 13.5 m made of pipe pile with diameter 720 mm. The pile row consists of mantle pipes with diameter 800 mm which are immersed in increments of 2.0 m. The relieving platform 4.0 m-width is located on the pipes. The backfill ground is medium sand $\varphi = 31^0$, foundation soil - semisolid loam $I_L = 0.4$, φ = 27°, c = 8 кPa.

The calculation results of the covered type of sheet pile wharf in the SCAD program are given on Figure 5.

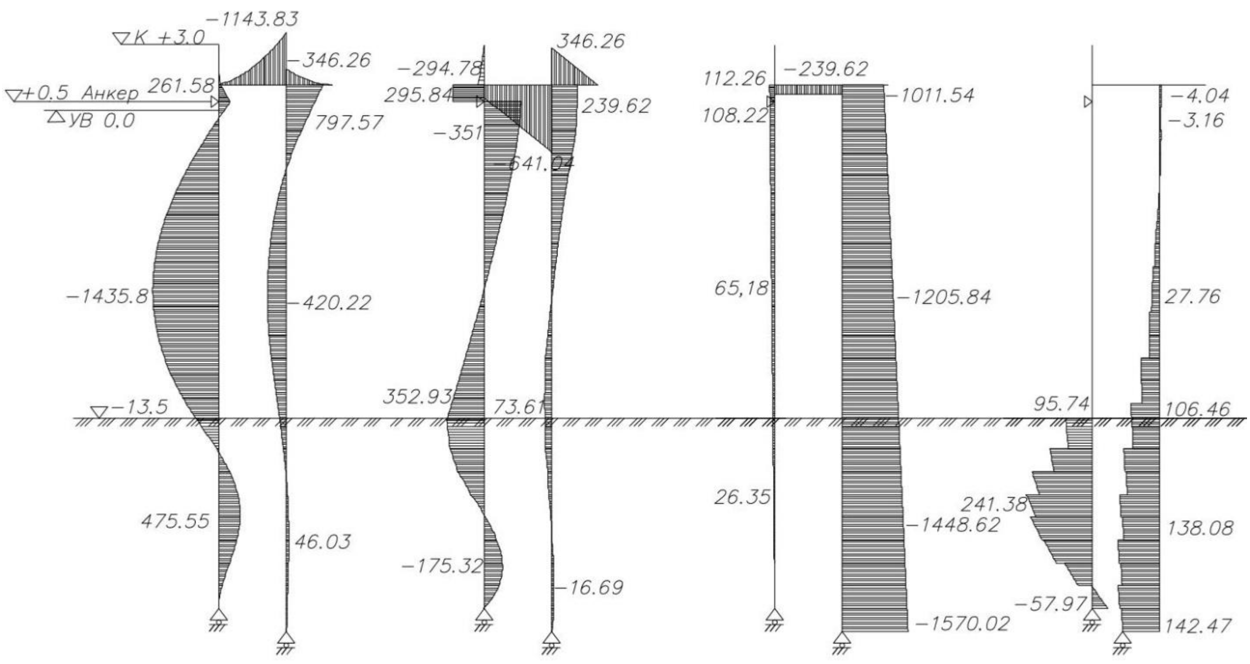

Figure 5. The diagram of internal forces in the elements of the covered type of sheet pile wharf. Conventional signs: a) Bending moment diagram; b) Shearing force diagram; c) Axial force diagram considering the weight of pipes and sand inside it; d) Diagrams of reactive soil pressure on the frame pillars.

The calculation results of the Example 2 are given in the Table 2.

Table 2. Comparative table of internal forces value in the covered type of sheet pile wharf

№	Name of the internal forces	The proposed method	The existing method [6]
1	Maximum bending moment in the pipe pile, kNm	1435.8	1250.0
2	Maximum bending moment in the pile row, kNm	470.22	750
3	Lateral forces, kN		
	The pipe pile	352.9	-
	The pile row	239.6	-
4	Axial force, kN		
	The pipe pile	112.26	-
	The pile row	1011.34	931.7
5	Maximum bending moment in the relieving platform, kNm	1143.83	1500.0
6	Maximum deflection in the pipe pile, mm	60.75	-

Discussion

The calculation of covered type of sheet pile wharf in the form of substandard rack-mount frame is an improved particular case of N.M. Gersevanov method in respect to the pile foundation grillage.

The article gives an engineering solution for definition of diagrams of lateral pressure on the front wall taking into account the redistribution of pressure on pile row.

The article gives a practical implementation of previously proposed engineering multi-purpose method for calculation of berths. The calculation model is proposed in the form of substandard rack-mount frame where resilient ground attachment is accepted instead of rack lower anchorage. This model uses stiffness characteristics of the ground in the form of variable foundation modulus.

Comparative variants of calculations of proposed and existing methods have shown a significant impact of ground deformation characteristics on berth elemental forces in proposed method in comparison with the existing method [6].

Conclusions

1. In the existing method a maximum bending moment in the pipe pile and pile row is determined by grapho-analytical method. Maximum bending moment in the pipe pile without shielding, M_{pp} = 2053.0 kNm.

2. Maximum axial force considering the increasing weight of pipe and soil in it 1570.0 kH.

The difference in bending moment diagram in the elements of covered type of sheet pile wharf in comparative Table 2 is connected with the fact that the deformation-free method satisfactorily describes the work only in flexible walls in dense soils base. In extra stiff wall according to the proposed method, the moment of anchorage appears inconspicuous.

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Смоленкова А.В., Коровкин В.С., Орлова Н.С., Рагулин К.Г., Кузина А.Д. Экранированный больверк в виде нестандартной стоечной рамы // Инженерно-строительный журнал. 2017. № 4(72). С. 3–11.

doi: 10.18720/MCE.72.2

Program Evaluation and Review Technique as the tool for time control

Метод анализа и оценки программ как механизм контроля сроков

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Key words: project control; time control; investment and construction project; probabilistic modeling; Program Evaluation and Review Technique; activity chart

Ключевые слова: контроль проекта; контроль сроков; инвестиционно-строительный проект; вероятностное моделирование; метод анализа и оценки программ; сетевой график

Abstract. The free market economy comes to the life of the people and creates the new structure of the work relationships between the investors and builders. One of the most important thing of the investment and construction project is the meeting of the target date. The delays in the implementation of this project entail the material and image losses. But the building is a complex process, many factors such as delivery delays, adverse weather conditions, the correcting of low quality works, machinery and mechanisms breakdowns influence on it. During the planning stage it is unknown which of these situations may arise in the implementation process and disrupt the work schedule. That is why building projects require the strongest time control. This article presents one of the best ways to control the building process, which is named Program Evaluation and Review Technique (PERT). Although there were mentioned other variations of the time control methods, such as Graphical Evaluation and Review Technique (GERT) and method of the statistic modeling (Monte Carlo), the advantages of PERT and disadvantages of them are presented. It allows to control the expectancy of the erection and the probability of the project completion in time. This means that it allows manager to control the activity time of the project. The example of the PERT usage is represented with the comparison between the controlled project and the non-controlled project.

Аннотация. Рыночная экономика плотно вошла в нашу жизнь и создала новую структуру рабочих отношений между заказчиками и строителями. Одной из важнейших задач менеджера инвестиционно-строительного проекта является строительство объекта в установленный срок. Задержки в реализации таких проектов влекут за собой не только материальные, но и имиджевые (репутационные) потери. Однако строительство – это сложный составной процесс, на него могут повлиять многие факторы, такие как задержки поставок, неблагоприятные погодные условия, исправления некачественно выполненных работ, поломки машин и механизмов. На стадии планирования неизвестно, какая из этих ситуаций может возникнуть в процессе реализации проекта и нарушить график работ, поэтому строительные проекты требуют усиленного контроля сроков. В статье представлен один из лучших способов управления процессом строительства, который называется метод анализа и оценки программ (PERT). Также были рассмотрены и другие методы вероятностного моделирования, такие как Графический метод анализа (GERT) и метод статистических испытаний (Monte Carlo), проведен сравнительный анализ преимуществ и недостатков данных методов. Метод PERT позволяет контролировать ожидаемую продолжительность проекта, а также рассчитать вероятность его завершения в срок. Пример использования метода PERT для контроля проекта представлен сравнением проекта с контролируемым процессом строительства и полностью неконтролируемым.

Mishakova A.V., Vakhrushkina A.V., Anishchenko D.R., Tatarkina Y.A. Program Evaluation and Review Technique as the tool for time control. *Magazine of Civil Engineering*. 2017. No. 4. Pp. 12–19. doi: 10.18720/MCE.72.2.

Introduction

From the beginning of the free market economy there is the natural contest between companies, which are working in the different spheres of social life. Their rivalry increases the rate of any type of the construction such as residential or commercial property. The definition "property" came as the economical term not so long ago in Russia and nowadays it is still developing because of Russia history.

The property is the one of the most profitable investment in today life. Capital providers are concerned in the fastest time erection and the nearest facility completion deadline in case of the investment efficiency rate. The speed of the erection affects the start of the pay-off period which influences on the profit from this property [1]. The erection of the building to the deadline time needs to be controlled by the developer, not only on technology process and workmanship, but also of the time control [2, 3]. The cash flow of the residential property will begin almost immediately after the company receipts of a building permit. In this moment the building works have not already started so the sense of the facilities completion deadline has not such big influence on the result profits.

The importance of the time control could not be exaggerated during the national significant building such as stadiums for the FIFA World Cup 2018. The collapse of the facility completion deadline destructs the meaning of the idea realization and falls into disrepute country on the global stage.

The project success depends on the detailed working calendar plan and control system, which should monitor the progress of the erection [4, 5]. It could be realize with the periodical acquisition of the facts, the comparison of them with the planning data, the analysis of the results and making of the management decision. The effects of them must destruct the negative factors and allow the accomplishment of the project target [6].

This article describes the using of Program Evaluation and Review Technique (PERT) method as the tools for the time control of the building and evaluation of the investment project final cost [7].

PERT was created by the Navi's Special Project Office and connected with the Polaris-Submarine weapon System and Fleet Ballistic Missile. It was developed "to save time in achieving end-objectives" [8]. Graphical Evaluation and Review Technique (GERT) was developed in 1966 by Pritsker [9]. It is only a modification of PERT and allows following several different distribution, nevertheless it is not as spreading as PERT in case of its complexity to the computer adaptation.

Ameen developed special program which help to teach project management techniques for his students in 1987 [10]. Later, in 1991, Badiru makes another one simulator and called it STARC, which allows to determine the probability of the expiration into deadline [11, 12] "Additional authors which have studied various PERT problems via simulation include Kltngel (1966), Gray (1969), Burt (1971), Herbert (1979), Schonberger (1981), and Dodin (1984), and Kidd (1986)" [13].

This interest in the modeling allows a possible profit from PERT. Spending of the recourses, times and money is the reason of the economical losses, which could come to the stagnation of the whole region, if it touches something meaningful for this site.

The purpose of this study is to adapt the probabilistic modelling methods to control project time. The main objective is to consider the use of probabilistic models as a time-control mechanism through the PERT method.

Methods and Results

How it was mentioned earlier, there are a few methods of the project planning, which take into account the stochastic building characteristic. The best known of them are PERT, GERT and Monte Carlo method.

Program evaluation and review technique (PERT) is the method of analysis and program evaluation, which is based on the three activity time estimation. There are optimistic, pessimistic and prospective activity time estimations, which are made by experts [14].

Graphic evaluation and review technique (GERT) is a method of operation modelling. This method shows the variations of the project completion in case of special kind of the algorithms and connects with ending a few of the previous algorithms [15].

The method of the statistic modeling or Monte Carlo method is based on a large number of the non-connected realization, which is overlooked in the network model [16].

The methods do not have any practical profits for the time planning of the project, because during the building there is the divergence of the basic plan and there is no reason to use the old plan after it.

This article presents another way of the using of the probability distribution methods. This is method of the time control of the project on the example of PERT methods [17, 18]. A peculiarity of PERT is the list of all or the definite activity time probability for the counting of all project time [19, 20].

As it was said earlier, PERT uses three experts mentioned:

- Optimistic – activity couldn`t be completed faster than $t_{i\ opt}$
- Pessimistic – activity couldn`t be completed slowly than $t_{i\ pes}$
- Most likely (normal) – most likely time will take $t_{i\ norm}$

If there is three values, then it is possible to count the expected activity time t_{ie} with formula:

$$t_{ie} = \frac{t_{i\,opt}+4t_{i\,nor}+t_{i\,pess}}{6}, \quad (1)$$

where: $t_{i\ opt}$ – the minimum value, when it is take into account that every task meet the target time or is made earlier

$t_{i\ norm}$ – the time value, when it is take into account that everything is as usual

$t_{i\ pess}$ – the maximum value, when it is take into account that every task do not meet target time (excluding of the massive catastrophe)

The degree of indeterminacy of activity time estimate may be shown by the dispersion:

$$\sigma_i^2 = \left(\frac{t_{pess}-t_{opt}}{6}\right)^2. \quad (2)$$

PERT allows to get the normal dispersion of the project time planning probability, which mode is according to the expected activity time. The standard deviation of normal distribution curve should be calculate to find the probability of completition of the project in time, which is differing from the expected. It shows the stage of the indeterminacy for the whole project:

$$\sigma_{Te} = \sqrt{\sum \sigma_i^2}. \quad (3)$$

This formula takes into account only activity dispersions, which create the critical path.

According to the probability theory, the probability of the project accomplishment is in the range from T_e-σ_{Te} to T_e+σ_{Te} [21] equals to 68.27 %, the probability of the project accomplishment is in the range from T_e-3σ_{Te} to T_e+3σ_{Te} equals to 99.73 % (Fig.1).

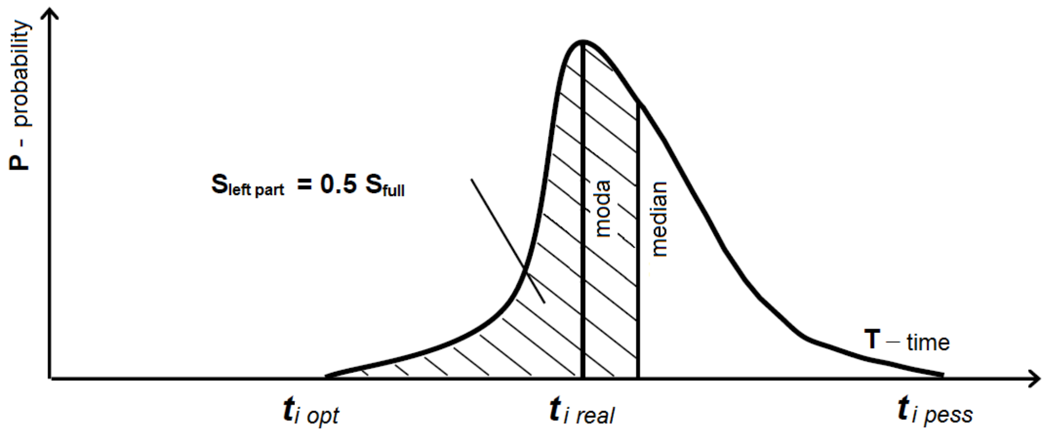

Figure 1. The frequency probability curve of the duration of each activity

In fact, the investors and builder are interested the probability of the project completion to the deadline, for example, to contract date T_{plan}. It could be found from the formula:

$$Z = \frac{T_{plan} - T_e}{\sigma_{Te}}, \qquad (4)$$

where: T_{plan} – planning time of the target date meeting;

T_e – expected activity time – the probability of the project implementation in expected time activity or faster equals to 0.5 (50 %). To count the expected activity time of the project it is necessary to define for all tasks the expected activity time t_{ie} as the target value.

In account to Z value, it is possible to find the probability of the project completion with using of the special tables, and express it in terms of per cent or unit fraction [22].

The using of PERT method gets the possibility to find the diapason of the task deadline, also it allows to make a decision about the probability of the end activity in time according to the task, which were done to the monitoring time [23].

As the example of the PERT using during the building process, we would consider the project, collateral to the activity chart (Fig.2)

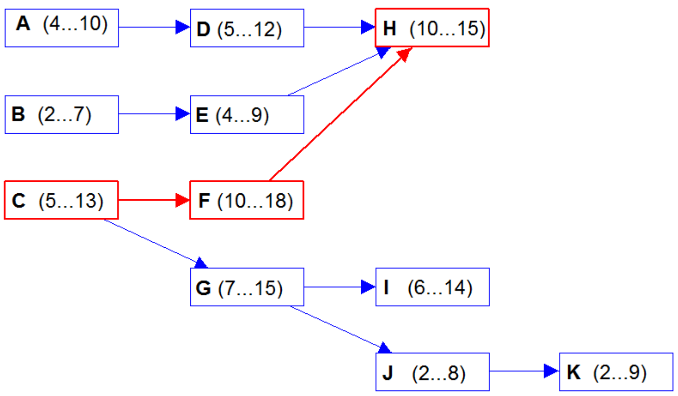

Figure 2. The activity chart

The list of the expert activity time estimations, the results of the expected activity time estimation and the full time of the task are showed in the Table 1.

Table 1. The time estimations of the tasks

Activity	**Time estimates**			**Expected time**
	Optimistic	Normal (Most likely)	Pessimistic	
A	4	6	10	6.3
B	2	4	7	4.2
C	5	8	13	8.3
D	5	8	12	8.2
E	4	6	9	6.2
F	10	13	18	13.3
G	7	11	15	11.0
H	10	12	15	12.2
I	6	9	14	9.3
J	2	3	8	3.7
K	2	5	9	5.2
Full duration of the project	25	33	46	33.8

In the beginning of the project, it easy to see that the full time of the project task is in the range of 25 to 46 days. This estimation is the main advantage of PERT method, because it allows to count probability of the project finishing to the negotiated deadline.

This probability could change in the time, so it is reasonable to compare two variants of the project. The first one shows the near-estimated time of each task, another one slows-up of the project.

Table 2. The probability of the project completion in time

	Status date	Actual value of the tasks	The estimation of the full time considering executed tasks			Expected	P %	Required time
			Opt.	Norm.	Pess.			
I variant	0	0	25.0	33.0	46.0	33.8	70	25-46
	10	4	28.0	33.0	41.0	33.5	75	18-31
	20	7	29.5	31.5	36.0	31.9	92	9.5-16
	30	11	30.0	30.0	30.0	30.0	99	0.0
II variant	0	0	25.0	33.0	46.0	33.8	70	25-46
	10	3	28.5	33.5	41.5	34.0	67	18.5-31.5
	20	6	32.5	34.5	41.5	35.3	41	12.5-21.5
	30	10	32.5	34.5	37.5	34.7	54	2.5-7.5

During the works, the project was monitored each 10 days in this example. The appraisal of the tasks and the comparing of the spending time with the mentioned estimation provide the possibility of the additional time evaluation. So there is new probability of the project finishing to negotiated deadline.

Also the trend of the activities completing must be tracked. The data base of the trends lets to trace the effects of the factors, which influence on the activity time and how much are their effects. In the future it makes a possibility to eliminate critical errors and to plan project with a glance to this trends.

The graph shows the increasing of the successful deadline of the first variant project because of the tasks, which are done in planned period. The probability curve locates nearby the targeted line. The curve of the second variant shows the fall of the probability in case of the activity delay. This mean there was not any profitable actions.

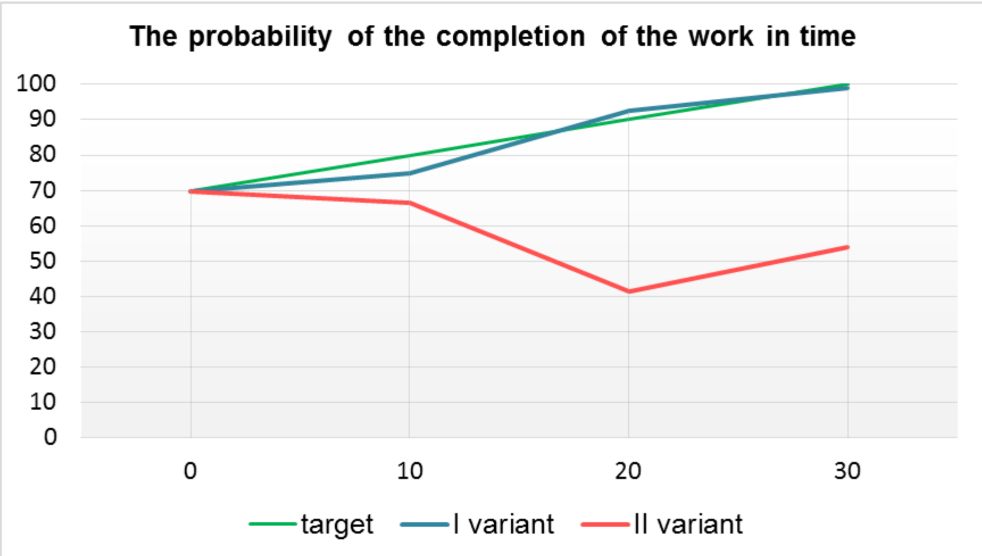

Figure 3. The probability of the completion of the project within the contract period

The project monitoring with some frequency make a possibility to find the time lag and to influence the situation. So if some actions will be done, the delay of the activity could be escaped. In contrast, the neglect of the time collapse reduces the successful deadline. This information provides the possibility of the work acceleration in case of the strict deadline or the extension of the object entry date.

Mishakova A.V., Vakhrushkina A.V., Anishchenko D.R., Tatarkina Y.A. Program Evaluation and Review Technique as the tool for time control. *Magazine of Civil Engineering*. 2017. No. 4. Pp. 12–19. doi: 10.18720/MCE.72.2.

It is critical to underscore that there is no possibility to influence on the finished task, so all actions must touch on the processing or non-started activity. The reducing of the project time is supplied with the acceleration of the critical task, the amplification of the workmen, vehicles and devices, the using of the progressive methods and highly-energetic equipment. Also some tasks could be excluded from the list of the task and made after the setting to work.

The timeout of the project could be estimated to the contract date with the penalty function or the function of “the lost benefits” or the comparing of the acceleration activity cost and the vindictive damages. The developer has to evaluate his actions. Are the vindictive damages comparing with the cost of the actions? Is it cheaper to pay penalty?

But these actions touch only the commercial project, which deadline does not have the global consequences and does not devalue the idea of the project. For example, there can be the national significant buildings such as Olympic objects in Sochi or the soccer world championship stadiums.

Discussion

There was considered PERT methods using as the mechanism of the time control. The method was presented on the example of the simple project with 11 tasks, but the project with the bigger scope of the task is reasonable to control with the MS Project program. Unfortunately, the authors exclude PERT method from the last version, and now it is usable only in the old version of 2003 and 2007 years [24] or with the help of the superstructure [25] or formulas in the new version.

The additional problem associated with the subcritical path, which might change the probability of the project finishing to the deadline [26]. It is possible if the scatter of the subcritical path activity duration as well as the dispersion of them is bigger than the similar critical task evaluation of the time. There is a problem to measure the probability of the activity transformation from the non-critical to the critical task and to measure the diapason of the float time for each activity.

It is widely thought that the accuracy of the networked model computation and the program analyze depends on the numbers of the activities. More than 30 tasks on one path guarantee the high level of the accuracy; otherwise, PERT method gives only the approximated time of duration of the project.

The reason of PERT method usage should to be examined according to the costs of the building rate of the growth and the possible penalty payment. [27, 28, 29] The sanction could to be much cheaper than the costs of the accelerative erection [30].

Conclusion

The application of PERT method does not make economic sense without the time control. To finish project in time, it is necessary to count the probability of the completion of the project within the contract period not only in the beginning of the erection, but during the project realization. It is should be done in case of the unexpected circumstances that could influence on the deadline of the project. During the estimation of the situation it requests the analysis of the task, which was done before, and there is no possibility to influence on them, so all actions appertain to the future activities. The using of PERT method allows to the recounting of the possible times of the remaining task and takes into account the optimistic and pessimistic time estimation.

PERT method could be used with the Microsoft Project program; this is the advantages in case of the practical needs.

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doi: 10.18720/MCE.72.3

Effect of the supplement based on calcium hydrosilicates on the resistance of lime coatings

Влияние добавки на основе гидросиликатов кальция на стойкость известковых покрытий

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Key words: supplement; calcium hydrosilicates; dry mixes; diatomite

Ключевые слова: добавка; гидросиликаты кальция; сухие строительные смеси; диатомит

Abstract. It was proposed to use synthesized calcium hydrosilicates in lime finishing dry mixes as a modifying supplement. The effect of substances containing amorphous silica, which are used in the synthesis, on the activity of the modifying supplement was established. The effect of the synthesis mode of supplement on the structure formation of lime compositions was illustrated. It was found that the injection of supplements of hydrosilicates accelerates the increase of mechanical strength. The efficiency of the using of modifying supplements of amorphous silica, such as diatomite, in the synthesis was shown. The evaluation of hydrophysical properties of coatings based on lime dry mix with the using of supplements synthesized in the presence of calcium hydrosilicates was provided. It was shown that the injection of supplements of hydrosilicates accelerates the increase of water resistance and frost resistance of coatings. It was found that the adhesion of the lime compositions with the supplement based on calcium hydrosilicates, synthesized in the presence of diatomite, is higher than that of the compositions with the addition of calcium hydrosilicates synthesized without diatomite.

Аннотация. Предложено применять в качестве модифицирующей добавки в известковых отделочных сухих строительных смесях синтезированные гидросиликаты кальция. Установлено влияние веществ, содержащих аморфный кремнезем, используемых при синтезе, на активность модифицирующей добавки. Показано влияние режима синтеза добавки на структурообразование известковых составов. Выявлено, что введение добавок гидросиликатов способствует ускорению набора механической прочности. Показана эффективность применения при синтезе модифицирующих добавок аморфного кремнезема, в частности, диатомита Инзенского месторождения. Приведена оценка гидрофизических свойств покрытий на основе известковой сухой строительной смеси с применением добавок, синтезированных в присутствии гидросиликатов кальция. Показано, что введение добавок гидросиликатов способствует повышению водостойкости и морозостойкости покрытий. Установлено, что адгезия известковых составов с добавкой на основе гидросиликатов кальция, синтезированной в присутствии диатомита, выше чем у составов с добавкой гидросиликатов кальция, синтезированной без диатомита.

Introduction

For the restoration and finishing of the walls of buildings and structures, lime compositions, including dry mixes, are used. Coatings based on lime dry mixes have high values of vapour permeability and bioresistance, which allow using them as traditional mixes for finishing architectural monuments and renovating structures in places of historical buildings. However, lime mixes are characterized by low

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values of strength and durability, which limiting their using. To avoid the destruction of lime dry mixes, modifying supplements are injected to their recipe [1, 2].

Previous studies has confirmed the efficiency of the injection of mineral supplements, based on synthesized calcium hydrosilicates, in the recipe of finishing lime dry mixes, that increase resistance of lime coatings, [3, 4, 5].

Synthesis of supplement was the deposition of soluble glass in the presence of calcium chloride solution [6, 7, 8]. Lime compositions with the using of synthesized calcium hydrosilicates (CHS) form coatings increased water resistance (softening coefficient is K_{soft} = 0.61).

In addition, it has been found that the disadvantages of the above methods for the obtaining of hydrosilicate supplements are the formation of high basic and low basic hydrosilicates during mixing, where the high basic hydrosilicates predominate to a greater degree. It has been established that high basic calcium hydrosilicates have less water resistance and strength compared to low basic hydrosilicates, so it is important to obtain a hydrosilicate supplement, which will contain more low basic calcium hydrosilicates. [9, 10, 11].

The aim of the work is to develop a technology of the synthesis of a supplement containing low basic calcium hydrosilicates.

To achieve this aim, it is necessary to solve the following:

– to reveal patterns of structure formation of lime compositions in the presence of supplements based on calcium hydrosilicates synthesized in the presence of diatomite;

– to develop a recipe for a lime dry mix, coatings based on it have increased operational resistance;

– to establish technological and operational properties of lime dry mixes and coatings based on it.

Given that low basic calcium hydrosilicates have higher strength, compounds, containing amorphous silica, such as diatomite of Inza deposit, were used in synthesis of supplement in further research [12, 13, 14].

Methods

Two CHS synthesis modes were used in work:

- mode 1 –the deposition in the presence of 15% $CaCl_2$ solution in an amount of 50% by mass of soluble glass;
- mode 2 –the deposition in the presence of 10% $CaCl_2$ solution in an amount of 50% by mass of soluble glass with the addition of diatomite, wherein the ratio of liquid:solid phase (L:S) was (L:S) = 1:2.

The resulting sediment was dried at a temperature of 100°C.

The synthesized supplements were used to develop the recipe of lime dry mix [15, 16].

The first synthesis mode was used in accordance with the data given in [17]

The developed recipe includes lime powder of ClassII, quartz sand 80 % of fraction 0.63–0.315, 20 % of fraction 0.315–0.14. Plasticizer Kratasol PFM in an amount of 1 % by mass of lime, redispersible powder Neolith P-4400 in an amount of 0.3 % by mass of lime, hydrophobisator Zincum-5 in an amount of 0.5 % by mass of lime were injected in the recipe to control structural and mechanical characteristics of the lime composite. The content of CHS supplement was 30 % by mass of lime. Mixes were prepared with water: lime ratio W/L = 1.2.

Water holding capacity of finishing composition was determined in accordance with Russian State Standard GOST 5802-86 "Mortars. Test methods". Before the test, 10 sheets of filter paper of 150 × 150 mm each were weighed with an error of up to 0.1 g, placed on a glass plate of 150 × 150 mm and 5 mm thick, one layer of cheesecloth was put on the filter paper, a metal ring with an inside diameter 100 mm and a height of 12 mm was installed on top, and the whole installation was weighed again. Then the thoroughly mixed solution was put flush with the edges of the metal ring, weighed and allowed to stand for 10 minutes. After that, the metal ring with the solution was carefully removed along with the cheesecloth, and the filter paper was weighed with an error of up to 0.1 g.

The water- holding capacity of the finishing mixt was determined by the percentage of water content in the sample before and after the experiment, according to the formula:

$$V = 100 - \left(\frac{m_2 - m_1}{m_4 - m_2}\right) * 100, \quad (1)$$

where m_1 – mass of filter paper before the test, g; m_2 – mass of filter paper after the test, g; m_3 – mass of installation without solution mix, g; m_4 – mass of the installation with a solution mix, g.

The water holding capacity of the dry mix was determined twice for each sample and was calculated as the arithmetic mean of the results of the two determinations, differing by no more than 20 % from the smaller value.

To evaluate the operational resistance of coatings based on lime dry mix, frost resistance tests were carried out by cyclic freezing and melting of the finishing layer applied to the cement-sand base after 28 days of air-dry hardening. Appearance of coatings was evaluated according to Russian State Standard GOST 6992-68 "Lacquers and paints. Method for determination of coating weather-resistance". As the "failure", the state of the coating was taken, estimated of III.4 points (Table 3).

The kinetics of water absorption of coatings based on the dry mix was determined in accordance with Russian State Standard GOST 5802-86 "Mortars. Test methods".

Samples, previously dried to constant mass, were placed in a container filled with water. The water temperature in the container was (20 ± 2) °C. Samples were weighed at interval of 1 hour on conventional scales with an error of not more than 0.1 %. When weighing, samples taken from water are wiped with a damp cloth previously. The mass of water that leaked from the sample pores to the weighing pan was included in the mass of the saturated sample. The test was carried out until the results of two consecutive weighings differed by no more than 0.1 %. The water absorption of a single sample by weight W_m in percent was determined by the formula:

$$W_m = \frac{m_2 - m_1}{m_1} * 100\%, \quad (2)$$

where m_1 – mass of the dried sample, g; m_2 – mass of the water-saturated sample, g.

The ultimate compression strength of samples was determined in accordance with Russian State Standard GOST 5802-86 "Mortars. Test methods". As a equipment for testing the compression strength of samples, a test machine of the type "IR 5057-50" was used. Depending on the type of used power sensor of "IR 5057-50", the force measuring range was from 50 to 50000 N with an accuracy of 1 N (0.1 kgs). Built-in cross-bar speed controllers allow to set the speed of application of the load from 1 to 100 mm/min (in terms of displacement). The compression strength (MPa) of the samples was determined by the formula:

$$R_{\text{com}} = \frac{P}{F}, \quad (3)$$

where P – destructive force, N; F – cross-sectional area of the sample before the test, m^2.

Results and Discussion

It was found that after 28 days of air-dry hardening the compression strength R_{com} of the lime samples with supplement based on CHS, synthesized by the 2nd mode, is higher and is R_{com} = 5.5 MPa, while the compression strength of the lime samples with supplement based on CHS, synthesized by the 1st mode, is R_{com}= 2.86 MPa. Compression strength of control sample is R_{com}= 1.475 MPa.

Frost resistance was tested by cyclic freezing and melting of the finishing coat, applied to the cement-sand base, after 28 days of air-dry hardening to evaluate the operational resistance of coatings, based on lime mix. Evaluation of the appearance of the coatings was carried out according to Russian State Standard GOST 6992-68 "Lacquers and paints. Method for determination of weather-resistance of coatings". State of coating was assumed as the "failure", if it estimated at III.4 points (Table 1).It was found that the lime samples with supplement, based on CHS, have passed 35 cycles of the test. While the state of the coating after 35 test cycles is estimated at V.5 points that corresponds to the state of coating with loss of gloss to 5 %, with a slight change in color and the absence of blushing, bronzing, dirt retention, peeling, cracking, bubbling.

The results of the studies showed that the supplement based on calcium hydrosilicate has high activity, which is determined by the value of solubility in a 20 % KOH solution [18].

Table 1. Impact of the synthesis mode on the activity of the synthesized calcium hydrosilicate

Supplement	Solubility M, %	Activity A
Control composition	65	260
Lime composition with supplement, based on calcium hydrosilicates synthesized by 1st mode	68	350
Lime composition with supplement, based on calcium hydrosilicates synthesized by 2nd mode	70	370
Diatomite	61	299

The injection of a supplement based on calcium hydrosilicates, synthesized in the presence of diatomite, in the lime composition contibutes to an increase in water holding capacity. Table 2 shows the values of water holding capacity of lime samples.

Table 2.Water-holding capacity of the lime composites

Composition	Water-holding capacity, %
Control composition	95.5
Lime composition with supplement, based on calcium hydrosilicates synthesized by 1st mode	97.2
Lime composition with supplement, based on calcium hydrosilicates synthesized by 2nd mode	97.9

Thus, the water holding capacity of the composition based on the supplement synthesized in the presence of diatomite is 97.9 %, while of the composition based on the supplement synthesized without the use of diatomite – 97.2 %. The water holding capacity of the control composition is 95.5 %.

From the experimental data (Table 2) it follows that lime compositions with additives of calcium hydrosilicates are characterized by a sufficient water holding capacity of 97.2–97.9 %.

It was found that there is a difference in the state of coatings after 24 cycles of freezing and melting. Thus, state of the coatings, based on composition with CHS (1st synthesis mode), was estimated at V. 6 points (Table 3), and state of the coatings, based on composition with CHS (2nd synthesis mode), was estimated at V. 7 points after 30 cycles (Table 3).

Table3. Quality of the appearance of the coatings

Composition	Number of cycles	Points
Lime composition with supplement based on calcium hydrosilicates, synthesized on 1stmode	before test	V. 8
	6	V. 8
	12	V. 8
	18	V. 7
	24	V. 6
	30	V. 6
	35	V. 5
	40	IV. 4
Lime composition with supplement based on calcium hydrosilicates, synthesized on 2nd mode	before test	V. 8
	6	V. 8
	12	V. 8
	18	V. 7
	24	V. 7
	30	V. 6
	35	V. 5
	40	V.4
	45	IV. 4

Higher resistance to cyclic freezing and melting of the coatings based on composition with CHS (2nd synthesis mode) is due to a change of the porous structure, in our view. It was found that the injection of supplements, based on CHS, in the recipe of lime compositions reduces porosity. Thus, porosity of the samples with supplement, based on CHS synthesized by 1st mode, was estimated at P = 29.7 % (Table 4), and porosity of the samples with supplement, based on CHS synthesized by 2nd mode, was estimated at P = 26.7 % (Table 4).

Table 4. Porosity of the lime composites

Composition	**Porosity, %**		
	P_{cl}	P_{op}	P_{ov}
Control composition	7.1	28	35.1
Lime composition with supplement, based on calcium hydrosilicates synthesized by 1st mode	5.4	24,3	29.7
Lime composition with supplement, based on calcium hydrosilicates synthesized by 2nd mode	4.5	22.2	26.7

Adhesion of the lime samples aged 28 days of hardening with a supplement based on calcium hydrosilicates, synthesized in the presence of the diatomite, is R_{adg} = 0.89 MPa, while adhesion of the samples with the addition of CHS, synthesized without diatomite, is R_{adg} = 0.8 MPa [19, 20].

It was found that lime coatings based compositions with the addition of calcium hydrosilicates are characterized by higher water resistance. Thus, softening coefficient of lime samples with the supplement, based on CHS synthesized by the 1st mode, is K_{soft} = 0.61, while softening coefficient of lime samples with the supplement, based on CHS synthesized by the 2nd mode, is K_{soft} = 0.73. Softening coefficient of the control samples is K_{soft} = 0.29.

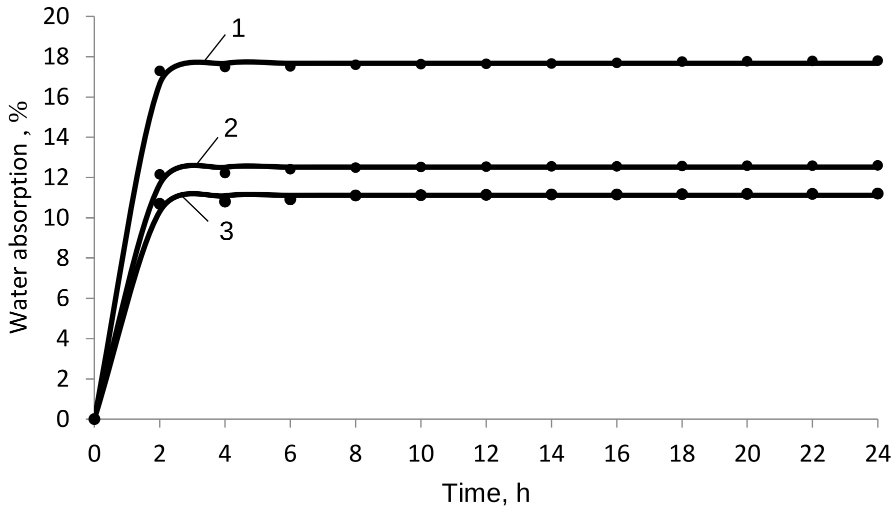

Figure 1. Water absorption by mass of lime composites:
1 – control composition with lime binder;
2 – composition with supplement, based on CHS synthesized by the 1st mode;
3 – composition with supplement, based on CHS synthesized by the 2nd mode

Figure 1 shows the curves of water absorption by mass of lime composites. Approximation of data was performed using software CurveExpert 1.3. Based on the results of approximation and experimental studies (Figure 1), it follows that lime samples with supplement, based on CHS synthesized by the 2nd mode, have lower water absorption than lime samples with supplement, based on CHS synthesized by the 1st mode.

The curves shown in Figure 1 were described by the exponential equation

$$y = a\,(1 - e^{-bx}), \tag{4}$$

where *a* – constant, taking into account the highest possible water absorption; *b* – water absorption rate constant; *x* – time.

The values of the constants *a* and *b* shown in Table 5.

Table 5. The values of the constants of water absorption equation

Composition	***a***	***b***
Control composition	17.671	1.902
Lime composition with supplement, based on calcium hydrosilicates synthesized by 1st mode	12.517	1.754
Lime composition with supplement, based on calcium hydrosilicates synthesized by 2nd mode	11.117	1.512

The calculation results have shown that the water absorption rate constant of the samples based on compositions with addition of CHS is significantly lower, in the range of 1.512 to 1.754 hour^{-1}.

Compression strength of lime samples aged 28 days air-dry hardening has been estimated. For comparison, lime samples were made only using diatomite in an amount of 30% by weight of lime.

It is found that the compression strength lime samples made with using calcium hydrosilicate supplement synthesized without diatomite is 4.7 MPa (Table 6), while at the lime samples made with using calcium hydrosilicate supplement synthesized in the presence of diatomite – 7.59 MPa (Table 6). Compression strength of control mix is 2.12 MPa.

Table 6. Compression strength of lime samples

Supplement	**Compression strength, [MPa]**
The control mix (no supplement)	2.12
Lime composition with supplement, based on calcium hydrosilicates synthesized by 1st mode	4.7
Lime composition with supplement, based on calcium hydrosilicates synthesized by 2nd mode	7.59
Diatomite	3.25

Table 7 presents the technological and operational properties of the finishing composition and coatings based on it, which were based on the developed dry mix and the prototype composition.

Table 7. Technological and operational properties of the finishing composition

Parameter name	**Parameter value for developed composition**	**Prototype**
Compression strength, MPa	5.5	2.5
Adhesion strength, MPa	0.89	0.7
Frost resistance, cycles	35	35
Water-holding capacity, %	97.9	97
Water absorption, %	10.15	12
Water resistance	0.73	-
Shrinkage strain, %	0.024	-
Water vapour permeation µ, mg/m·t·Pa	0.049	0.01
Viability, h	1.5	2-3
Presence of cracks due to shrinkage	no cracks	no cracks

The technological and operational properties of the developed dry mix were compared with the properties of the lime plaster mix "Kreps Antique", produced by the company "Kriks".

Thus, coatings based on the developed dry mix have higher operational properties such as compression strength, adhesion strength, frost resistance, etc.

Conclusions

The possibility of increasing the resistance of coatings by applying a supplement based on calcium hydrosilicates synthesized in the presence of diatomite, has been substantiated, which reduces the total porosity, accelerates the curing of coatings, increases the strength and water resistance due to the formation of low basic calcium hydrosilicates.

It was found that the supplement based on hydrosilicates synthesized in the presence of diatomite, is characterized by high activity, A = 370 mg / g.

It has been established that the injection of a calcium hydrosilicate supplement, synthesized in the presence of diatomite, in composition of the dry mix contibutes to accelerate in curing of the coatings.

Thus, compositions with the supplements, based on CHS synthesized in the presence of diatomite, form coatings with improved water resistance and resistance to cyclic freezing and melting. Lime mixes with synthesized supplement are characterized by high compression strength, vitality of 3–4 hours, adhesive strength of 0.9–1.1 MPa.

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doi: 10.18720/MCE.72.4

Buildings quasi-stationary thermal behavior

Квазистационарные температурные режимы ограждающих конструкций

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Key words: enclosure structure; thermal stability; energy-efficiency; average temperature; construction materials

Ключевые слова: ограждающие конструкции; теплоустойчивость; энергоэффективность; средняя температура; температурный режим; температурная волна

Abstract. The typical building constructions absorb the temperature wave caused temperature fluctuations of external air. It means this building construction has a thermal stability. Therefore, there is a reserve for warmth accumulation which can be used for decreasing of thermal losses. The developed mathematical model of temperature distribution in an enclosure structure allows estimating the cooling velocity of assorted designs of enclosure structures. And it shows the time which it is possible to turn off heating during the non-working period with maintenance required temperature condition in working period. This method allows reducing losses of heat energy considerably. It gives the chance to perform the optimum choice of periodic schedules of heating. Cost efficiency of implementation of the periodic mode of heating of the building is proved.

Аннотация. Ограждающие конструкции поглощают температурную волну, вызываемую температурными колебания наружного воздуха, т.е. конструкция обладает аккумулирующей способностью. В связи с этим возникает резерв для управления потоками теплоты, которые можно использовать для снижения тепловых потерь. Данная работа посвящена решению проблемы максимально эффективного использования энергетических ресурсов, расходуемых на отопление здания. Были выведены зависимости для определения распространения температурной волны в стене. Разработанная модель позволяет оценить скорость остывания различных конструкций стен, а также время, на которое можно отключить отопление в нерабочий период. Время зависит от аккумуляционной способности стены. Данный метод, позволяет значительно сократить потери тепловой энергии. В работе приведены экономические и энергетические последствия предложенных решений по повышению энергоэффективности конструкции и обеспечению периодического режима отопления.

Introduction

The the most effective use of energy resources is one of the important tasks of state policy in the field of energy saving for economic recovery and worthy life of the population.

It is possible to attain the economy of heat energy by applying a periodic duty of heating system for the buildings functioning only in the afternoon (sports, administrative, educational buildings, etc.).

However, it is necessary to solve a problem about optimum control of periodic duty for getting the maximum effect.

The papers deal with international research activities in the field of climate specific building design. Various comfort and energy monitoring surveys of office buildings as well as residential buildings provide substantial information about the occupants' behaviour and their needs during specific situations under different outdoor climates. This information allows summarizing basic climate dependent design principles which architects should keep in mind during the early stages of the design process. It also helps to develop strategies aiming at reducing building energy demand and at the same time consider comfort aspects [1–5].

Tarasova D.S., Petritchenko M.R. Buildings quasi-stationary thermal behavior. *Magazine of Civil Engineering*. 2017. No. 4. Pp. 28–35. doi: 10.18720/MCE.72.4.

The papers present the results of the research into energy balance of enclosure walls depending on geometric characteristics and glazed areas of a building [6–9].

This paper show how the building automation systems (BAS) are a powerful tool for companies face some permanent or temporary changes that can occur in the surrounding environment, which can affect the welfare of users, increase the energy consumption and/or demand more financial investment to strengthen or to replace the actual systems to attend the needs of users [10–22].

However, all stated methods of management of the thermal mode have the approximate disorder nature, also nobody researched the parameter of time on which it is possible to turn off heating in the conditions of maintenance of the set level of thermal comfort indoors in working hours and economic feasibility of this method.

The purpose of this work is to develop a method of costs minimization for heating with maintains the set level of thermal comfort and optimum control of the thermal duty of the buildings functioning only in the afternoon.

Materials and Methods

The subject of the research

The subject of the research is the Federal State-Funded Educational Institution of Higher Professional Education.

The model of the studied subject is a multilayered enclosing structure. The scheme of this model is represented at the Figure 1.

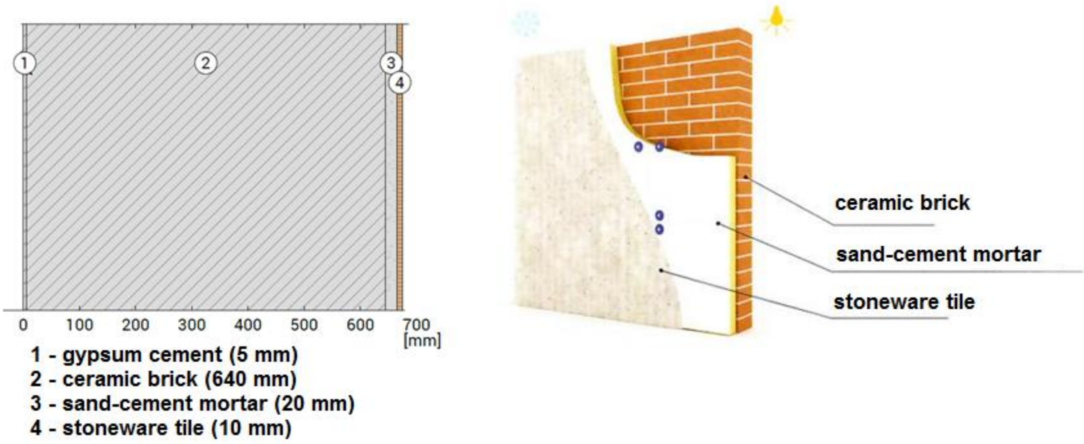

Figure 1. The scheme of multilayered enclosing structure

The characteristics of external structure of the building are presented at the Table 1.

Table 1. The Characteristic of external structure

Type of structure	**Characteristic**	**Materials**			
		gypsum cement	ceramic brick	sand-cement mortar	stoneware tile
External structure	δ, mm	5	640	20	10
	λ, (W/m^{2}·ºC)	0.35	0.47	0.025	0.8
	ρ, kg/m^3	1500	2000	1.25	2400
	c, J/kg·°C	840	880	1000	200

The advanced model of the studied subject is a multilayered enclosing structure. The scheme of this model is represented at the Figure 2.

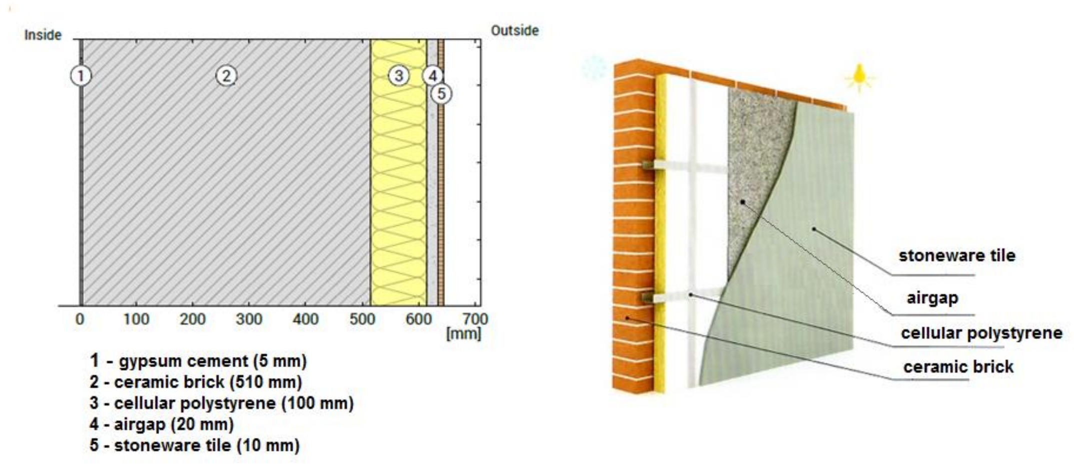

Figure 2. The scheme of advanced multilayered enclosing structure

Design structure of the advanced multilayered enclosing structure is presented at Table 2.

Table 2. The Characteristic of advanced external structure

Type of structure	Characteristic	Materials				
		gypsum cement	ceramic brick	cellular polystyrene	air gap	stoneware tile
External structure	δ, mm	5	510	100	20	10
	λ, (W/m²·ºC)	0.35	0.47	0.033	0.025	0.8
	ρ, kg/m³	1500	2000	100	1.25	2400
	c, J/kg·°C	840	880	1340	1000	220

Distribution of a temperature wave in a wall

The analysis of literature has shown that the one-dimensional motion of heat energy in a wall can be presented in the form of the differential equation Fourier:

$$\frac{\partial T}{\partial t} = a \cdot \frac{\partial^2 T}{\partial x^2}, \tag{1}$$

where T – temperature in any part of a body, °C;

t – time point, s;

x – wall coordinate, m;

a – internal coefficient of heat transfer, W/ (m²·°C).

The initial differential Fouriers equation is replaced by an integral relation:

$$\frac{d}{d\tau}\int_0^\infty T(\tau,\xi)\,d\xi = -\left(\frac{\partial T}{\partial \xi}\right)_{\xi=0}. \tag{2}$$

where $\tau := \frac{t}{t_0}, \xi := \frac{x}{\sqrt{at_0}}, h := \alpha\sqrt{\frac{t_0}{\lambda \rho C_p}}$

Distribution of temperature is defined in the form of:

$$T(\tau,\xi) = a(\tau)e^{-\xi} + b(\tau)e^{-2\xi} \tag{3}$$

The time-temperature transformation follows the sine theorem:

$$\theta(\tau) = a\sin 2\pi\tau, \tag{4}$$

Then we receive such solution of the differential equation and final expression for definition wall temperature in any part of enclosure structure at any moment:

$$T(\xi,\tau)=\left\{2a\sin 2\pi\tau-\frac{3}{2}\pi a\cos 2\pi\tau\right\}e^{-\xi}+\left\{\frac{3}{2}\cdot\pi a\cos 2\pi\tau-a\sin 2\pi\tau\right\}e^{-2\xi}. \quad (5)$$

Schedules of dependence of temperature from time for different points of a wall are submitted in Figures 3–4.

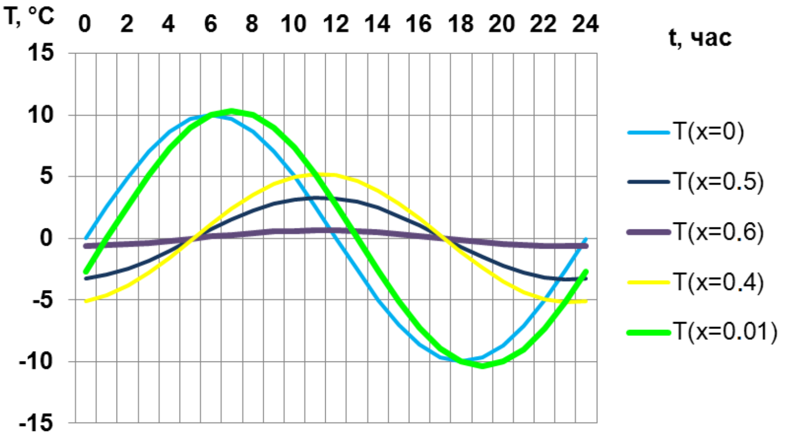

Figure 3. Distribution of temperature in a wall in time for a wall No. 1

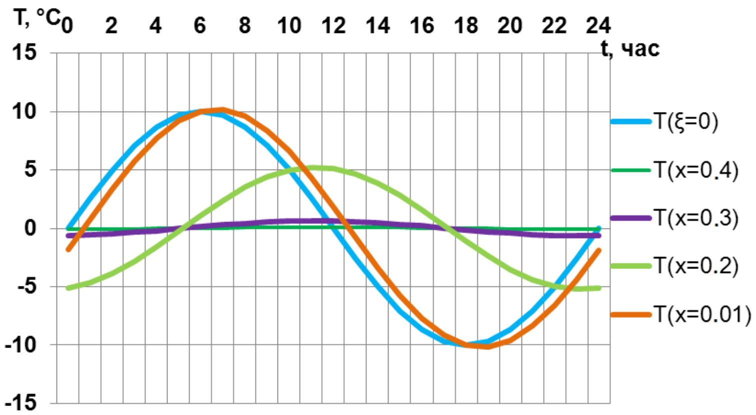

Figure 4. Distribution of temperature in a wall in time for a wall No. 2

From this schedules it is known that the temperature amplitude of oscillations decay in the thickness of structure. There is a phase shift of temperature oscillations in a structure, or in other words the delay of these oscillations in time.

Graphically it can be presented so in a Figures 5–6.

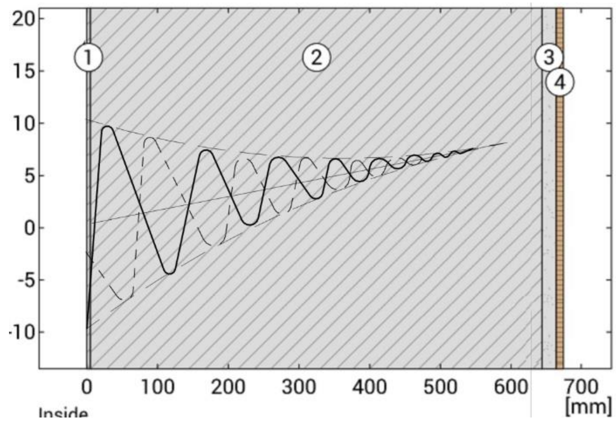

Figure 5. The schedule of temperature fluctuations in a wall No. 1

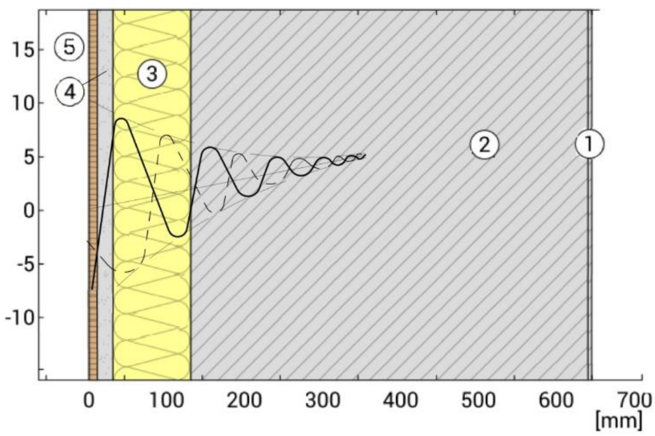

Figure 6. The schedule of temperature fluctuations in a wall No. 2

In the structure with a low heat transfer resistance the temperature fluctuations penetrate through the walls, but in energy efficient structure the temperature fluctuations are localized at a cold side of a wall.

Assessment of thermal stability of a design

We determined the average stationary temperature for each type of wall by the formula:

$$T_\infty = \frac{\alpha_c \cdot T_c + \alpha_h \cdot T_h \cdot \left(1 + \alpha_c \cdot \int_0^\delta \frac{dx}{\lambda(x)}\right)}{\alpha_c + \alpha_h \cdot \left(1 + \alpha_c \cdot \int_0^\delta \frac{dx}{\lambda(x)}\right)} + \frac{\alpha_c \cdot \alpha_h \cdot (T_h - T_c)}{\alpha_c + \alpha_h \cdot \left(1 + \alpha_c \cdot \int_0^\delta \frac{dx}{\lambda(x)}\right)} \cdot \frac{1}{\delta} \cdot \int_0^\delta \frac{(\delta - \xi) d\xi}{\lambda(\xi)} \quad (6)$$

where α_c = 23 W/(m^2·˚C) – outer surface heat transfer coefficient of the building envelope;

α_h = 8.7 W/(m^2·˚C) – inner surface heat transfer coefficient of the building envelope;

T_h – the internal air temperature in the premises of a residential building in St. Petersburg, taken in accordance with Russian State Standard GOST 30494 at 20 °C;

T_c – the average monthly outdoor air temperature in January, taken in accordance with Russian Building Norms and Regulations SNIP 23-01-99* [15] at -8.7 ° C;

δ – thickness of the wall, m;

λ – thermal conductivity, W/(m·°C).

Now we determine the instantaneous average temperature of the wall:

$$\overline{T} = (T_h - T_\infty) \cdot \theta + T_\infty. \quad (7)$$

where $\theta = \exp\left(-\frac{t}{t_h} - \frac{t}{t_c}\right)$; $t_h = \frac{\rho \cdot c_p \cdot \delta}{\alpha_h}$; $t_c = \frac{\rho \cdot c_p \cdot \delta}{\alpha_c}$.

Then time on which it is possible to turn off heating can be defined as time $\overline{T}(\tau)$ for which will approach average value of temperature of a wall in the stationary mode $\overline{T}_\infty$. Mathematically it can be written down so:

$$t_c = \frac{\overline{T} - \overline{T_\infty}}{v} \quad (8)$$

Results of calculation of instant temperature of a wall are presented in the table of the application. Schedules of instant temperature of a wall, or cooling, are submitted in Figure 7.

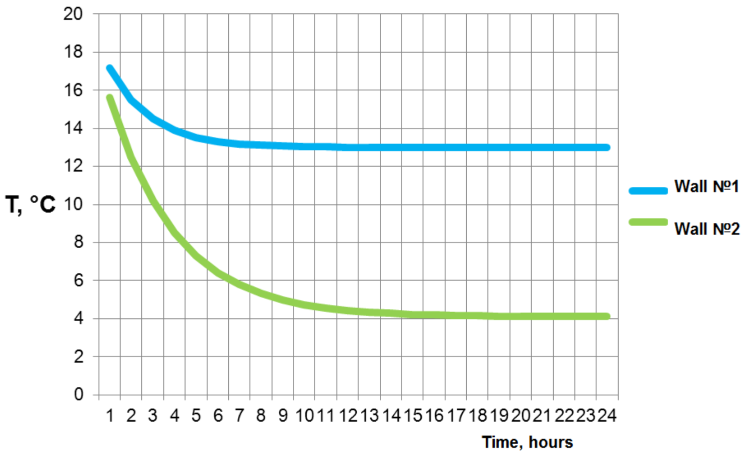

Figure 7. The schedule of cooling of a design of walls at a temperature of external air of -10 °C

The heating shutdown time is about 4 hours/day. We can provide the periodic heating duty.

Economic efficiency of decisions

The heattechnical calculation has been made and thermal losses of the building are defined (Fig. 8).

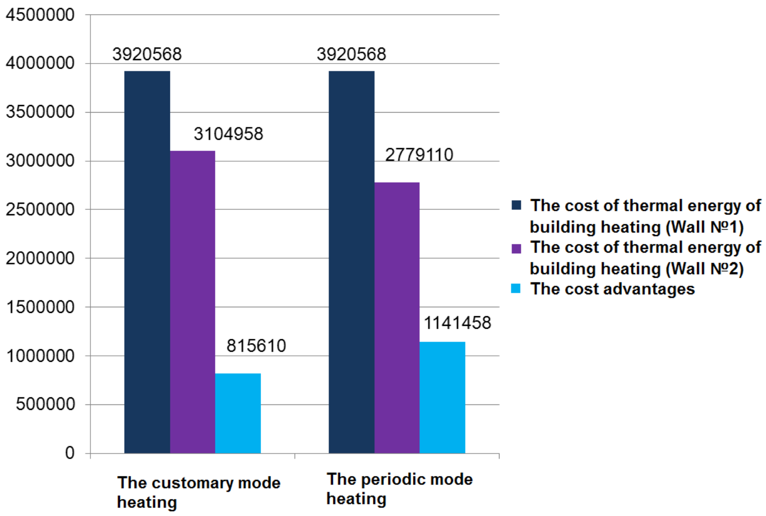

Figure 8. The schedule of cost of thermal energy on heating of the building

At increase in thermal resistance of a design of Gidrokorpus-2 the economy of thermal energy is about 800 thousand rub/year.

At advent of automated control station and periodic heating duty to the building the economy of the thermal energy for heating of the building is about 900 thousand rubles/year.

Results and Discussion

As a part of the research the following results have been received:

1. The fluctuation of temperature in energy efficient enclosure structure are localized at a cold side of a wall, and in construction with a low thermal resistance temperature fluctuations "penetrate" a wall.
2. High-frequency (daily, week) fluctuations of temperature are localized in a thin layer of fluctuations and do not cause noticeable change of average temperature of a wall on big times.
3. The velocity of cooling of a hot penetration conduit is less than the velocity of cooling in a structure with a large-scale thermal resistance.
4. The measure of energy efficiency of a wall protection is inversely proportional λ, and heat assimilation and thermal stability $\lambda^{1/2}$ is proportional; therefore, increase in energy efficiency reduces thermal stability and vice versa.

5. At increase in thermal resistance of a buildings construction the economy of thermal energy is about 800 thousand rub/year. At implementation of domestic heating plant to the building and providing the periodic mode of heating of the building the economy of the thermal energy for heating is about 900 thousand rubles/year. Increase the energy efficiency of a wall and use of accumulative ability from the economic point of view are equivalent.
6. The measure of energy efficiency of a wall protection is inversely proportional λ, and heat assimilation and thermal stability $\lambda^{1/2}$ is proportional; therefore, increase in energy efficiency reduces thermal stability and vice versa.

Authors [1–10] propose the constructive solutions of external walls providing their high thermal stability. Authors [11–22] cconsider the factors influencing the thermal mode of the room after shutdown of heat supply of the building. On rate of cooling the greatest influence is exerted by the size of warm losses through the enclosure structure at the expense of a heat transfer and on heating of infiltration air. The big areas of a glazing of the room are a factor of increase in heatlosses and, therefore, quickly cooling of the building after heating shutdown [23].

Conclusions

The typical building construction absorbs the temperature wave caused temperature fluctuations of external air. It means this building construction has a thermal stability. Therefore, there is a reserve for warmth accumulation which can be used for decreasing of thermal losses.

The measure of energy efficiency of a wall protection is inversely proportional to coefficient of heat conductivity, and heat assimilation and thermal stability is inversely proportional to coefficient of heat conductivity; therefore, increase in energy efficiency reduces thermal stability and vice versa.

Therefore, in future researches it would be possible to define conditions and resistance of a heat transfer of the protecting designs under which two concepts "energy efficiency" and "thermal stability" are crossed in one point.

Acknowledgement

Financial support for this project provided by the Peter the Great St. Petersburg Polytechnic University is gratefully acknowledged.

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Тарасова Д.С., Петриченко М.Р. Квазистационарные температурные режимы ограждающих конструкций // Инженерно-строительный журнал. 2017. № 4(72). С. 28–35.

doi: 10.18720/MCE.72.5

Qualified method of layer-by-layer summation to define the settlement of foundation

Уточненный метод послойного суммирования для определения осадки фундаментов

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Key words: elastic half-space; settlement; deformation modulus; triaxial compression

Ключевые слова: упругое полупространство; осадка; модуль деформации; трехосное сжатие

Abstract. The calculation of setting is the main criterion while designing shallow foundations as the complete setting. Methods of defining deformation in soil body show the calculation value of the final settlement which is several times different from the real. It is stated that the deformability of the ground coat is defined to a great extent by the degree of the horizontal intensity caused by natural pressure and by exterior additional loading. Here is offered a specified method of calculation of shallow foundations settlement based on the method of layer-by-layer summing. This method takes into account the separation of the exterior additional loading diagram into component parts which cause elastic and elastic-plastic deformations as well as changing the module of the general deformation of the layers of the earth foundation depending on the state of stress. To test the method there was conducted a comparative analysis with field testing of the ground coat of natural consistency by rigid stamps with its static loading. The calculation of setting by the method offered by the authors has a high convergence with the results of the experiments and reflects the general picture of deformations of the soil body while loading as well as unloading.

Аннотация. Расчет осадки является основным при проектировании фундаментов мелкого заложения. Зачастую методы по определение деформаций в грунтовом массиве показывают расчетное значение конечной осадки в несколько раз отличающееся от действительного. Отмечено, что деформируемость грунта во многом определяется величиной горизонтальных напряжений, возникающих от бытового давления и от внешней дополнительной нагрузки. Предложен уточненный метод расчета осадки фундаментов мелкого заложения, основанный на методе послойного суммирования. Данный метод учитывает разделение эпюры дополнительного внешнего нагружения на составляющие, которые приводят к появлению упругих и упруго-пластических деформаций, а также изменение модуля общей деформации слоев грунтового основания в зависимости от напряженного состояния. Для апробации метода проведен сопоставительный анализ с полевым испытанием грунтов естественного сложения жесткими штампами, при их статическом нагружении. Расчет осадки по методу, предложенному авторами, имеет высокую сходимость с экспериментальными результатами и адекватно отражает общую картину деформирования грунтового массива, как при нагрузке, так и при разгрузке.

Introduction

Calculation of settlement is challenging when designing shallow foundations (SF), since the absolute settlement and the resultant unevenness of settlements are the specified values directly determining serviceability of buildings and structures. Despite numerous methods which were developed by domestic and foreign authors for evaluating deformations in soil body, the calculated value of the final settlement sometimes differs several times from the actual one [1–4]. In engineering practice, in sample

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calculation of SF, the settlements exceeding the specified values may be obtained. Thus, cheaper shallow foundations are to be refused in favor of pile or pile-slab ones, but the opposite situation is possible in weak soils [5–8].

The final settlement is affected by some subjective and objective factors and primarily, the model of foundation and soil compressibility characteristics. When designing foundations, it is the engineer's responsibility to choose a geomechanics or contact model. Any model of soil bed used in the specific problem makes it possible to take into account the features of soil body deformation, but with the assumptions typical for each model [9, 10].

The elastic half-space model (EHM) has been the basic model of soil bed for a long time [11]. Application of the geomechanics model makes it possible to evaluate the stress state in unspecified external loading at any point of the soil body. Under certain conditions in initial loading, deformation of soil body predicted by EHM is a linear function and generally agrees with the experimental data [12–15].

Ignorance of nonlinear behavior of the soil bed, significant exaggeration of distribution capacity of the bed and strong forces concentrated at the edges of the foundation are the obvious drawbacks of this model. Numerous improvements and modernizations of EHM did not provide a universal tool to adequately predict the final settlement of shallow foundations different in areas and levels of loading [16, 17].

According to [18–20], the value of deformation modulus is of great importance for valid description of the deformability of soil body. At present, there exist some procedures to calculate the value; in here, the results can differ by several times. For historical reasons, the deformation modulus evaluated with the stamp, 5000 cm^2 in area, is considered to be the reference in Russia. Due to complexity of the stamp tests, their application in everyday engineering practice is quite difficult. However, as seen from the in-situ tests, the stamp modulus does not guarantee the adequate description of layer-by-layer deformations in depth, particularly when the foundation size significantly exceeds the stamp. Laboratory methods evaluating soil deformability characteristics are widespread, but a designer can obtain different results due to different representations. For instance, the in-situ tests showed that in transition from the constrained modulus to stamp deformation one, the multiplying coefficient m_k does not always correctly illustrate the resulting settlement and all the more, the actual deformation of soil bed in depth [19].

Despite evident values of the deformation moduli obtained by different methods and above all, by the stress state generated during the tests, design models usually include a constant that determines deformability of the layer in the entire area of loading. Given the in-situ test data which show that total compression mainly occurs within a sufficiently thin layer under the foundation while the rest is distributed to a greater depth, some authors propose to divide these zones into the elastic-plastic (large) and elastic (small) deformations. In [17] it is made by taking into account the structural strength of soil p_{str} and bilinear model where deformability in vertical stresses does not exceed p_{str} and is determined by the modulus of elasticity E_e;; in greater stresses – by the modulus of elastoplasticity E_{pl}.

According to [21, 22], the deformability of soil is strongly affected by the stress state of soil body. In their books on testing silty clays and sandy soils in stabilometers G.G. Boldyrev [18] and I.K. Aimbetov [23] show that the soil deformation modulus depends linearly upon the lateral (horizontal) stresses. Thus, E increased 2 times in semisolid sandy loam with the increase of horizontal stresses from 100 to 300 kPa and 3 times – in fine sand.

Thus, the "braking" effect of soil's own weight onto deformation of soil bed can be explained as opposed to the weightless half-space, especially when OCR increases. Thus, deformability of soil is largely dependent upon horizontal stresses being the sum of the horizontal overburden pressure σ_{xg} and horizontal normal component of the stress state σ_{xp} arising from the external load. Therefore, the deformation modulus at each point of the soil bed is an integral parameter of soil properties and its stress state and, consequently, it cannot be constant even in one geotechnical element. If the external pressure increases, the modulus will change under changing stress state and, above all, under the values of lateral pressure.

Methods

Proposed is the qualified method to calculate settlements in shallow foundations, and principally slab foundations, based on the convenient method of layer-by-layer summation adopted in Russian Building Regulation SP 22.13330.2011 [24] and DIN 4019. The proposed method takes into account the elastic-plastic deformation of soils and possesses the following features [25]:

- Separation of the diagram σ_{zp} (Fig. 1) into components: σ^{*}_{zp} – results in elastic and σ^{**}_{zp} – elastic-plastic deformations. The component σ^{*}_{zp} corresponds to the initial stress state in soil bed and may be equal to the dead weight pressure at a given depth $\sigma^{*}_{zp} = \sigma_{zg}$ or part of it $\sigma^{*}_{zp} = k\sigma_{zg}$ or structural strength of soil p_{str}, in turn, their values may be corrected with *OCR.*

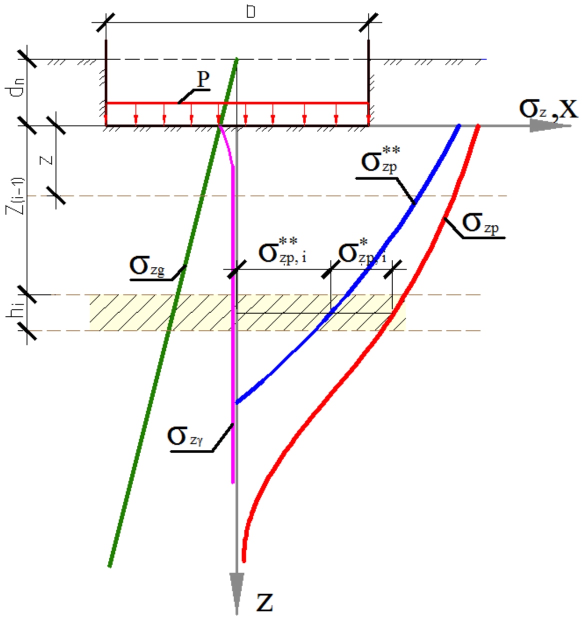

Figure 1. Design diagram of the bed to calculate settlement in a shallow foundation after the proposed method (SP*)

- Consideration of the changed total deformation modulus *E* of soil bed layers due to the increase of load and stress state.

Calculation of the settlement applying the qualified method (SP*) in a construction pit less than 5m in depth is evaluated by the formula:

$$s = \sum_{i=1}^{n} \frac{(\sigma^{**}_{zp,i} - \sigma_{z\gamma,i}) \cdot h_i}{E_i} + \sum_{i=1}^{n} \frac{\sigma^{*}_{zp,i} \cdot h_i}{E_{e,i}}, \tag{1}$$

where $\sigma^{**}_{zp,i}$ – vertical stress component resulting in elastic-plastic deformation of soil in the *i*-layer due to the average pressure across the foundation footing;

$\sigma^{*}_{zp,i}$ – vertical stress component resulting in elastic deformation of soil in the *i*-layer due to the average pressure across the foundation footing;

h_i – height of the *i*-layer;

E_i – deformation modulus of the *i*-layer of soil across the branch of primary loading;

$E_{e,i}$ – deformation modulus of the *i*-layer of soil across the branch of secondary loading;

It is needed to calculate the total deformation modulus *E* of every given engineering-geological element by means of stabilometers [26]; this makes it possible to consider stress-strain state of the samples in different values of the lateral pressure, when unloading and constructing the function $E = f(\sigma_{xp} + \sigma_{xg})$ (Fig. 2). The approach is advantageous since Poisson's ratio μ, being controversial for soils but influential for their compressibility, is not necessary to be taken into account directly in the formula when calculating the settlement through the non-dimensional coefficient *β*.

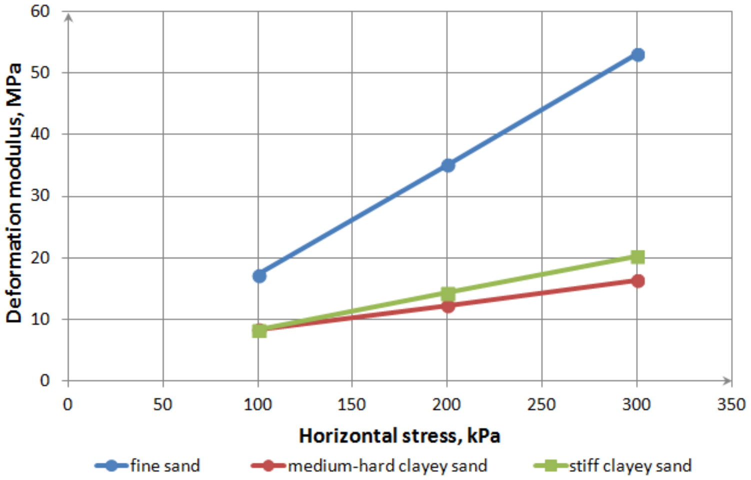

Figure 2. G.G. Boldyrev's dependency diagram $E = f(\sigma_x)$ **obtained after stabilometer tests [17]**

The method aims at calculating the lateral pressures σ_{xg} and σ_{xp} which directly affect the total deformation modulus *E*. In lack of the reliable values of *OCR*, the value σ_{xg} is taken in accordance with geo- or hydrostatics, i.e. $\sigma_{xg} = \sigma_{zg} \dfrac{\mu}{1-\mu}$ (2) and $\sigma_{xg} = \sigma_{zg}$ (3) correspondingly. Hydrostatics is used more frequently, but there is not any consensus here [27]. The value σ_{xp} may be obtained from solutions of the EHM theory. It is necessary to consider that Poisson's ratio μ=0.5, in its classical formulation for elastic half-space, may considerably differ for real soils. Thus, to calculate σ_{xp} after the given dependencies seems to be the most appropriate for soil beds loaded with flat footings.

Results and Discussion

In order to evaluate the proposed method, a comparative analysis was conducted; in here, the in-situ tests were conducted for soils being loaded statically and tested with rigid round stamps, 1200 mm in diameter.

The soil bed of the experimental site was composed of naturally occurred silty clays. Their physical and mechanical characteristics are given in Table 1.

Table 1. Physical and mechanical characteristics of soils at the experimental site

Engineering-Geological Element (EGE)	Depth of layer, m		Soil density, gr/cm²	Solid particles density, gr/cm²	Dry soil density, gr/cm²	Water content, %	Void ratio e, f. u.	Degree of saturation, f. u.	Liquidity index I_L, f. u.	Specific gravity γ, Kn/m³	Angle of internal friction φ, degrees	Cohesion C, kPa	Compressive deformation modulus E_c, MPa	Coefficient m_k, Russian Building Regulations	Calculated deformation modulus E, MPa	Soil
	from	to														
1	0.0	1.0	1.74	2.75	1.54	13	0.79	0.46	<0.0	17.4	15	31	3.3	5.7	19	clay
2	1.0	6.0	1.85	2.7	1.5	23	0.80	0.79	0.62	18.5	15	17	2.7	3.0	8	loam
3	6.0	6.6	19.1	2.65	1.57	22	0.69	0.84	average area	19.1	31	2	25	–	25	sand

Initially, deep marks were installed in the central and edge zones of the soil body at a distance from 0.25 D to 2.00 D (Fig. 3) to record layer-by-layer deformation of soil layers. The stamp surface was loaded through the distribution system across four points. The thruster with a pre-calibrated pressure gauge was used to create pressure in the system (Fig. 4).

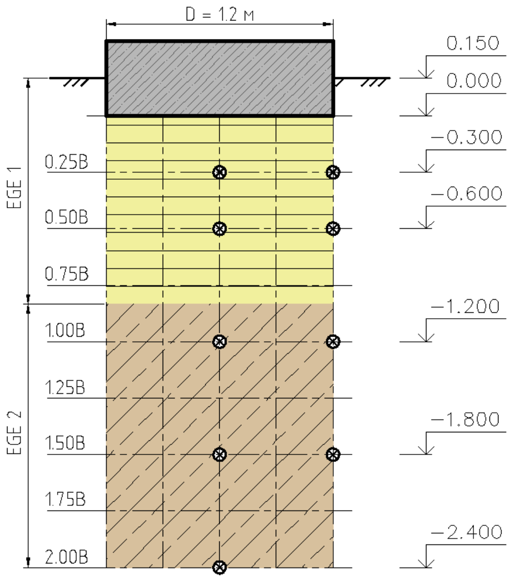

Figure 3. Location of deep marks in soil body

Figure 4. System of load transmission on the stamp surface

To study the effect of lateral squeeze on deformability of soils which occurs in the active zone, the stabilometer tests were conducted in the type A chamber. To obtain the dependencies of the changed deformation modulus upon horizontal squeezing of soil, the samples were tested in confining pressures – 50 kPa, 100 kPa and 200 kPa (Fig.6). The tests resulted in the values of the deformation modulus obtained for additional pressure in different ranges (Formula 1).

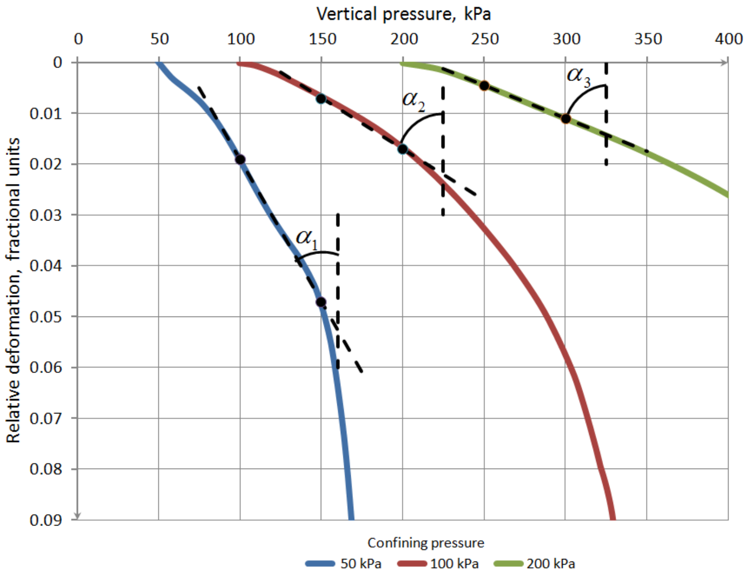

Figure 6. The results of stabilometer tests for EGE №2 – high-plastic loam

$$\boldsymbol{E_1 = tg\alpha_1,\qquad E_2 = tg\alpha_2,\qquad E_3 = tg\alpha_3} \tag{1}$$

Dependency of the deformation modulus upon the value of horizontal stresses is of linear character (Fig.7); in here, soil rigidity is increased proportionally due to horizontal pressure. It is necessary to underline that the deformation modulus of high-plastic loams increases 1.5 times faster than that of solid clays. Obviously, this happens due to the structure of soil skeleton.

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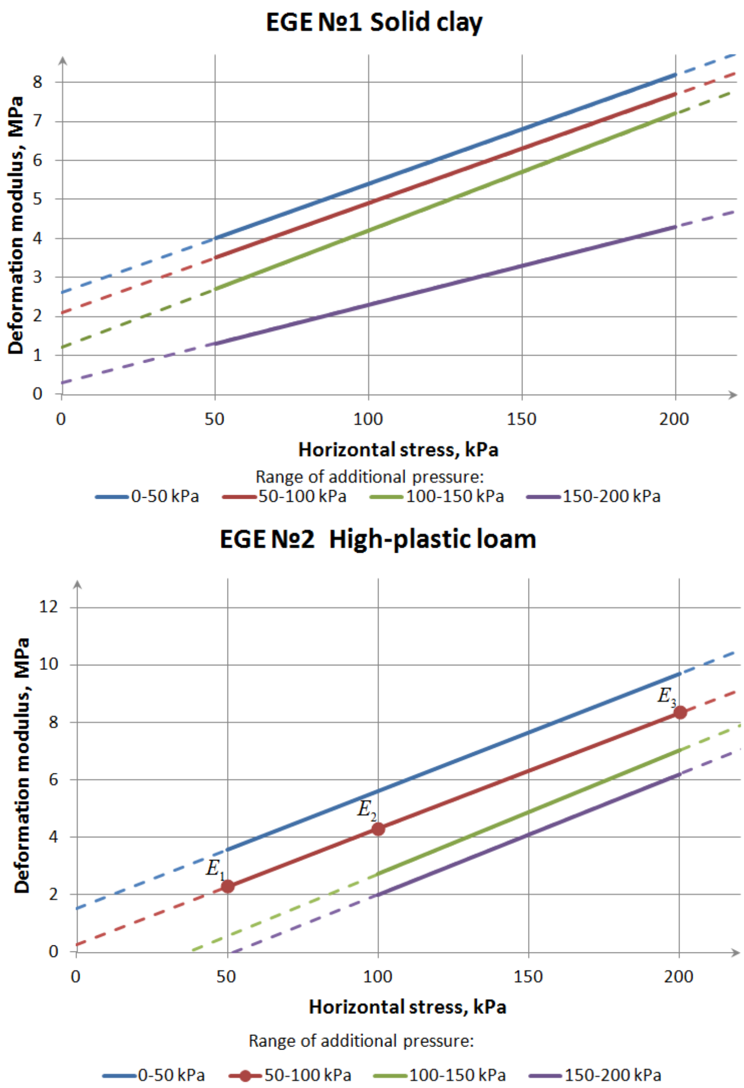

Figure 7. Dependency diagrams of the deformation modulus and the value of horizontal squeezing

The diagrams indicating dependency of the stamp settlement upon vertical pressure were obtained after the in-situ tests (Fig. 8). First, the experimental value of initial critical pressure upon soil was evaluated as if for a flat round foundation after the formula of K.E. Egorov and T.I. Finaeva ($p^* = 100$ *kPa*). The design resistance of soil evaluated by the formula given in Russian Building Regulations SP 22.13330.2011 was $R = 140$ *kPa*.

As seen from the diagrams, calculation of settlement possesses 40% of allowance across the whole range of loading without reference to the time factor (in $t \approx 0$) in accordance with the proposed method which applies the dependencies obtained after the stabilometer tests. The normative technique, i.e. the compressive deformation modulus and coefficient m_k, resulted in significantly underestimated value of the final settlement as compared to the experimental results. Calculation of settlement by means of DIN 4019 using the results of compressive tests and ignoring the transition coefficients exceeds the experimental values in 90–110 %. In accordance with the investigations [28] carried out on weak water-saturated soils, the settlements continue to increase with time ($t \to \infty$) after normal loading of the soil body and exceed, on average in 20–30 %, the initial settlements which are recorded when loading is stopped. Therefore, the settlements tend to approach the ones calculated after SP* with disagreements to be 5–15 % in the limits of the calculated design resistance *R*.

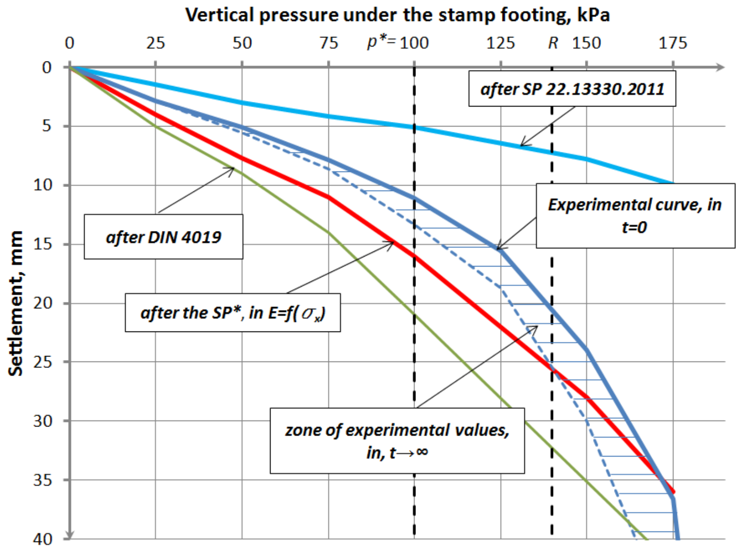

Figure 8. Dependency diagrams of the stamp settlement and vertical pressure

Calculations made after Russian Building Regulations SP 22.13330.2011 revealed considerably underestimated values of final settlements in the foundation model in gradual stepwise loading. Disagreements with the experimental results in the whole range of loading was nearly 100 %; if taking into account the factor of settlements which increases with time, then the results will differ in 2.5–3 times. These scattered results appear due to large values of the deformation modulus which was calculated after the compressive tests and resulted in the stamp modulus with the multiplying coefficient m_k.

The in-situ tests and the proposed technique revealed similar performance of the soil body when evaluating layer-by-layer deformation of soil (Figs. 9, 10). Nearly 70 % of settlement is generated in the surface zone, up to 0.8 D and within the depth of compressible thickness to be 2.0 D, determined in accordance with Russian Building Regulations SP 22.13330.2011. The diagrams of relative and absolute settlements plotted after the qualified method and experimental data obtained from the deep marks disagree in nearly 20 % across the overall thickness of the active zone. It is necessary to point out that weakening of deformations across the depth occurs much earlier than it is given in the solution after EHM theory, i.e. it agrees with the experimental data [29–31]. Calculation diagram of the settlements of soil layers and relative vertical settlement across the overall depth of compressible thickness shows the actual deformation. Difference in 7 mm in settlements on the soil bed surface can be explained by cutting the soil with stamp edges in the process of loading, as given in [17, 21]. This fact can be optionally considered when designing shallow foundations.

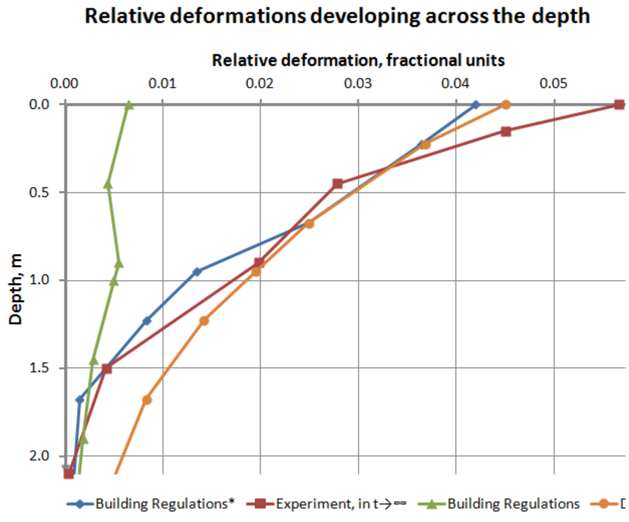

Figure 9. Diagram of relative settlements developing across the depth in p=175 kPa

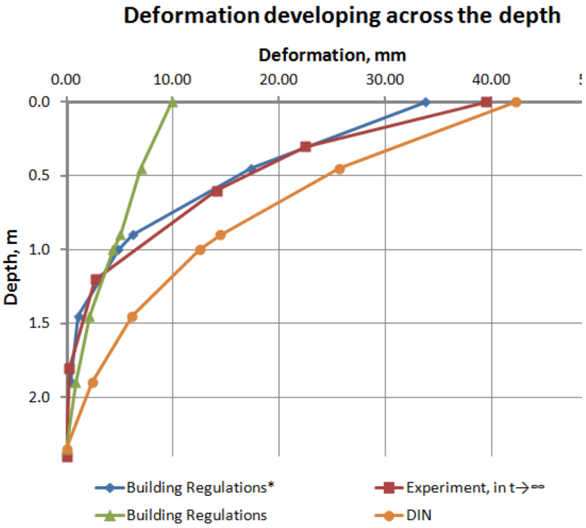

Figure 10. Diagram of settlements developing across the depth in p=175 kPa

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During the in-situ tests, the soil bed loaded statically was gradually unloaded in order to calculate elastic deformations in soil. The value of elastic deformation obtained experimentally was 8 mm, i.e. 18 % from the overall settlement of the stamp. This value almost agrees with the calculated one, i.e. 15 % (6 mm) and thus, “physicality” of the proposed method is proved.

This method does not intend to be used for settlement computation of problematic soil bed and in specific condition. The computation in specific soil coditions needs to further experimental and theoretical research.

Conclusions

The qualified method used to calculate settlements makes it possible to:

- take into account the diagram separated into components due to additional external load acting on the soil bed; this results in elastic and elastic-plastic deformations;
- use the characteristics of deformation obtained after triaxial compression indicators (stabilometers), i.e. the effect of the changed deformation modulus *E* as the horizontal stress function is taken into account; this allows considering the values adapted to specific conditions, e.g. after the value of over-consolidation ratio (OCR), structural strength $p_{str.}$ and specific soil properties;
- ignore the controversial coefficient *β* given in Russian Building Regulations SP 22.13330.2011, since it can result in significant errors when calculating the settlement;
- take into account the elastic-plastic nature of soil; in here, the calculated diagram of absolute and relative settlements of layers across the overall depth of the compressible thickness illustrates the real deformation of soil bed, i.e. "physicality" of the method is shown.

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doi: 10.18720/MCE.72.6

Design of steel beams cross three directions with sprengel

Проектирование стальных перекрестных балок трех направлений со шпренгелем

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Key words: dimensional steel construction; cross beams with spatial tightening; terms beams strength; terms strength puffs; deflection of the structure

Ключевые слова: пространственная стальная конструкция; перекрестные балки с пространственной затяжкой; условия прочности балок; условия прочности затяжек; прогиб конструкции

Abstract. The article presents one of the possible formulations and solutions of the optimization problem of designing the cross-beams of the combined system, reinforced bongs. The basis for efforts as functions of variable parameters was a displacement method. Approximate analytical depending forces and displacements in beams, and the puffs. Analytical expressions possible to formulate the problem of how to design optimization. The article points out that the problem was posed and solved for a fixed geometry puffs and beams. This narrows the area of optimal solutions, but greatly simplifies the solution itself. The objective function – linear. Restrictions – nonlinear functions, therefore considered the problem relates to the problems of nonlinear programming. Optimum obtained in one of the points of intersection of two active constraints – for tightening strength and maximum permissible deflection of the structure. Because of this discrete mathematical optimum mix does not coincide with the physical, so the solution obtained approximately.

Аннотация. В статье приводится одна из возможных постановок и решений оптимизационной задачи проектирования комбинированной системы перекрестных балок, усиленных затяжками. Основой для получения усилий как функций варьируемых параметров явился метод перемещений. Получены приближенные аналитические зависимости усилий и перемещений, как в балках, так и в затяжках. Аналитические выражения позволили сформулировать задачу проектирования как оптимизационную. В статье обращается внимание на то, что задача ставилась и решалась при фиксированной геометрии затяжек и балок. Это суживает область оптимальных решений, однако существенно упрощает само решение. Целевая функция – линейна. Ограничения – нелинейные функции, поэтому рассмотренная задача относится к задачам нелинейного программирования. Оптимум получен в одной из точек пересечения двух активных ограничений – по прочности затяжки и по предельно допустимому прогибу конструкции. Вследствие дискретности сортамента математический оптимум не совпадает с физическим, поэтому полученное решение приближенно.

Introduction

Cross beams three areas have long established themselves as expressive architectural coatings are widely used in practice [1–5]. The task of structural design can be supplied as an optimization. This formulation combines the strength and rigidity of conditions in a single task.

Advantages of this generalization are that a minimum while achieving highest objective function and, in addition, it is clear – conditions which are decisive for the design. An important advantage in that the process of designing a number of unrelated directly manual calculation methods is naturally transferred to the computer base.

To date, the time period studied in detail the flat load-bearing structures, including both conventional ties and ties with prestressed [6–11].

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The spatial beam structures with ties studied enough and need to be studied [12].

Methods

Considers cross continuous steel beams three directions hinged at its ends to the hexagonal plan. The system of cross-beams of rolled I-beams supported by a permanent stiffening spatial Sprengel (Fig. 1).

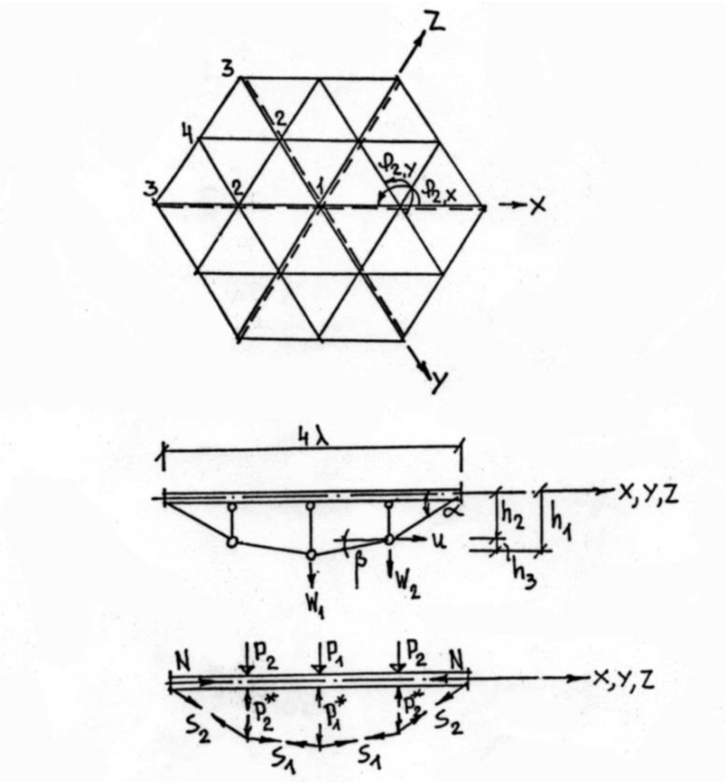

Figure 1. The investigated construction

A uniformly distributed load applied to the symmetric beams crossing sites in the form of concentrated forces. Torsion beams and shear deformations are neglected. Design Conditions coupling beams correspond to the case only a vertical interaction between them. The adopted scheme reflects well the real more complex contact conditions and significantly reduces the number of unknowns [13]. The material works elastically in accordance with Hooke's law. Stability of beams is considered wealthy. A deformation resistant is neglected.

Variable parameters are positive values:

$x_1 = \mathrm{A}$ – the cross sectional area of beams; $x_2 = \mathrm{A}_{\circ}$ – area of ties; $x_3 = \rho$ – core distance; $x_4 = h_6$ – the height of the beams; $x_5 = J$ – moment of inertia of beams.

The objective function, without the expense of influence on steel racks and therefore roughly expressing the volume of steel, can be written:

$$Z = 30\lambda x_1 + 6\lambda\left(\frac{1}{cos\alpha} + \frac{1}{cos\beta}\right)x_2 => min \tag{1}$$

A feature of optimization problems bearing structures as opposed to the economic problems, a small number of restrictions, but there is cumbersome analytical expressions of these restrictions. Sometimes itself getting analytical expressions in closed form is not simple and often independent research task.

Restrictions on the objective function (1) form three groups of equalities and inequalities:

I. Equality satisfying the conditions of equilibrium, physical and geometrical equations:

$$r \cdot [w] + [p] = 0, \tag{2}$$

where r – the coefficient matrix of the canonical equations of motion method comprising variable parameters;

[w] – the displacement vector components;

[p] – load vector.

The expanded form of equations (2) can be written as (3)

$$\left[\begin{array}{cccc|c} \left(72\frac{EJ}{\lambda^3}+36\frac{E_0A_0}{\lambda}\sin^2\beta\cos\beta\right) & -\left(72\frac{EJ}{\lambda^3}+6\frac{E_0A_0}{\lambda}\sin^2\beta\cos\beta\right) & -36\frac{EJ}{\lambda^2} & 0 & -6\sin\beta\cos^2\beta\frac{E_0A_0}{\lambda} \\ -\left(12\frac{EJ}{\lambda^3}+6\frac{E_0A_0}{\lambda}\sin^2\beta\cos\beta\right) & \left[21\frac{EJ}{\lambda^3}+\frac{E_0A_0}{\lambda}(\sin^2\beta\cos\beta+\sin^2\alpha\cos\alpha)\right] & 3\frac{EJ}{\lambda^2} & -6\frac{EJ}{\lambda^2} & \frac{E_0A_0}{\lambda}(\sin\beta\cos^2\beta\sin\alpha\cos^2\alpha) \\ -6\frac{EJ}{\lambda^2} & 3\frac{EJ}{\lambda^2} & 7\frac{EJ}{\lambda} & 0 & 0 \\ 0 & -3\frac{EJ}{\lambda^2} & 0 & 5\frac{EJ}{\lambda} & 0 \\ -6\frac{E_0A_0}{\lambda}\sin\beta\cos^2\beta & (\sin\beta\cos^2\beta+\sin\alpha\cos^2\alpha)\frac{E_0A_0}{\lambda} & 0 & 0 & (\cos^3\beta+\cos^3\alpha)\frac{E_0A_0}{\lambda} \end{array}\right] \tag{3}$$

$$x\begin{bmatrix} W_1 \\ W_2 \\ \varphi_{2x} \\ \varphi_{2y} \\ u \end{bmatrix} = \begin{bmatrix} P_1 \\ P_2 \\ 0 \\ 0 \\ 0 \end{bmatrix}$$

where W_1 – vertical movement of the i-th node

φ_{2i} – rotation angles of the node 2

u – horizontal movement of the coupling assembly of ties

II. Inequalities express terms of strength and stiffness of individual bearing elements and structure as a whole:

In particular:

1) The condition of strength beams according to the criterion of normal stresses

$$g_1 = 1 - \frac{M_1}{W_x R_y} - \frac{N}{A R_y} \geq 0\,, \tag{4}$$

where W_x – section modulus;

N – the longitudinal force in the beam;

i – beam cross section under consideration;

R_y - standardized resistance of steel.

2) Conditions strength of ties

$$g_2 = 1 - \frac{S_2}{A_0 R_y} \geq 0, \tag{5}$$

where S_2 – the greatest force in the tie.

3) Conditions design stiffness

$$g_3 = 1 - \frac{W_1}{[f]} \geq 0, \tag{6}$$

where $[f] = \frac{1}{400} * l$ – allowable amount of deflection.

III. Inequalities, reflecting the peculiarities of the constructive form

$$g_4 = A > 0 \tag{7}$$

$$g_5 = A_0 > 0 \tag{8}$$

$$g_6 = A - A_0 > 0 \tag{9}$$

Condition (4–9) are natural for the designer, but it is very important to the mathematical formalization of the problem. In order to reduce the number of constraints can be eliminated system (2) of the problem, and then analytically approximated expression of internal forces as continuous functions of variable parameters. For simplicity, we will solve the problem fixed geometry of ties ensemble:

$$h_1 = \frac{9}{24}\lambda;\ \ h_2 = \frac{\lambda}{3}\quad \text{and denote}\ \ K = \frac{E_0}{E} * \frac{A_0}{J} * \lambda^2$$

The dimensionless parameter "K" reflects the impact of the tightening of all varying sizes.

In this case the system of equations (3) can be rewritten in the form (10).

$$\begin{bmatrix} (72+0.0622К) & -(72+0.01036К) & -36 & 0 & -0.249168К \\ -(12+0.01036К) & (21+0.09649К) & 3 & -6 & 0.326076К \\ -6 & 3 & 7 & 0 & 0 \\ 0 & -3 & 0 & 5 & 0 \\ -0.249168К & 0.326076К & 0 & 0 & 1.85102К \end{bmatrix} \mathrm{x} \begin{bmatrix} \widehat{W_1} \\ \widehat{W_2} \\ \widehat{\varphi_{2x}} \\ \widehat{\varphi_{2y}} \\ \hat{u} \end{bmatrix} = \begin{bmatrix} 1 \\ 1 \\ 0 \\ 0 \\ 0 \end{bmatrix} \tag{10}$$

The constancy of geometry narrows the area of optimal solutions, but greatly simplifies the approximation expression. The solutions of the system (10) for different values of "K" are presented in Table 1. As can be seen from Table 1 tightening spatial trussed significantly increases rigidity, and greatly reduces the internal forces in the beams lowering steel consumption. Table 1 allows to obtain approximate analytical expressions for beams, and for delay. With an accuracy of less than 5 % in the range 100 ≤ K ≤ 250 true formula (11–13) (for A <100 can quadratic approximation):

$$N = (2.41925 + 0.002927K)P \tag{11}$$

$$S_2 = (2.54867 + 0.003093K)P \tag{12}$$

$$M_1 = (0.13 - 0.00032K)P\lambda \tag{13}$$

Table 1. The solutions of the system (10) for different values of "K"

Deformation and strength factors			**System without ties K=0**	**$K = \frac{E_0 A_0}{E J_x} * \lambda^2$ System with ties**				**Factor**
				K=100	K=150	K=200	K=250	
Linear displacement	Vertical	W_1	0.561	0.131	0.095	0.075	0.062	$\frac{P\lambda^3}{EJ_x}$
		W_2	0.390	0.090	0.065	0.050	0.041	
	Horizontal	u_x	-	0.002	0.001	0.001	0.001	
The angular displacement		$\varphi_{2,x}$	0.313	0.074	0.054	0.043	0.035	$\frac{P\lambda^2}{EJ_x}$
		$\varphi_{2,y}$	0.234	0.054	0.039	0.030	0.025	
bending moments		M_1	0.397	0.101	0.076	0.062	0.053	$P\lambda$
		$M_{2,x}$	0.230	0.047	0.032	0.023	0.018	
		$M_{2,y}$	0.468	0.108	0.078	0.060	0.049	
Transverse force		Q_{1-2}	0.167	0.054	0.044	0.039	0.036	P
		Q_{2-3}	0.230	0.047	0.032	0.023	0.018	
		Q_{2-4}	0.468	0.108	0.078	0.060	0.049	
Efforts in ties		S_1	-	2.714	2.942	3.073	3.154	
		S_2	-	2.859	3.099	3.237	3.328	
Largest strength in the longitudinal beams		N	-	2.712	2.939	3.070	3.151	
Efforts in spacers		P_1^*	-	0.678	0.734	0.767	0.787	
		P_2^*	-	0.791	0.857	0.896	0.919	

The deflection at the geometric center of the "K" = 100 ÷ 250 is approximately

$$W_1 = (0.1777 - 0.000464K)\frac{P^H\lambda^3}{EJ} \tag{14}$$

The analytical expressions are substituted into the limit II and III Group.

Having specific values: P=10т ; $\lambda = 6m$; $E = 21 * 10^6\ t/m^2$;

$[f] = \frac{l}{400} = \frac{\lambda}{100}; \frac{P}{pH} = 1.2;\ R_y = R_0 = 21000\ t/m^2$

and approximate dependence: $J \approx \frac{A\rho h_6}{2} = \frac{x_1 x_3 x_4}{2};\ h_6 \approx \frac{\rho}{0.32};\ x_3 \approx 1.818\sqrt{x_1}$ can be reduced to the task with five varying parameters to two independent parameters x_1 и x_2. Thus the objective functions with a fixed geometry and flow of ties neglecting steel rack written more specifically:

$$Z = 180x_1 + 74.16x_2 \ => \ min \tag{15}$$

Restrictions:

The strength of beams, which receive the thrust of ties:

$$g_1 = 1 - \frac{0.00115}{x_1} - \frac{0.0000996x_2}{x_1^3} - \frac{0.000204}{x_1\sqrt{x_1}} + \frac{0.0000035x_2}{x_1^3\sqrt{x_1}} \geq 0 \tag{16}$$

Durability of ties:

$$g_2 = 1 - \frac{0.00121}{x_2} - \frac{0.000102}{x_1^2} \geq 0 \tag{17}$$

The rigidity of the structure:

$$g_3 = 1 - \frac{0.000049}{x_1^2} + \frac{0.000000073x_2}{x_1^4} \geq 0 \tag{18}$$

Restrictions defined physical meaning of the problem:

$$g_4 = x_1 > 0 \tag{19}$$

$$g_5 = x_2 > 0 \tag{20}$$

Restrictions defined structural features form:

$$g_6 = x_1 - x_2 > 0 \tag{21}$$

Characteristic mathematical model is reduced to four positions:

1. The mathematical model described by continuous and differentiable functions.

2. The objective function – linear.

3. Restrictions on the objective function – nonlinear.

4. The problem relates to a class of constrained optimization problems.

The scientific literature on the issues of language and methods of solving nonlinear programming problems is extensive [14–16], and implemented by the mathematical ideas very diverse [17–20].

Results and Discussion

One possible solution to this problem of constrained optimization is to convert it into unconstrained optimization problem. To do this, enter all six additional restrictions are non-negative variables $u_i^2 \geq 0$, thus converting inequality constraints limit equality. Further, writing the Lagrangian:

$$F(x, m, u) = Z(x) + \sum_{i=1}^{8} m_i[g_i(x) + u_i^2 - b_i], \tag{22}$$

determine the necessary conditions for the existence of extremum, the so-called Kuhn-Tucker conditions:

$$\begin{aligned} &\frac{\partial F}{\partial x_j} = 0 && j = 1, 2 \\ &\frac{\partial F}{\partial m_i} = 0 && i=1,\dots,6 \\ &\frac{\partial F}{\partial u_i} = 2m_i u_i = 0 && i=1,\dots,6 \end{aligned} \tag{23}$$

In our case, we get a system of 14 nonlinear equations. Among all those roots only examine the system that satisfy the system of constraints (16) – (21),i.e. ,

$$x_1^* = 0.00682\, m^2;\ x_2^* = 0.0016\, m^2;\ \ m_1^* = 0.1124;\ m_2^* = 0.6231$$

The Hessian matrix at the extremum point is positive:

$$H = \begin{bmatrix} \dfrac{\partial^2 F}{\partial x_1^{*2}} & \dfrac{\partial^2 F}{\partial x_1^* \partial x_2^*} \\ \dfrac{\partial^2 F}{\partial x_2^* \partial x_1^*} & \dfrac{\partial^2 F}{\partial x_2^{*2}} \end{bmatrix} = \begin{bmatrix} 24509 & 12669 \\ 12669 & 66408 \end{bmatrix} = 146709{\times}10^4 > 0$$

Therefore, found a fixed point is a point of mathematical minimum.

Because of this discrete mix (I №40 B1) [21], the actual point of optimum

$x_1^{**} = 0.00746;\ x_2^{**} =\ 0.0017$ It is somewhat different from the mathematical.

As seen in Figure 2. Mathematics optimum is at the border of the feasible region at the intersection of two active constraints – on the condition tightening strength and maximum allowable deflection structure.

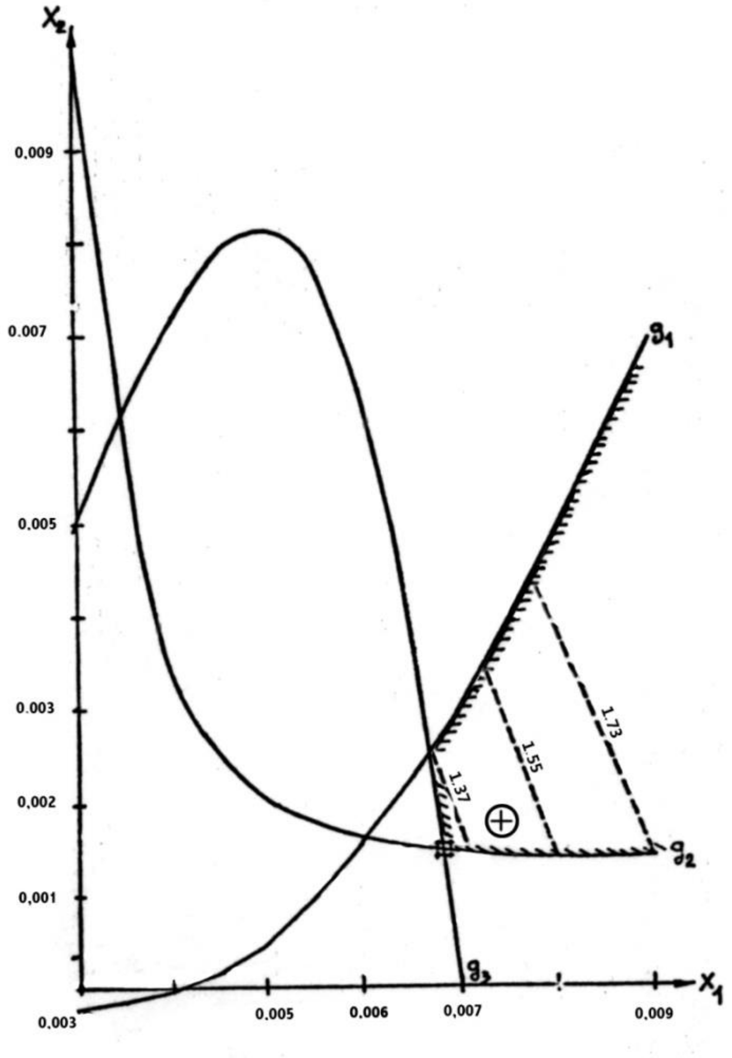

Figure 2. Graphical interpretation of the problem being solved

where ⌑ – Point of mathematical optimum;

⊕ – the optimum point in connection with the readability of the assortment;

– – – line levels of the objective function;

—— – restrictions

– the range of permissible values

The results of the decisions shows that the parameter K = 247.9, and is within the accepted linear approximations (11–14).Terms beams strength g_1 are not critical and are beyond the tolerance range varying variables, so a more accurate calculation of the deformed scheme does not lead to the results of other solutions. In particular, for the case settlement of the deformed scheme causes an additional increase in stress in the beams by 3.5 %, which is less than the permissible 5% according to the norms. [22]

The slope of the line levels of the objective function shows that the most effective way to further reduce the consumption of steel is to increase the height of the Sprengel.

Steel flow without weight ratio of spacers and the construction is 25.6 kg/m2, which is a very good indicator.

Conslusion

The solution provided by the problem is not unique. It is worth considering, and other formulations, and use other methods of solution. The accumulation of experience solving similar close to the practice, the tasks, the proposed methodology will improve and clarify many of the questions of the real design.

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doi: 10.18720/MCE.72.7

Mechanical properties of nanocarbon modified cement

Механические свойства наноуглеродного цемента

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Key words: buildings concrete; CNT; cement; civil engineering; construction materials; carbon nanotubes; nanofibers

Ключевые слова: бетонные здания; УНТ; цемент; гражданское строительство; строительные материалы; углеродные нанотрубки; нановолокна

Abstract. This article covers use of nanocarbon modified cement (nCMC) that was made with use of chemical vapor deposition method. The tests were performed with traditional destructive method as well as with the use of special ultrasonic equipment. The specimens with nCMC was tested for compressive and tensile strength. However, the results showed that use of nCMC made only minor effect on these characteristics, when results from other research groups indicate that nCMC improves cement's properties greatly. Therefore, authors hypothesize that characteristics of nCMC modified paste matrix highly dependent on the geometrical characteristics of nanofiber such as length, type of structure (single-walled, multi-walled) and chirality. This hypothesis is based on the ultrasonic tests' results. Furthermore, the positioning, distribution and the proportion in the material of carbon nanotubes (CNTs) needs to be researched more.

Аннотация. В статье рассмотрено использование наноуглеродного цемента (НУЦ), который был получен методом химического осаждения из газовой фазы. Испытания цементных материалов проводились традиционным разрушающим методом, а также с использованием специального ультразвукового оборудования. Образцы с НУЦ испытывали на прочность при сжатии и растяжении. Однако результаты показали, что использование НУЦ оказывает незначительное влияние на прочностные характеристики, в то время как результаты других исследовательских групп показывают, что НУЦ значительно улучшает свойства цемента. Поэтому авторы выдвигают гипотезу о том, что характеристики модифицированной пастообразной матрицы НУЦ сильно зависят от геометрических характеристик нановолокна, таких как длина, тип структуры (однослойные, многостенные) и хиральность. Кроме того, необходимы дополнительные исследования положения, распределения и доли в материале углеродных нанотрубок (УНТ).

Introduction

Nanomaterials in construction industry became one of the most innovative line of research. Even though there are not many cases of their practical use, this field of search has many perspectives due to unique characteristics of these materials. Nanostructures allows to significantly improve such properties as: compressive strength, conductivity, and resistance to electromagnetic pulse (EMP) [1, 2]. Furthermore, use of nano-modified concrete can greatly reduce the cost of construction. This can be achieved by reducing the amount of cement in the compound of concrete.

Concrete being the most used material in construction is valued for its characteristics such as compressive strength, hardness and modulus of elasticity. However, nanotechnologies allowed to greatly improve these characteristics. A group of scientist from Saint Petersburg Polytechnical University (SPBPU) managed to greatly increase the compressive strength of concrete with synthesizing carbon nanofiber on cement particles [3]. The result of their research was hardened cement paste that was twice as strong as the control one. Moreover, civil engineers have been already using concrete with modifiers based on fullerene for a long time [4, 5]. These modifiers can also greatly improve compressive strength

Cherkashin A.V., Pykhtin K.A., Begich Y.E., Sherstobitova P.A., Koltsova T.S. Mechanical properties of nanocarbon modified cement. *Magazine of Civil Engineering*. 2017. No. 4. Pp. 54–61. doi: 10.18720/MCE.72.7.

of the concrete. However, the problem of equal distribution of modifier through the volume of the material still needs to be resolved for all types of nanomodifers. So it can be concluded that there are two main trends in research of nano-concrete: synthesis of nanostructures on matrix material and direct addition of nanostructures into modified material [6–10].

Analysis of recent researches indicated the lack of data covering the test of cement hardened paste modified with nanocarbon modified cement (nCMC). Nevertheless, there is a lot of information about the concrete produced with use of special cavitational setups [19], although this method of producing of concrete is quite complicated and there is no data considering simple mixing of nCMC with cement in certain proportions. Moreover, further analysis of this problem revealed that there is actually no information about attempts to define characteristics of nCMC that is simply mixed with standard cement in concrete mixture.

Therefore, the main goal of this research is to precisely examine the properties of nCMC. To achieve it our team had to produce the samples in lab conditions under the control of the supervisor, test them and analyze all gathered data.

Experimental methods

To create specimens of modified cement was used special setup made on the base of Laboratory of lightweight materials and structures. The process of chemical vapor deposition allows to grow carbon nanostructures that may be formed into multi and single-walled structures. The possibility of formation of such structures is obtained due to the presence of iron in the clinker mineral celite. With increasing temperature and duration of the synthesis increases the amount of nanostructures, however the critical factor is the emergence of free CaO, which has negative impact on cement products. Regarding this, the temperature of synthesis is limited to 650 ^{o}C.

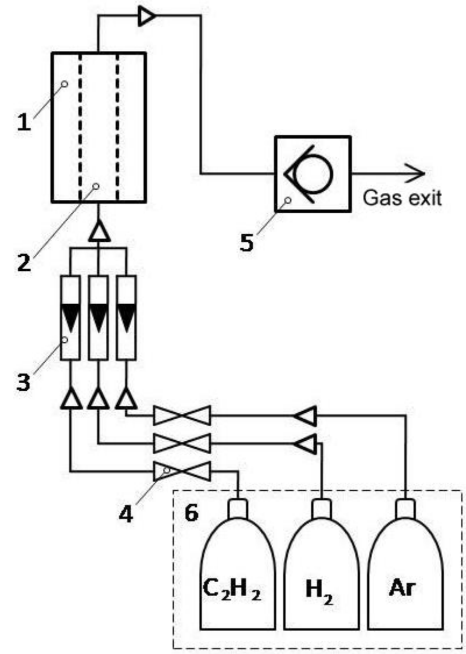

Figure 1. Schematic diagram of the device used for nCMC synthesis:
1 – furnace, 2 – chemical reactor, 3 – rotameter, 4 – valves, 5 – hydraulic lock, 6 – gas tanks

The cement used for this research has the following characteristic and compound: grade M500 D0, no additional mineral supplement, C_3S – 60.42 %, C_2S – 12.94 %, C_3A – 5.72 %, C_4AF – 12.46 %. Placed in a reactor with an argon atmosphere the cement of clinker minerals is heated to a temperature of 650 °C, the heating allows to recover hydrogen, and then argon got replaced by acetylene. As a result of catalytic decomposition of carbon in acetylene-hydrogen environment under atmospheric pressure, there can be noticed a growth of carbon nanotubes and nanofibers using carbon supersaturated iron catalyst. The catalyst particles can stay at the top of the nanofibers or nanotubes (apical growth) or in ground (root growth). The number of nanostructures depends on the amount of iron catalyst. In this case, 1 % by weight of the cement matrix

Tests of the specimens were performed in accordance with Russian State Standard GOST 310.4-81. "Cements. Methods for determination of flexural strength and compression".

To determine the water-cement ration of the mortar mix was used 1,500 g of standard uniform sand and 500 g of cement.

Sand and cement were mixed in a spherical cup, pre-rubbed with a wet cloth, and hand mixed for 1 minute. In the center of the dry mixture was made the deepening to which was added 200 g of water, in order to maintain water-cement ratio of 0.4. Water soaked for 30 seconds, and then the mixture was stirred again for 1 minute. After that mortar was put into Pan grinder (Fig. 2) for 150 seconds (20 rpm).After the additional mixing it was divided and put in the cone layer by layer. Then the form was mounted on a vibrating table. Each layer was also rodded 10–15 times. Then all the excesses were removed the mixture was shaken 30 times for 30 seconds. After all these procedures the form was removed and the lower base of the cone was measured in order to define the flow of this particular mixture.

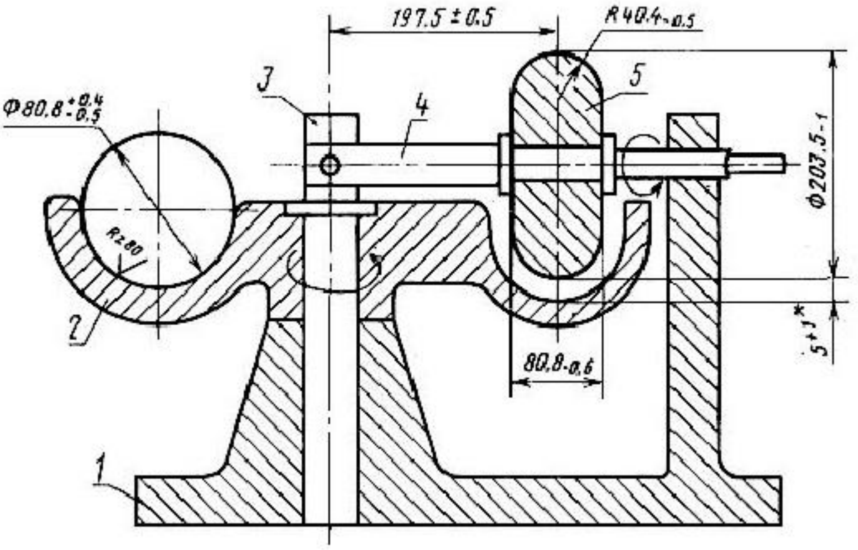

Figure 2. Pan grinder:
1 – base, 2- bowl, 3 – bowl axis, 4 – axis slider, 5 – runner

With W/C = 0.40 the optimal flow of the mortar mix is considered to be around 106 ÷ 115 mm. If the flow is less than 106 mm, W/C should be increased in order to achieve the flow of 106 ÷ 108 mm. If the flow is more than 115 mm W/C need to be decreased to obtain the flow around 113 ÷ 115 mm. In our case the W/C = 0.48

For the production of control test specimen were used the following compound: 500g of cement (PC 500 D0), 1500g standard uniform sand and 240ml water (W/C = 0.48).

For the manufacturing of the beam specimens the mortar mix was prepared the same way as the mix for the definition of flow with the exception for the shaking part. The cement paste of standard consistency was laid in special forms with size 40 x 40 x 160 mm, that were attached to the vibrating table, and compacted by vibration for 3 minutes. Before switching on the vibration each section was filled with a 1 cm layer of a mortar mix. The remaining part of mixture was put during the first 120 seconds of vibration. After the vibrating, excesses of the mixture were removed and the surfaces were smoothed. Then specimens wereput for 24 hours into chamber with high humidity level. Until the tests beams were stored in water under the temperature of 20+-2C.

The test of the beams was performed at the age of 3, 7 and 28 days. The specimens were tested for their tensile and compressive strength using special presses.

Method of preparation of the control mortar mix was also used in the manufacture of test beams with nCMC in which certain amount of cement was replaced with nCMC respectively (1 %, 5 %, 10 %).

Furthermore, the specimens were tested with non-destructive method of ultra-sonic defect identification. All the specimens were tested with surface (Fig. 3a) and direct method (Fig. 3b). It should be also mentioned that the surface method demanded three measurements of ultrasonic impulse velocity and each was determined from different side of the specimen.

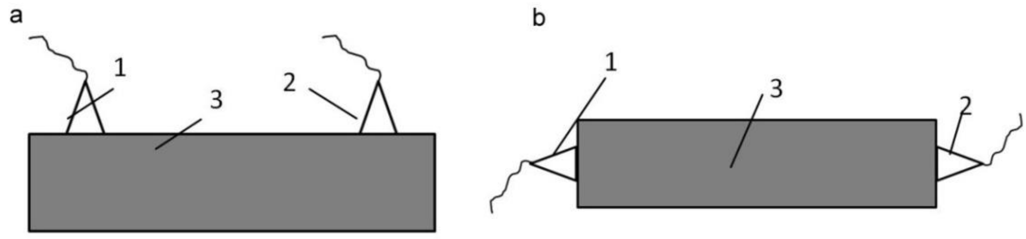

Figure 3. Ultrasonic pulse velocity test: a – The surface (indirect) method, b –The direct method (cross probing). 1, 2 – transducer head, 3 – specimen

Experimental results

The data below states (Tab. 1) that noticeable improvement of the compressive strength (Fig. 4) was demonstrated only by the specimens that contained 1 % and 5 % of nCMC. Nevertheless, the control specimen turned out to have the maximum tensile strength (Fig. 5). It seems that these results are quite different in comparison to the group of researchers that managed more than 100 % of compressive strength improvement. [20–22]. But it may be only an indicator of the geometric characteristics of nanostructures. Only their presence is not a determining factor. Consider the length, thickness, density and mass of carbon nanostructures

Table 1. Compressive and tensile strength of the specimens

№ (%)	Age, days					
	3		7		28	
	Tensile strength (Rt), MPa	Compressive strength (Rc), MPa	Tensile strength (Rt), MPa	Compressive strength (Rc), MPa	Tensile strength (Rt), MPa	Compressive strength (Rc), MPa
Control specimen	7	37.5	8.2	50.36	8.6	60.2
1% nCMC	6.9	37.5	8.1	51.85	8.2	61.8
5% nCMC	6.6	35.7	7.8	49.85	8.2	63.47
10% nCMC	6.45	33.14	7.5	48.65	8.3	58.82

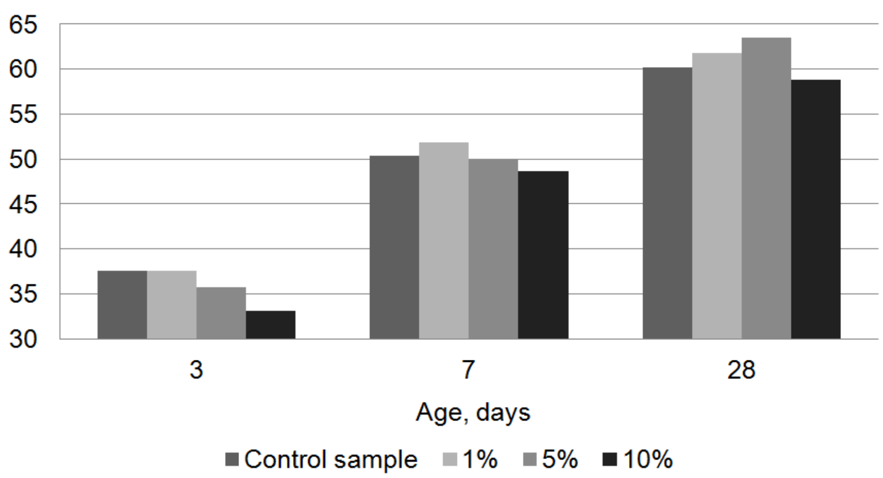

Figure 4. Compressive strength of the specimens

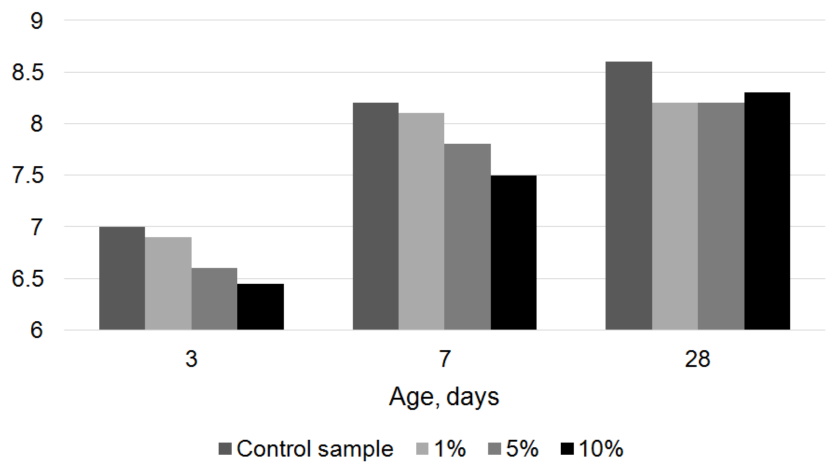

Figure 5. Tensile strength of the specimens

However, the ultrasonic tests discovered that velocity of the ultrasonic pulse highly varies between control and nCMC specimens (Fig. 6). The results showed that the speed of the impulse during the surface control of the compressive strength is about 16% higher in the modified specimens. Nevertheless, the direct method showed quite the similar numbers for all specimens. It should be also stated that during the compression tests the nCMC specimens was destroyed with a certain blast sound.

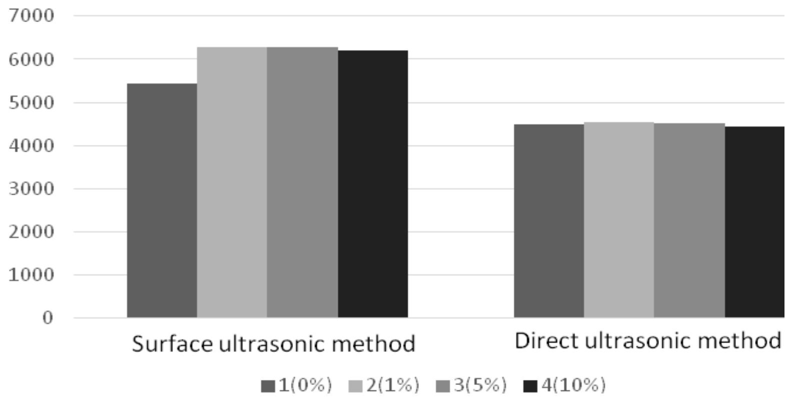

Figure 6. Results of ultrasonic impulse tests

The images below demonstrate two types of nCMC. The first one with lower density of CNTs (Fig.7a) represents the material that was tested within the scope of this article. The second one (Fig.7b) is the perspective material that is going to be tested in order to define the optimal geometric characteristics of CNTs synthesized on the cement matrix.

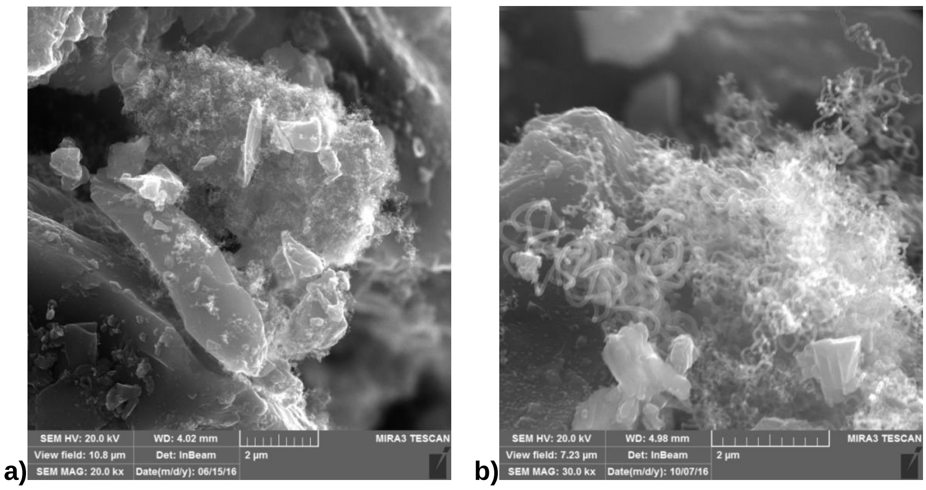

Figure 7. SEM images of the cement matrixes with different types of geometrical characteristics of CNTs. a – the content of the nanostructures 1 %, b – the content of the nanostructures 6 %

Discussion

The effectiveness of nCMC synthesized under different conditions can be only determined experimentally, however photos of the specimens allow us to suppose that compressive strength correlates with geometrical configuration of nanofibers on the cement. It seems that the synthesis of longer carbon nanotubes with higher density may lead to greater improvement of strength that just may be a confirmation of the study [23–25], where presented the research of foamed concrete, where they tested carbon nanomaterial that was a mixture of nanotubes and nanofibers with an average diameter of 20–40 nm and 2 μm long.

The figure shows (Fig.8) the approximate area of maximum hydration, which produces maximum internal stress

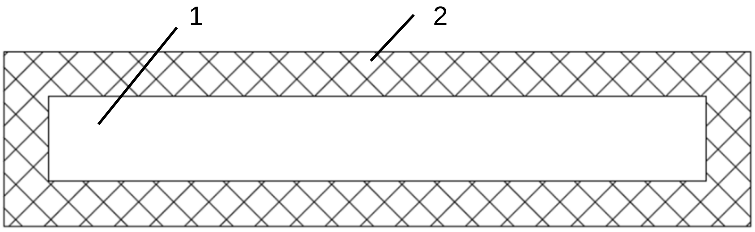

Figure 8. Approximate spreading of CNFs throughout the specimens:
1 – area presumably less hydration, 2 – area presumably the maximum hydration

Therefore it may be concluded that proper geometrical characteristics of nanofibers still have to be defined.

Furthermore, on the images in results section it is clearly seen that the length and density of nanofibers on the new specimens have multiplied and with following researches we will be able to compare different geometrical characteristics of CNTs.

Conclusions

However, as it was stated before, final conclusion can only be made after proper experiments. Nevertheless, on this stage of research we can already made following conclusions:

- Geometrical characteristics of nanofiber can greatly improve mechanical features of the concrete. Therefore, the main goal is to determine the most effective set. It is very perspective field of study because the majority of researches only defined the optimal concentration of nCMC or tested only concentration of the CNTs.
- During lab tests was noticed the fact that all nCMC-containing specimens had better results only in surface ultrasonic tests. With such dispersion of the CNTs throughout the specimens we can made a hypothesis that cement in the middle of the specimens had no feasibility to be properly hydrated. This can be explained by hydrophilicity of carbon, which prevented the penetration of water in the middle part of the sample. Nevertheless, it could be also stated that the porosity of the material in the outer layers lowered. This can lead to better water and frost resistance characteristics.

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doi: 10.18720/MCE.72.8

Analysis of ground temperature variations on the basis of years-long measurements

Анализ изменения температуры грунта на основе многолетних измерений

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Key words: ground temperature; ground heat exchangers; ground warming; thermal regime; geothermal heat pump system

Ключевые слова: температура грунта; грунтовый теплообменник; потепление грунта; термический режим; геотермальная теплонасосная система

Abstract. The article deals with results of the analysis of years-long measurements of the ground temperatures, for example: Russia, Moscow. Data are given on variations of the ground temperatures in the course of a year, at a 10-days interval, in two plots of land – one under exposed surface, the other under natural cover. Trends of ground temperature variations have been shown, for a period of over 50 years. As a part of research, an important application task is being handled, that is acquisition of information on a typical curve of ground temperatures to enable carrying-out more correct calculations of ground heat exchangers for any region of any country, and making forecasts of operating efficiency of geothermal heat pump systems, including long-term forecasts.

Аннотация. В статье представлены результаты анализа данных многолетних измерений температуры грунта на примере России г. Москвы. Приводятся данные по изменению температуры грунта в течение года с 10-дневным интервалом независимо для двух участков - с обнажённой (exposed) поверхностью и под естественным покровом. Выявлены и продемонстрированы тенденции изменения температуры грунта за период более 50 лет. В ходе исследования решается важная в прикладном плане задача – получение климатической информации о типовом ходе температур грунта, которая позволит более корректно выполнять расчёты грунтовых теплообменников для любого региона любой страны, а также прогнозировать эффективность работы геотермальных теплонасосных систем, в том числе и на длительную перспективу.

Introduction

Geothermal heat pump systems (GHPS) are gaining ground everywhere, including Russia [1–5]. As a heat source GHPS uses ground heat exchangers of various designs, both vertical and horizontal [6]. At present, greater emphasis is put on issues of improved operation efficiency of ground heat

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exchangers. Thus, in studies [7–10] various modifications of existing designs of ground heat exchangers are offered, and in [11–13] new versions of heat exchangers are under review. In addition to the design of the ground heat exchanger itself, a crucial factor that must be obligatorily taken into account for design process of a GHPS is a set of ground parameters in which the ground heat exchanger will operate including, among other things, data on its temperature [14, 15]. Importance of climatic conditions consideration during ground heat exchangers design process is analyzed in [16], where conclusion was made that air temperature fluctuations can lead to a reduction in ground heat exchangers depth. The purpose of the present study is analysis of years-long data on thermal conditions of the ground at a depth up to 5 meters in the Moscow region over a period, appropriate to show climatic trends. This way, an important application task is being handled, that is acquisition of climatic information on a ground temperature curve to enable carrying-out more correct calculations of ground heat exchangers and making forecasts of operating efficiency of geothermal heat pump systems, including long-term forecasts.

Measuring methods

As recommended by the World Meteorological Organization, under today climatic conditions, affected by various regional trends, for research one shall use a series of continuous measurements, taken over a period of at least 30 years [17]. With these considerations, the period from 1982 till 2011 has been chosen for a 10-day analysis.

As source data on ground temperatures, results of years-long measurements have been used, taken by the meteorological observatory of the Lomonosov Moscow State University, where thermal regime of the ground has been observed since 1955 at eleven depth points. Considered were mean 10-day ground temperatures at specified depths as follows: 20 cm, 40 cm, 60 cm, 80 cm, 120 cm, 160 cm, 240 cm and 320 cm.

The measurement method of the parameter discussed in the article (soil temperature) is established on the state network of meteorological stations during the last fifty years. According to it, in standard meteorological time the current and maximum surface temperatures are measured by mercury thermometers, and the minimum temperature – by an alcohol thermometer, as well as the ambient air temperature in the psychrometric box [22]. This operation is performed every 3 hours throughout the year. Measurements in the soil stratum are carried out as follows: to more accurately measure the parameters of the thermal balance of the earth's surface in the period from spring to autumn, temperatures at a depth of 5, 10, 15 and 20 cm are measured with the help of Savinov cranked mercury thermometers. Deeper horizons (starting from 20 cm) use bottom-hole thermometers TPV-50. Unlike the Savinov thermometers, these devices are arranged as follows: to reduce vertical heat transfer they are shielded by a protective tube and copper or brass sawdust is put around the reservoir to increase their inertia. At present, both ebonite and low-density polyethylene serve as the material of the tubes [22]. This method of measurement should be recognized as fairly accurate: the error in measuring temperature is in the range from 0 to 40 °C is ± 0.2 °C, and in the range of negative temperatures from 0 to -20 °C is ± 0.3 °C.

Every day, every three hours, measurements of current, maximum and minimum temperatures on an underlying surface are taken (warm season: soil surface, cold season: snow surface). To measure a surface temperature a standard mercury-filled thermometer was used; to measure a maximum surface temperature a mercury-filled meteorological thermometer was used; and minimum temperature was measured by an alcohol meteorological thermometer.

As far as ground heat exchangers may be placed both under maintained areas with no vegetation cover, where snow is removed in winter time (parking areas, roads, pedestrian ways), and under lawns, during research it was enabled to compare thermal regime of a surface with vegetation cover and surface free of it, in all seasons. For that, measurements were taken simultaneously in two plots of land: in grounds under exposed surface and under natural cover (Figure 1).

Bottom-hole thermometers are read once a day, with the exception of depths of 20 and 40 cm, where measurements are taken 8 times a day under natural cover and 4 times a day under exposed surface. With a significantly high snow cover (more than 15 cm, when it increases, and more than 5 cm, when it melts), all the 8 bottom-hole thermometers are read once a day.

Area of a plot of land with exposed surface is 12×20 m (Figure 1.a).

a)  b) 

Figure 1. Subsurface temperature measurements a) exposed surface; b) natural cover

Calculation methods

Considering that depths at which ground heat exchangers can be placed may vary depending upon tasks being handled, designs of the ground heat exchangers and plot features, data on ground temperature conditions up to deeper depths than those that turn out to be possible for standard measurements could appear to become necessary. To obtain this data, values of ground temperatures at deeper levels have been calculated on a basis of measurements in higher layers.

In the present study mean 10-day data on ground temperatures at a depth of 40–320 cm were used, collected for a period from 1982 till 2011. Values of ground temperatures at a depth of 400 cm and 480 cm have been calculated by means of Fourier method. The method is based on the Fourier law, where the temperature conductivity coefficient in depth is supposed to be constant. That is described by an equation as follows:

$$\frac{\partial T}{\partial t} = k\frac{\partial^2 T}{\partial z^2}, \qquad (1)$$

where "T" is ground temperature (°C), "z" is depth (m), "t" is time (s), "k" is temperature conductivity coefficient (m^2/s). In this manner, the temperature conductivity coefficient of the ground and temperatures at higher levels are sufficient to calculate values at deeper levels.

From Equation (1) the temperature conductivity coefficient has been calculated for a deepest of measured levels – 160–320 cm – using an equation as follows:

$$k = \frac{\partial T}{\partial t} \Big/ \frac{\partial^2 T}{\partial z^2}, \qquad (2)$$

where derivatives have been approximated using central differences:

$$\frac{\partial T}{\partial t} = \frac{T_{i+2}-T_i}{t_{i+2}-t_i}, \qquad (3)$$

$$\frac{\partial^2 T}{\partial z^2} = \frac{T_{j-1}-2T_j+T_{j+1}}{(z_j-z_{j-1})^2}, \qquad (4)$$

where "i" is a number of a 10-day period, "j" is depth. On the basis of the mean (throughout the period) temperature conductivity coefficient, ground temperatures at deeper levels have been calculated:

$$\mathrm{T}_{j+1} = \frac{(\mathrm{T}_i-\mathrm{T}_{i+2})(z_j-z_{j-1})^2}{(t_i-t_{i+2})k} - \mathrm{T}_{j-1} + 2\mathrm{T}_j, \qquad (5)$$

where T_{j+1} is a desired value of temperature at a deeper level. Equation (5) has been used to obtain temperatures at a depth of 400 cm and 480 cm.

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Results and Discussion

After measurements for 80–320 cm have been averaged, as well as calculations for depths of 400–480 cm, for each 10-day period tables were drawn up, to contain mean temperatures for the period from 1982 till 2011 (exposed surface / natural cover). Tables 1 and 2 present these results, correspondingly.

Table 1. Mean 10-day ground temperatures at various depth points, 1982–2011 (exposed surface, with no turf in warm season and with no snow in cold season)

Month	Decade No.	80 cm	120 cm	160 cm	240 cm	320 cm	400 cm
January	1	-0.1	2.2	3.9	6.8	8.4	9.2
January	2	-0.6	1.6	3.3	6.3	8.0	8.7
January	3	-1.0	1.1	2.8	5.8	7.5	8.4
February	1	-1.7	0.7	2.4	5.3	7.1	8.2
February	2	-2.2	0.1	1.9	4.9	6.8	8.0
February	3	-2.0	-0.1	1.6	4.6	6.4	7.7
March	1	-1.7	-0.2	1.4	4.3	6.1	7.4
March	2	-1.0	0.0	1.3	4.0	5.9	7.2
March	3	-0.5	0.2	1.3	3.8	5.6	7.0
April	1	0.0	0.4	1.3	3.7	5.4	6.9
April	2	0.9	0.9	1.6	3.6	5.3	6.8
April	3	2.8	1.9	2.2	3.8	5.2	6.9
May	1	5.8	3.8	3.4	4.2	5.3	7.1
May	2	8.7	6.5	5.4	5.0	5.6	7.6
May	3	11.1	8.8	7.4	6.1	6.3	8.1
June	1	13.5	11.1	9.4	7.4	7.1	8.6
June	2	14.8	12.6	10.9	8.6	8.0	9.1
June	3	15.7	13.7	12.1	9.7	8.9	9.6
July	1	16.9	14.8	13.1	10.7	9.7	10.1
July	2	18.0	15.9	14.1	11.6	10.4	10.8
July	3	18.6	16.7	15.1	12.5	11.2	11.3
August	1	18.6	17.1	15.7	13.2	11.9	11.7
August	2	18.0	17.0	15.9	13.7	12.5	12.1
August	3	17.1	16.5	15.7	14.0	12.9	12.4
September	1	15.8	15.7	15.3	14.1	13.2	12.5
September	2	14.1	14.6	14.6	14.0	13.3	12.5
September	3	12.6	13.3	13.7	13.6	13.2	12.4
October	1	10.9	12.0	12.7	13.2	13.0	12.3
October	2	9.4	10.7	11.6	12.6	12.7	12.0
October	3	7.5	9.2	10.4	11.9	12.3	11.7
November	1	5.8	7.7	9.2	11.1	11.8	11.4
November	2	4.5	6.4	8.0	10.3	11.2	11.1
November	3	3.4	5.4	7.0	9.5	10.7	10.7
December	1	2.4	4.5	6.1	8.8	10.1	10.3
December	2	1.7	3.7	5.4	8.1	9.5	9.9
December	3	0.7	2.9	4.6	7.4	9.0	9.5

Table 2. Mean 10-day ground temperatures at various depth points, 1982–2011 (plot of land with natural cover, turf height not exceeding 5 cm)

Month	Decade No.	80 cm	120 cm	160 cm	240 cm	320 cm	400 cm	480 cm
January	1	2.3	3.3	4.6	6.0	7.0	7.7	8.0
January	2	2.1	3.1	4.2	5.6	6.7	7.4	7.7
January	3	1.9	2.8	3.9	5.2	6.3	7.0	7.5
February	1	1.8	2.6	3.7	4.9	6.0	6.8	7.4
February	2	1.7	2.5	3.5	4.7	5.7	6.6	7.2
February	3	1.6	2.3	3.3	4.5	5.5	6.4	7.0
March	1	1.6	2.3	3.2	4.3	5.3	6.2	6.8
March	2	1.5	2.2	3.1	4.1	5.1	5.9	6.5
March	3	1.7	2.2	3.0	4.0	4.9	5.7	6.4
April	1	2.2	2.5	3.0	3.9	4.8	5.6	6.3
April	2	3.5	3.3	3.5	3.9	4.7	5.5	6.3
April	3	5.3	4.6	4.3	4.3	4.8	5.5	6.2
May	1	7.2	6.1	5.4	4.9	5.0	5.5	6.2
May	2	8.6	7.5	6.6	5.7	5.4	5.7	6.3
May	3	9.9	8.7	7.6	6.5	5.9	6.0	6.4
June	1	11.4	10.0	8.7	7.3	6.5	6.3	6.6
June	2	12.6	11.1	9.7	8.1	7.1	6.7	6.8
June	3	13.5	12.0	10.6	8.8	7.7	7.2	7.1
July	1	14.3	12.9	11.4	9.6	8.3	7.7	7.5
July	2	15.2	13.7	12.1	10.2	8.9	8.1	7.8
July	3	15.8	14.4	12.9	10.9	9.5	8.6	8.1
August	1	15.9	14.7	13.4	11.5	10.0	9.0	8.4
August	2	15.7	14.7	13.6	11.9	10.5	9.4	8.7
August	3	15.2	14.5	13.6	12.2	10.8	9.8	9.0
September	1	14.4	14.0	13.4	12.3	11.1	10.0	9.2
September	2	13.3	13.3	13.0	12.2	11.2	10.2	9.3
September	3	12.1	12.3	12.4	11.9	11.2	10.3	9.5
October	1	10.9	11.3	11.7	11.5	11.0	10.3	9.5
October	2	9.7	10.3	10.9	11.0	10.8	10.3	9.5
October	3	8.2	9.1	10.0	10.4	10.5	10.1	9.5
November	1	6.8	7.8	9.0	9.7	10.1	9.9	9.4
November	2	5.5	6.6	7.9	9.0	9.6	9.6	9.3
November	3	4.5	5.7	7.1	8.3	9.0	9.2	9.1
December	1	3.7	4.9	6.3	7.6	8.5	8.9	8.8
December	2	3.1	4.3	5.7	7.0	8.0	8.5	8.5
December	3	2.7	3.8	5.1	6.5	7.5	8.1	8.3

Table 3 presents mean annual ground temperature at various depth points, 1955–2011, under natural cover and exposed surface

Trends of ground temperature distribution with depth under natural cover and exposed surface differ essentially from each other. On average, within 1955 – 2011, ground temperature under natural cover does not vary with depth, and its value is 7.7°C, accurate to a tenth. Ground temperature distribution with depth under exposed surface is characterized by temperature rise with depth increase, with a maximum value at a depth of 320 cm.

Table 3. Mean annual values of ground temperatures at various depth points, 1955–2011

Depth, cm	Under natural cover, °C	Under exposed surface, °C
20	7.7	6.9
40	7.7	6.9
60	7.7	6.9
80	7.7	7.0
120	7.7	7.2
160	7.7	7.8
240	7.7	8.3
320	7.7	9.1

Table 4 shows minimum and maximum measured ground temperatures at various depth points, 1955–2011.

Table 4. Minimum and maximum ground temperatures at various depth points, 1955–2011

Depth, cm	Under natural cover, °C		Under exposed surface, °C	
	Min.	Max.	Min.	Max.
20	-5.7	25.0	-17.3	43.0
40	-3.3	23.7	-15.8	31.9
60	-2.2	21.3	-11.5	26.6
80	-0.7	20.1	-10.7	25.0
120	0.3	18.3	-5.1	21.9
160	1.2	16.7	-1.9	19.9
240	2.6	14.6	1.4	17.7
320	3.6	12.8	-6.1	15.4

As the tables show, at a depth of 20 cm ground temperatures vary most as compared to deeper levels due to stronger impact of underlying surface conditions. That is why a temperature curves under natural cover and exposed surface differ essentially from each other at that depth. Under natural cover an annual temperature range makes 30.7 °C, while it is 60.3 °C under exposed surface due to absence of a thermal insulating layer at higher levels (snow cover in winter / turf and grass cover in summer). Under natural cover another ground temperature distribution trend is observed at this depth: it remains almost constant from January till April. This trend is related to seasonal snow cover within these months that prevents ground cooling in winter and ground warming in spring.

Figure 2 shows annual variations of ground temperatures at various depths. Figure 2a shows that after snow cover melting (March–April) at a depth of 20 cm ground temperature intensively rises, while under exposed surface ground temperature begins to rise at that depth a month earlier.

Ground at a depth of 160 cm (Figure 2b) under exposed surface cools down in winter and warms in summer more intensively than that under exposed surface. This effect is to be taken into account for design of horizontal ground heat exchangers for GHPS, because lower ground temperatures in winter, when GHPS operates in heating mode and heat is extracted from the ground, will cause lower operation efficiency of the system. In summer, when GHPS operates in cooling mode, higher ground temperatures will hinder heat discharge into the ground and use of passive cooling. Considering this, GHPS ground heat exchangers shall be rather placed under natural cover.

At a depth of 320 cm in winter inverse distribution of ground temperatures can be observed, as compared to levels mentioned above: under exposed surface the ground temperature turns out to be higher than that under natural cover. In summer a trend of temperature variations remains the same.

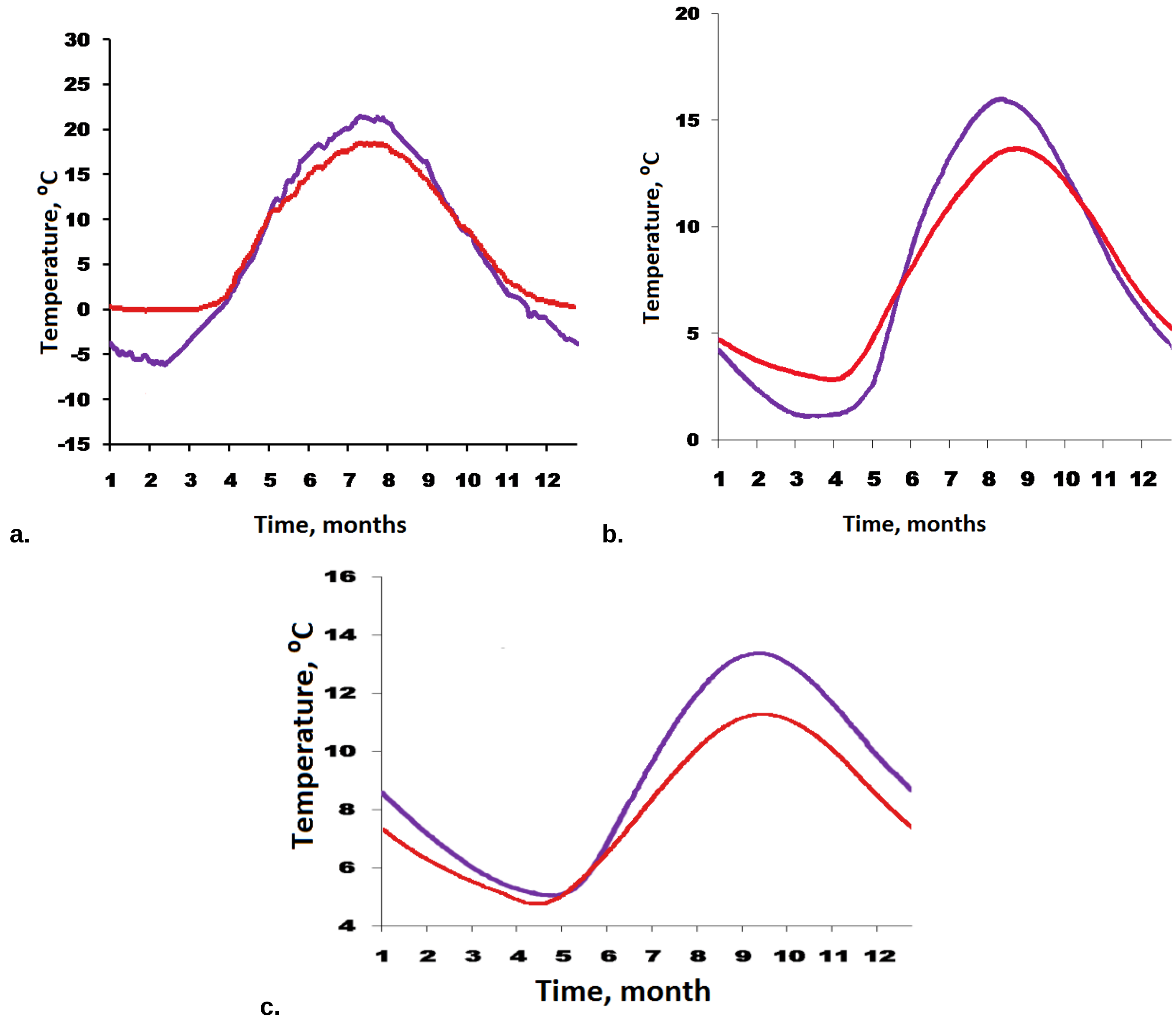


Figure 2. Annual curve of ground temperature under natural cover (red line) and exposed surface (blue line), average values within 1955-2011 at a depth of as follows: a) 20 cm; b) 160 cm; c) 320 cm.

Figure 3 shows annual variations of ground temperature at a depth of 160 cm and mean ambient air temperature throughout the period under review. Each parameter in the graph is used for a trend line to reveal the nature of variations within this years-long period.

It is worth noting that at this depth a steady trend to ground warming is observed. Rate of warming under exposed surface is close to a rate of ambient air temperature rise, while under natural cover a rate of warming is two and a half times slower (Figure 3).

In the study [18], based upon the data of the same meteorological station, similar trends are shown in the ground at a depth of 320 cm.

But this is not something special for Moscow city. The same effect of ground warming was previously observed by other researchers for different cities, for example for Karlsruhe [19], Tokyo, Seoul, Osaka and more [20]. In [19] a conclusion was made that the subsurface heat balance in the urban area in Karlsruhe is obviously dominated by anthropogenic heat sources. Among these heat sources thermal energy input from reinjections of thermal wastewater, sewage leakage and the district heating network together with the heat input from buildings were named. But the most of the ground heat gain is caused by increased ground surface temperature, which is in direct proportion to ambient air temperature.

In practice the fact of ground warming can be used to reduce the length of ground heat exchanger. According to [21], each additional degree of ground temperature could save around 4 m of the borehole length for the same heating power supply.

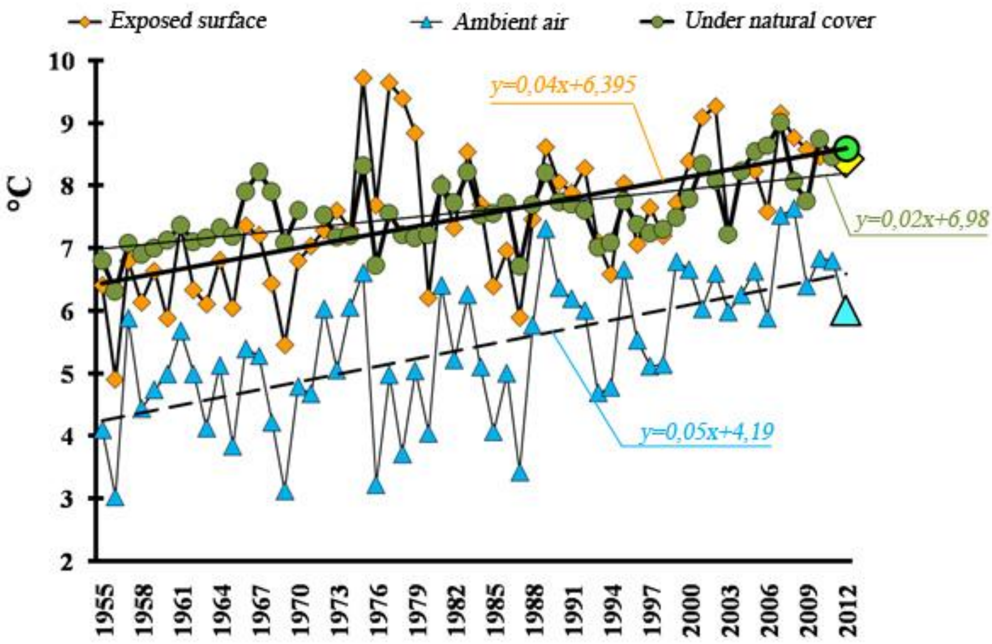
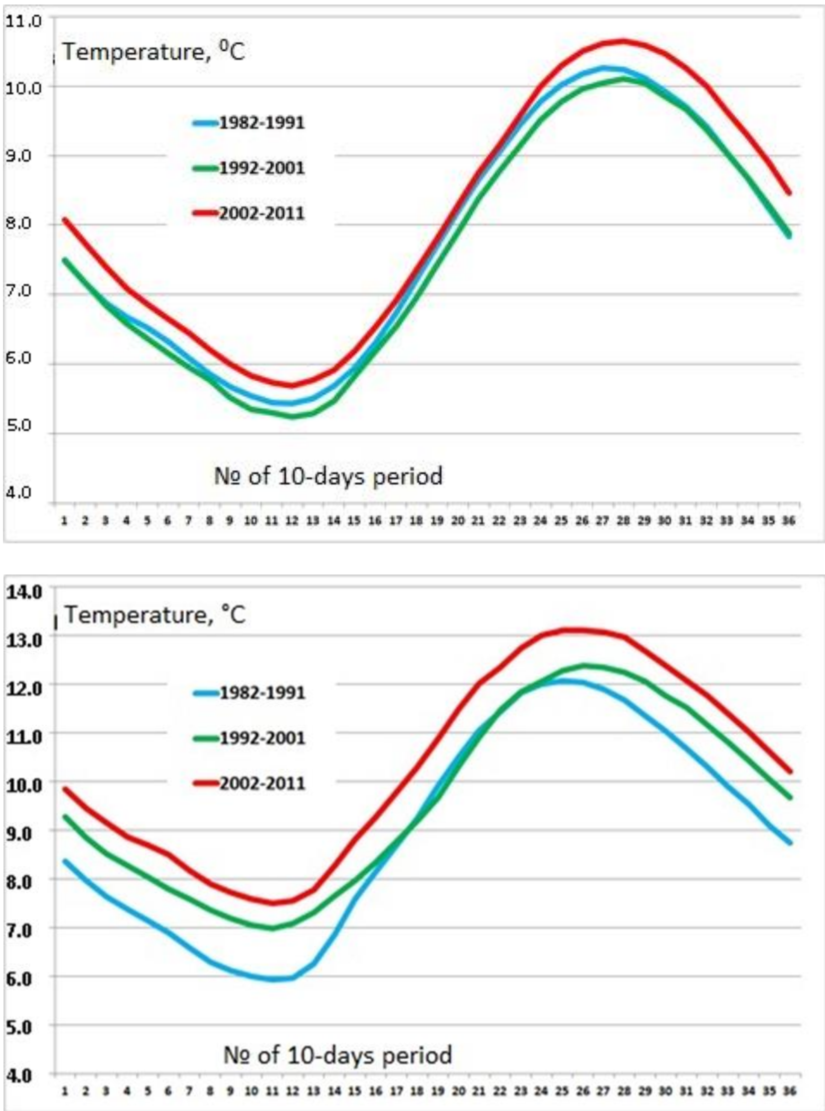


Figure 3. Ground temperature variations at a depth of 160 cm under natural cover and exposed surface, as well as ambient air temperature variations from 1955 till 2012

Horizontal ground heat exchangers of GHPS are often placed at a depth points, close to that of 160 cm. As the type of an underlying surface has a meaningful effect upon variations of the ground temperatures in the annual cycle and dynamics of these long-term variations, while designing the ground heat exchangers it is important to keep in mind, where they are going to be placed: under natural cover or under roads, sidewalks and other surfaces without turf and grass protecting layers in summer and snow cover in winter. [22]

To show long-term trends a temperature curve based upon mean values within consecutive decades (1982–1991, 1992–2001, 2002–2011) at a depth of 400 cm under natural cover and exposed surface (Figure 4) and variations of mean temperature values at the same depth (Figure 5) were examined.

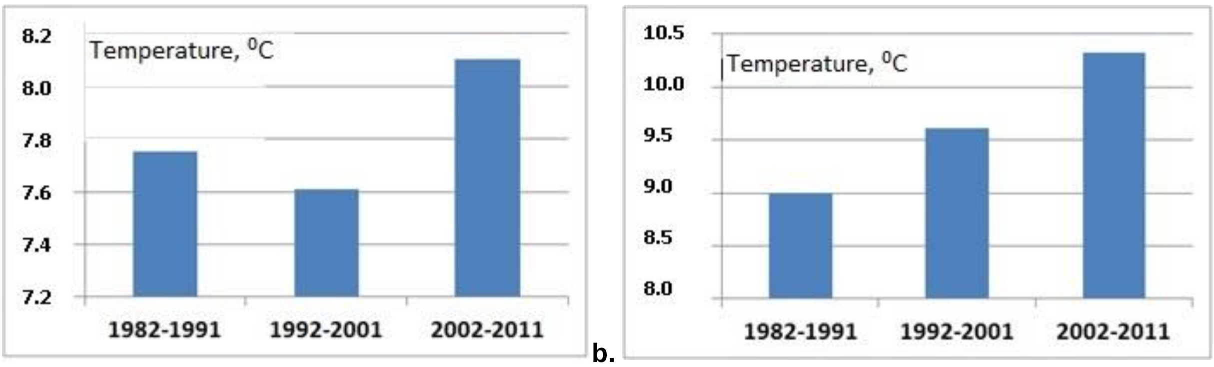
Figures 4 and 5 show that in the last 30 years temperatures at a depth of 400 cm also tend to rise. Within 1992–2001 local fall in temperature under natural cover were observed. It is difficult to explain that fall by ambient air temperature variations because graphs for exposed surface do not reveal a trend like that. On average, a rate of temperature rise at this depth makes about 0.43 °C within 10 years, and this rate is rather uniform.



a.

b.

Figure 4. Ground temperature at a depth of 400 cm, by decades in 1982–1991, 1992–2001 and 2002–2011, correspondingly: a) natural cover, b) exposed surface



a. b.

Figure 5. Variations of mean ground temperatures at a depth of 400 cm, by decades within 1982–1991, 1992–2001 and 2002–2011, correspondingly: a) natural cover, b) exposed surface

Conclusions

Data on years-long measurements of variations of ground temperatures in Moscow have been analyzed, at depths of 20 cm, 40 cm, 60 cm, 120 cm, 160 cm, 240 cm and 320 cm in grounds under

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exposed surface and natural cover. The data were collected from 1982 till 2011. Ground temperatures at a depth of 400 cm and at a depth of 480 cm were obtained using a calculation method.

On the basis of climatic data information has been obtained on a typical temperature curve at depths up to 5 m. These data can be used for ground heat exchangers design and modelling.

As a result of comparison of data for exposed surface and natural cover, impact of an underlying surface upon ground temperature variations in the annual cycle has been demonstrated.

At the depth points under review a steady trend to ground warming has been revealed. At that, a rate of temperature rise under exposed surface is faster than that under natural cover.

Acknowledgement

The Research was conducted by JSC "INSOLAR-INVEST" with the financial support of the Ministry of Education and Science of Russia. Unique identifier of the project RFMEFI57915X0115.

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doi: 10.18720/MCE.72.9

Fiber fine-grained concretes with polyfunctional modifying additives

Дисперсно-армированные мелкозернистые бетоны с полифункциональными модифицирующими добавками

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Key words: fine-grained concretes; modifiers; dispersive fiber; mineral additive; optimization; limit of tensile strength in bending; limit of compressive strength

Ключевые слова: мелкозернистый бетон; модификаторы; дисперсное волокно; минеральная добавка; оптимизация; предел прочности на растяжение при изгибе; предел прочности при сжатии

Abstract. The purpose of the experimental study was to research the efficiency of dispersive reinforcement and modifying of compositions of cement fiber fine-grained concretes with active mineral and chemical additives, and to optimize the developed compositions according to the strength efficiency criteria. Multicriteria optimization of compositions and properties of modified fiber fine-grained concretes is an urgent task in connection with the complexity of their formulations. The experimental study was planned on the basis of a D-optimal plan containing 15 experiments. Upon the experimental results, experimental and statistical models were built to reflect the dependencies of the limit of compressive strength and tensile strength in bending of fiber fine-grained concretes on the type and concentration of modifiers (mix I) and dispersive fibers (mix II). Analysis of study results of saturated D-optimal plan was carried out on triangular diagrams Gibbs-Roseboom, they was built with use polynomial models of "mixture I, mixture II, technology – properties" and allowed to trace influence of 6 to variable factors in two-dimensional space. Optimum fields of variation of fine-grained modified fiber concrete components are identified with use method of experimental-statistical modeling. The efficiency of modifying fine-grained concretes with polyacrylonitrile fiber, astralene-modified basalt microfiber as well as with highly active metakaolin has been established. Use of these modifiers in the compositions allowed to obtain cement composites with a wide range of strength characteristics: from 30 to 53 MPa at compression, from 3.7 to 6.6 MPa for bending tensile.

Аннотация. Целью экспериментального исследования являлось изучение эффективности дисперсного армирования и модифицирования активными минеральными и химическими добавками составов цементных дисперсно-армированных мелкозернистых бетонов, а также оптимизация разработанных составов по прочностным критериям эффективности. Многокритериальная оптимизация составов и свойств дисперсно-армированных модифицированных мелкозернистых бетонов является актуальной задачей в связи с многокомпонентностью их рецептур. Планирование экспериментального исследования осуществлялось на основе D-оптимального плана, содержащего 15 опытов. По результатам эксперимента производилось построение экспериментально-статистических моделей зависимости предела прочности при сжатии и на растяжение при изгибе мелкозернистых бетонов от вида и содержания модифицирующих добавок (смесь I) и дисперсных волокон (смесь II). Анализ результатов исследования осуществлялся по треугольным диаграммам Гиббса-Розебома, построенным по полиномиальным моделям типа «смесь I, смесь II, технология – свойства», позволяющим проследить влияние 6 варьируемых факторов в двухмерном пространстве. С помощью метода экспериментально-статистического моделирования выявлены оптимальные области варьирования компонентов модифицированных мелкозернистых дисперсно-

Низина Т.А., Балыков А.С., Володин В.В., Коровкин Д.И. Дисперсно-армированные мелкозернистые бетоны с полифункциональными модифицирующими добавками // Инженерно-строительный журнал. 2017. № 4(72). С. 73–83.

армированных бетонов. Установлена эффективность модифицирования мелкозернистых бетонов полиакрилонитрильным волокном, модифицированной астраленами базальтовой микрофиброй, а также высокоактивным метакаолином. Использование данных модификаторов в рецептуре позволило получить цементные композиты с широким диапазоном прочностных характеристик: от 30 до 53 МПа – при сжатии, от 3,7 до 6,6 МПа – на растяжение при изгибе.

Introduction

The primary task in designing plans of experimental studies developed in order to obtain compositions of construction materials is an opportunity to provide multi-criteria optimization and to reveal the most reasonable concentration of binding agents, fillers and aggregates, modifying additives, etc.

An increased number of components, e.g., an increased total number of formulation and process factors of cement compositions, results in the need to overcome difficulties caused by so called curse of dimensionality [1]. Furthermore, the optimization of compositions must guarantee high number of operational and process properties of the material, including resource saving criteria. As a rule, optimum coordinates of the quality system criteria under study do not overlap. Solving these multi-criteria tasks is possible in case of integrated realization of reasonable pre-requisites and upon theoretical pre-requisites, as well as by carrying out physical and calculation experiments, and when optimizing their results [2] when the issues related to taking compromise decisions arise.

Currently, a complicated concrete composition that includes 6-7 and more components is becoming a necessary reality [3–7]. Multiple additives and modifiers into concrete (hydrophobic and hydrophilic organic surfactants – super- and hyper- plasticizers [8, 9], fine fillers [5, 10], chemical and active minerals additives of natural and technogenic origin [11–14], dispersive fibers [15–21], including with the use of carbon nanoparticles [22, 23], etc.) are a unique key to solving many process tasks. Multifunctionality and complexity of applied modifiers allows efficiently controlling the structure formation processes at various stages of concrete production [24] and producing composites having high performance characteristics [3–7, 25]: High Performance and Ultra-High Performance Concretes [26–28], High Strength and Ultra-High Strength Concretes [29], Reactive Powder Concretes [30, 31], Self-Compacting Concretes [32], etc.

This causes some complications primarily associated with the cement and additive compatibility and additives' inter-compatibility [6, 33], which is a subject of many studies and discussions at global forums. Reputable Canmet forums distinguish the task of quantitative assessment of complex additives component compatibility between each other and cement/additive compatibility as the primary task [33]. When assessing compatibility, all factors are important, epically the type and concentration of additives in the mix. In various dosages, an additive can be either cure or poison, as the great German doctor Paracelsus once referred to drugs. Increased number of concrete mix components puts the Primum Non Nocere (do no harm) principle to the foreground.

Multi-criteria optimization of compositions and properties of modified fiber fine-grained concretes is undoubtedly an urgent task which requires using mathematical modeling and analysis methods to solve it competently. We believe that the method of experimental and statistical (ES) modeling proposed by V.A. Voznesenskiy [34, 35] and being actively developed at the moment [36, 37] is the most interesting one.

The purpose of this paper was to study the efficiency of dispersive reinforcement and modifying of compositions of cement fiber fine-grained concretes (CFFGC) by active mineral and chemical additives, and to optimize the developed compositions according to the strength efficiency criteria.

Materials and Methods

The experimental study was planned on the basis of a D-optimal plan containing 15 experiments [38]. Two groups of factors varied – the type and concentration of used additives: v_1 (condensed compacted silica fume (CCSF) by Keznetskiye ferrosplavy OJSC); v_2 (white highly active metakaolin (WHAM)); v_3 (Penetron Admix (Admix) sealant for concrete mixes), as well as the type and concentration of the applied fiber: w_1 (polypropylene multi-filament fiber (PP) with the cutting length of 12 mm, diameter of 25÷35 microns, density of 0.91 g/cm³); w_2 (specially treated polyacrylonitrile synthetic fiber FibARM Fiber WB (PAN) with the cutting length of 12 mm, diameter of 14÷31 microns, density of 1.17 ± 0.03 g/cm³); w_3 (astralene-modified basalt microfiber Astroflex-MBM (MBM) with the length of

100÷500 microns, average diameter of 8÷10 microns, bulk density of 800 kg/m^3, astralene concentration of 0.0001 ÷ 0.01 % of the fiber weight). The variance levels of the factors under study are given in Table 1.

In designing the experimental study plan, the following conditions were fulfilled:

$$0 \le v_i \le 1;\ \Sigma v_i = 1;\ i = 1,2,3;$$
$$0 \le w_i \le 1;\ \Sigma w_i = 1;\ i = 1,2,3. \quad (1)$$

In experimental studies, several series of sample prisms 40 x 40 x 160 mm were manufactured from fiber-concrete mixes whose compositions included the following modifiers (apart from those mentioned above): Portland cement grade CEM I 42.5R; fine aggregate – natural quartz sand from the Novostepanovsk pit (Smolniy settlement, Ichalovskiy region, Republic of Mordovia) with the grain size below 5 mm – 65 % of the solid phase weight; superplasticizer Melflux 1641 F – 0.5 % of the binder weight. The changes in the compression strength (Russian State Standard GOST 310.4) and tensile strength in bending (Russian State Standard GOST 310.4) were studied after aging for 28 days.

Table 1. Levels of variation of experimental research factors (% by weight of cement)

Variable factors			Levels of variation		
			0	0.5	1
Type of additive	v_1	CCSF	0	10	20
	v_2	WHAM	0	3	6
	v_3	Admix	0	0.75	1.5
Type of fiber	w_1	PP	0	0.5	1
	w_2	PAN	0	0.75	1.5
	w_3	MBM	0	2.5	5

Results and Discussion

Upon the experimental results, experimental and statistical models were built [38, 39] to reflect the dependencies of the studied physical and mechanical quality indicators of fiber fine-grained concretes on the type and concentration of modifiers (mix I) and dispersive fibers (mix II). The generalized ES model was defined as a reduced polynomial $M_I M_{II} Q$ "mix I, mix II – property" in the following form:

$$\hat{y} = b_{12} \cdot v_1 \cdot v_2 + b_{13} \cdot v_1 \cdot v_3 + b_{23} \cdot v_2 \cdot v_3 + d_{12} \cdot w_1 \cdot w_2 + d_{13} \cdot w_1 \cdot w_3 +$$
$$+ d_{23} \cdot w_2 \cdot w_3 + k_{11} \cdot v_1 \cdot w_1 + k_{21} \cdot v_2 \cdot w_1 + k_{31} \cdot v_3 \cdot w_1 + k_{12} \cdot v_1 \cdot w_2 + \quad (2)$$
$$+ k_{22} \cdot v_2 \cdot w_2 + k_{32} \cdot v_3 \cdot w_2 + k_{13} \cdot v_1 \cdot w_3 + k_{23} \cdot v_2 \cdot w_3 + k_{33} \cdot v_3 \cdot w_3.$$

Two types of models "mix I (modifiers) – property" $(\hat{y}(v_1, v_2, v_3)$; $M_I Q$) and "mix II (dispersive fibers) – properties" $(\hat{y}(w_1, w_2, w_3)$; $M_{II} Q$) were distinguished from the $M_I M_{II} Q$ model with recording the respective group of composition factors [7]. For each type of the models and each physical and mechanical characteristics under study, seven triangle Gibbs-Rosebom diagrams were built in the form of 2D maps of level lines by using Statistica 10.0.1011 software.

To further analyze the impact of modifiers on the properties of cement composites, a generalizing indicators was introduced – a numerical characteristic of the property field in the form of an absolute values of the studied indicator corresponding to its maximum $\hat{y}_{\max}$. ES-models "mix I – property maximum" ($\hat{y}_{\max}(v_1, v_2, v_3)$; $M_I Q_{max}$) and "mix II – property maximum" $(\hat{y}_{\max}(w_1, w_2, w_3)$; $M_{II} Q_{max}$) reflecting the connection between the varied factors and the maximums of the properties under study represent polynomial equations of the following form [37, 40]:

$$\hat{y}_{\max} = b_1 \cdot v_1 + b_2 \cdot v_2 + b_3 \cdot v_3 + d_{12} \cdot v_1 \cdot v_2 + d_{13} \cdot v_1 \cdot v_3 + d_{23} \cdot v_2 \cdot v_3 + k_{123} \cdot v_1 \cdot v_2 \cdot v_3; \quad (3)$$

$$\hat{y}_{\max} = b_1 \cdot w_1 + b_2 \cdot w_2 + b_3 \cdot w_3 + d_{12} \cdot w_1 \cdot w_2 + d_{13} \cdot w_1 \cdot w_3 + d_{23} \cdot w_2 \cdot w_3 + k_{123} \cdot w_1 \cdot w_2 \cdot w_3. \quad (4)$$

By using the data obtained, for each of the studied characteristic, two triangle Gibbs-Rosebome diagrams were built to reflect respective systems $\hat{y}_{\max}(v_1, v_2, v_3)$ and $\hat{y}_{\max}(w_1, w_2, w_3)$ [37, 40]. For each of the analyzed parameters, the pairs of models $\hat{y}_{\max}(v_1, v_2, v_3)$ and $\hat{y}(w_1, w_2, w_3)$ as well as $\hat{y}_{\max}(w_1, w_2, w_3)$ and $\hat{y}(v_1, v_2, v_3)$ are then synthesized. Secondary models $\hat{y}_{\max(v)}(w)$ and $\hat{y}_{\max(w)}(v)$ were formed representing a triangle (Figs. 1, 2) sliding along the bearing triangle and fixed in seven centroid points (3 corners + 3 side centers + center of gravity) [37, 39, 40].

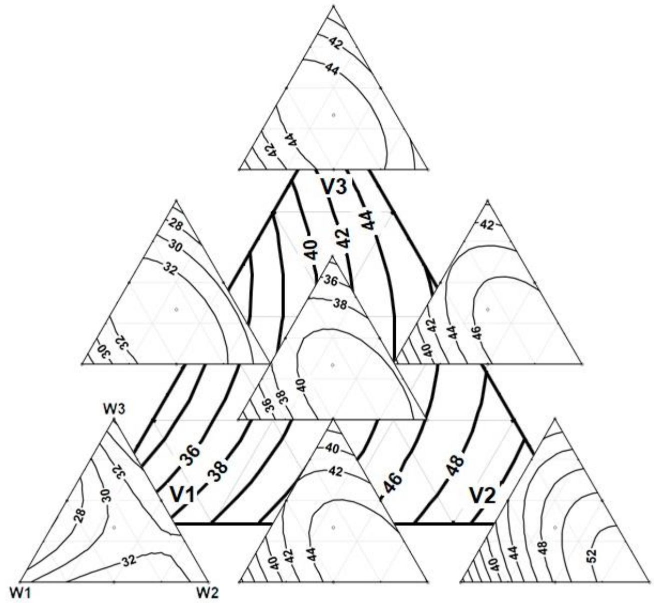


Figure 1. Diagrams "dispersive fibers – property" and contours of maximum values of the limit of compression strength of cement fiber fine-grained concretes (CFFGC) on triangle "modifying additives – property" [39, 40]

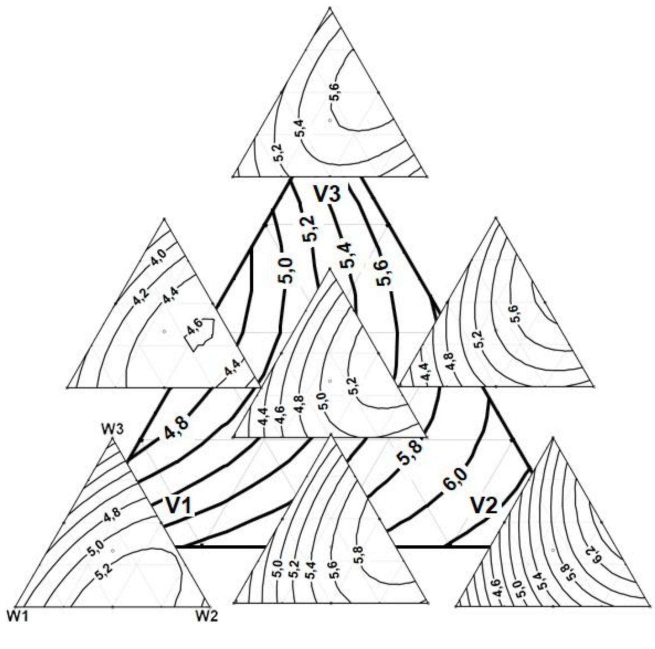
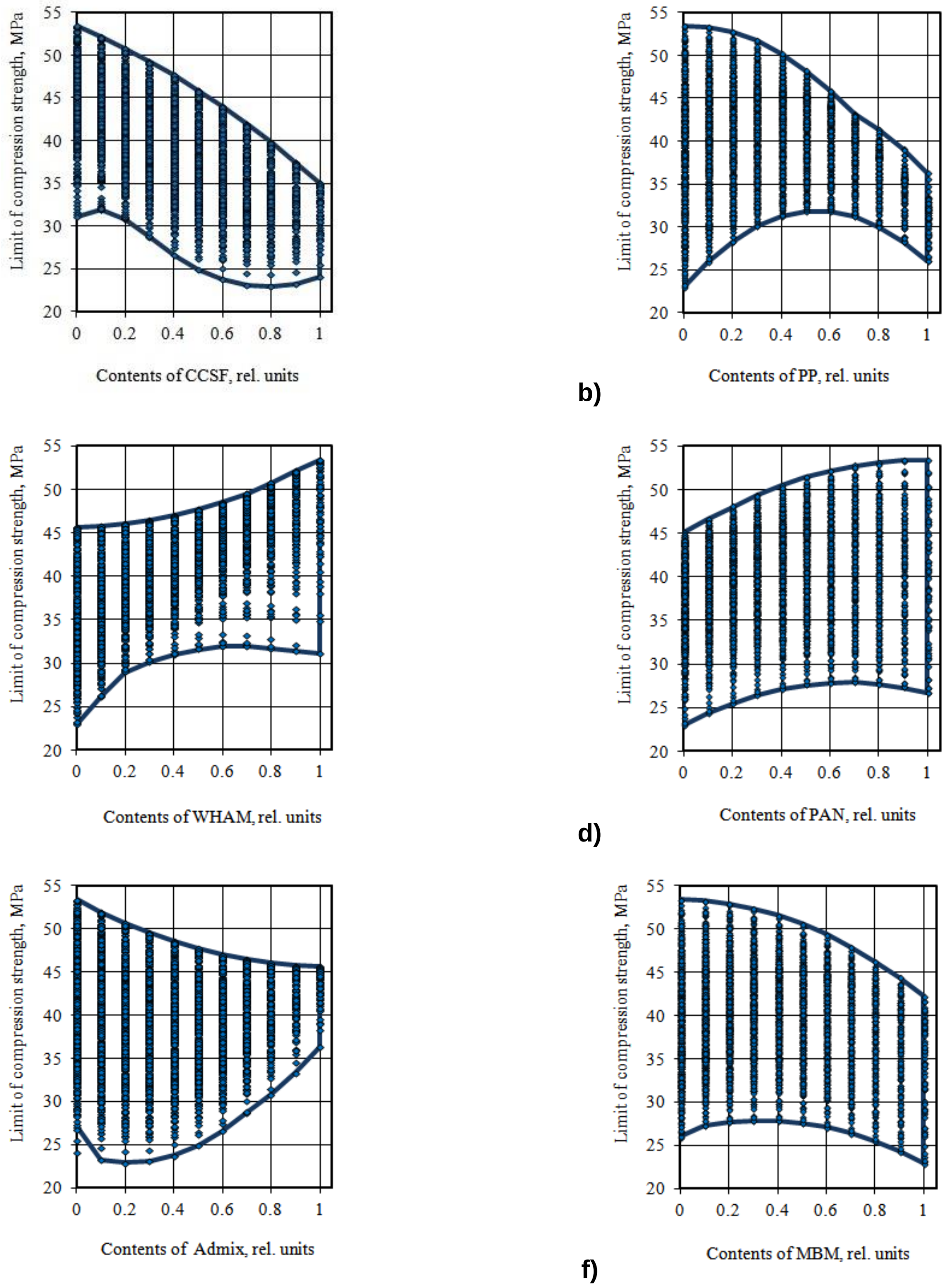


Figure 2. Diagrams "dispersive fibers – property" and contours of maximum values of the limit of tensile strength in bending of cement fiber fine-grained concretes (CFFGC) on triangle "modifying additives – property" [40]

The resulting secondary models $\hat{y}_{\max(v)}(w)$ and $\hat{y}_{\max(w)}(v)$ are highly informative multi-factor experimental and statistical models comparable to well-known analogues that are widely represented in the work [38]. These ES-models enable establishing a link and quantitative relations between investigated strength characteristics of the cement composites, process and operational factors with simultaneous minimization of labor costs and getting maximum information concerning the studied object.



a) b) c) d) e) f)

Figure 3. Field of allowable values of the limit of compression strength of cement fiber fine-grained concretes (CFFGC) based on content of modifying additives (a, c, e) and dispersive fibers (b, d, f) in compositions

At the final stage of experimental studies, the compositions of modified fiber fine-grained concretes were optimized. By using an ES-model as a polynomial equation (2), for each of the six formulation modifiers, generalizing values of the characteristics under study were obtained – the areas of permitted solutions describing the technology stability (Figs. 3, 4). These areas are confined by minimal and maximum possible values of controlled properties depending on the concentration of the modifier analyzed.

Figures 3 and 4 represent the areas of permitted solutions for the studied mechanical indicators of cement composites – compression strength and tensile strength in bending of fiber fine-grained concretes. It has been found that the increased share of metakaolin in the total mass of active mineral additives causes the compression strength (Fig. 3, c) and the tensile strength in bending (Fig. 4, c) of cement composites to icnrease. For its maximum concentration in composites (6 % of the Portland cement weight), we can produce fiber concretes with a wide range of strength characteristics: 30 to 53 MPa for compression strength and 3.7 to 6.6 MPa for tensile strength in bending.

Introducing condensed compacted silica fume into concrete mixes causes the strength of cement composites to decrease, which shows the CCSF negative impact on the structure formation processes of cement composites as compared to other types of applied additives (Figs. 3, 4, a).

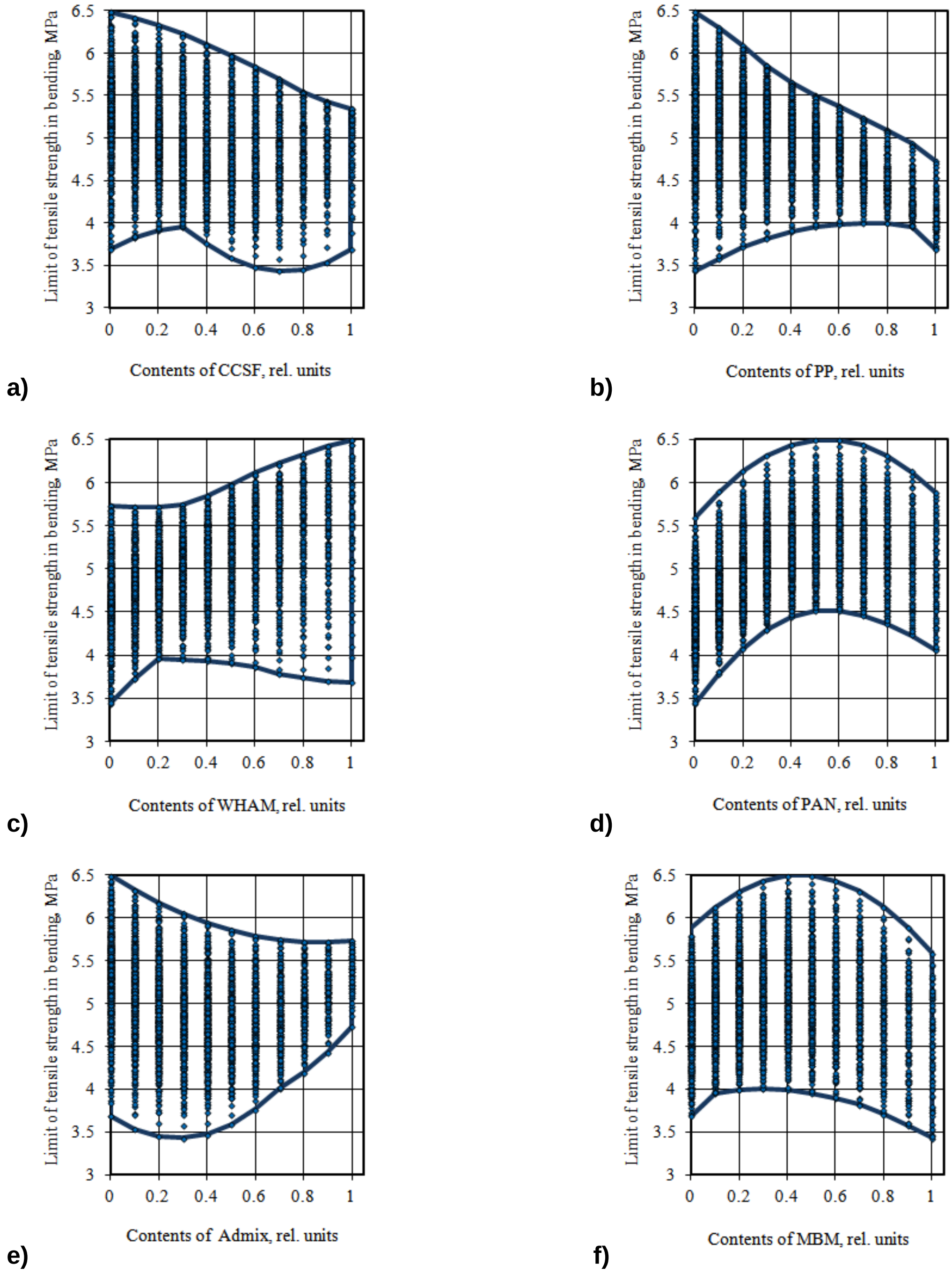


Figure 4. Field of allowable values of the limit of tensile strength in bending of cement fiber fine-grained concretes (CFFGC) based on content of modifying additives (a, c, e) and dispersive fibers (b, d, f) in compositions

Mounted according to the study results the WHAM efficiency as compared to CCSF is confirmed by data of the work [13]. This is explained by: higher (about 2–2.5 times) pozzolanic activity of metakaolin; different chemical nature of additives (silicate in CCSF and aluminosilicate in WHAM); accelerated reaction between WHAM and lime as compared to CCSF, which ensures its effective binding during the first day of setting; higher plasticity and performance of concrete and mortar mixes, no surface adhesiveness of concrete with WHAM typical of concretes with CCSF; lower water demand of mixes with WHAM, meaning lower consumption of superplasticizers needed to achieve the same mobility of concrete mixes.

When increasing the concentration by means of Admix mineral modifier, the maximum values of strength are somewhat decreased, and the minimum possible values are increased (Figs. 3, 4, e); the area of permitted values is substantially reduced for the maximum filling of this additive (1.5 % of the Portland cement mass) – 36 to 46 MPa for compression strength and 4.7 to 5.7 MPa to tensile strength in bending.

This effect can be explained by newly formed calcium hydrosulfoaluminates and hydrocarboaluminates when Penetron Admix components interact with cement hydration products. These new formations appearing when the volume grows, along with the initial thickening of the structure, may cause negative internal stresses in case of non-optimal use. This is confirmed by the study results of other authors, in particular [14]. Thus, taking into account that it is necessary to accurately select the dosage of this mineral additive in order to rationally control the crystallization process and form the structure of cement composites.

By analyzing the areas of permitted solutions when studying the effect of dispersive fibers on the tensile strength in bending, we can make a conclusion on the efficiency of reinforcing fine-grained concretes with PAN-fibers and MBM-fibers (Figs. 4, d, f), with the highest strength of 6.5 MPa obtained when using a complex of PAN+MBM fibers with equal (50 %) shares of these fibers. This proves the appropriateness and efficiency of the multi-level reinforcement of fine-grained concretes by using carbon nanostructures (applying polyacrylonitrile fiber is reinforcement on the macroscale structural level; applying astralene-modified basalt microfiber is reinforcement on the upper macroscale level).

Presented in [17–23] the study results confirm the efficiency of the multi-level reinforcement (including the use of carbon nanoparticles), this is based on the hypothesis of the proportionality of the reinforcing elements to the "blocked" cracks of the corresponding level of structure (micro-, meso-, macro-).

Increasing the percentage of polypropylene fiber when reinforcing composites results in decreased maximum possible strength (Figs. 3, 4, b), with the area of permitted solutions of this indicators decreased – 3.6 ÷ 4.8 MPa for tensile strength in bending and 26÷36 MPa for compression strength.

Conclusions

To produce materials of various functional purpose with a wide range of properties, a systemic approach is needed to be applied when selecting initial materials, composite production technologies, planning and analyzing methods for experimental studies. An important role is played by informative multi-factor experimental and statistical models that enable establishing a link and quantitative relations between material quality indicators, its structural parameters formulation, process and operational factors with simultaneous minimization of labor costs and getting maximum information concerning the studied object.

As result of experimental studies:

1. The efficiency of modifiers and dispersive fibers was assessed for a number of physical and mechanical properties (the limit of compression strength (Russian State Standard GOST 310.4) and the limit of tensile strength in bending (GOST 310.4) after aging for 28 days) in order to produce concretes of various functional purpose.

2. Secondary models $\hat{y}_{\max(v)}(w)$ and $\hat{y}_{\max(w)}(v)$ were formed representing a triangle (Figs. 1, 2) sliding along the bearing triangle and fixed in seven centroid points (3 corners + 3 side centers + center of gravity).

3. Thanks to optimization of compositions of fiber fine-grained concretes, the areas of permitted solutions were defined, reflecting the possible range of changes in the quality indicator under study depending on the formulation and the percentage of each of the applied modifiers.

4. By analyzing the areas of permitted solutions when studying the effect of modifying additives on the physical and mechanical properties, we can make a conclusion on the efficiency of modifying fine-grained concretes with white highly active metakaolin (WHAM) (Figs. 3, 4, c). For its maximum concentration in composites (6 % of the Portland cement weight), we can produce fiber concretes with a wide range of strength characteristics: 30 to 53 MPa for compression strength and 3.7 to 6.6 MPa for tensile strength in bending.

5. By analyzing the areas of permitted solutions when studying the effect of dispersive fibers on the tensile strength in bending, we can make a conclusion on the efficiency of reinforcing fine-grained concretes with PAN-fibers and MBM-fibers (Figs. 4, d, f), with the highest strength of 6.5 MPa obtained when using a complex of PAN+MBM fibers with equal (50 %) shares of these fibers.

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Инженерно-строительный институт
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Приглашает специалистов проектных и строительных организаций,
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на курсы профессиональной переподготовки (от 500 часов)
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П-01 «Промышленное и гражданское строительство»

Программа включает учебные разделы:

- ***Основы строительного дела***
- ***Инженерное оборудование зданий и сооружений***
- ***Технология и контроль качества строительства***
- ***Основы проектирования зданий и сооружений***
- ***Автоматизация проектных работ с использованием AutoCAD***
- ***Автоматизация сметного дела в строительстве***
- ***Управление строительной организацией***
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Программа включает учебные разделы:

- ***Основы строительного дела***
- ***Инженерное оборудование зданий и сооружений***
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- ***Управление строительной организацией***
- ***Экономика и ценообразование в строительстве***
- ***Управление строительной организацией***
- ***Организация, управление и планирование в строительстве***
- ***Автоматизация сметного дела в строительстве***

П-03 «Инженерные системы зданий и сооружений»

Программа включает учебные разделы:

- ***Основы механики жидкости и газа***
- ***Инженерное оборудование зданий и сооружений***
- ***Проектирование, монтаж и эксплуатация систем вентиляции и кондиционирования***
- ***Проектирование, монтаж и эксплуатация систем отопления и теплоснабжения***
- ***Проектирование, монтаж и эксплуатация систем водоснабжения и водоотведения***
- ***Автоматизация проектных работ с использованием AutoCAD***
- ***Электроснабжение и электрооборудование объектов***

П-04 «Проектирование и конструирование зданий и сооружений»

Программа включает учебные разделы:

- ***Основы сопротивления материалов и механики стержневых систем***
- ***Проектирование и расчет оснований и фундаментов зданий и сооружений***
- ***Проектирование и расчет железобетонных конструкций***
- ***Проектирование и расчет металлических конструкций***
- ***Проектирование зданий и сооружений с использованием AutoCAD***
- ***Расчет строительных конструкций с использованием SCAD Office***

П-05 «Контроль качества строительства»

Программа включает учебные разделы:

- ***Основы строительного дела***
- ***Инженерное оборудование зданий и сооружений***
- ***Технология и контроль качества строительства***
- ***Проектирование и расчет железобетонных конструкций***
- ***Проектирование и расчет металлических конструкций***
- ***Обследование строительных конструкций зданий и сооружений***
- ***Выполнение функций технического заказчика и генерального подрядчика***

По окончании курса слушателю выдается диплом о профессиональной переподготовке установленного образца, <u>дающий право на ведение профессиональной деятельности</u>