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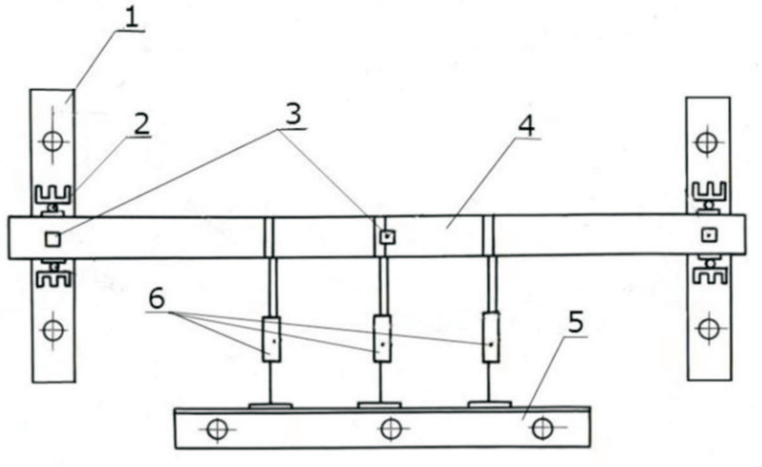

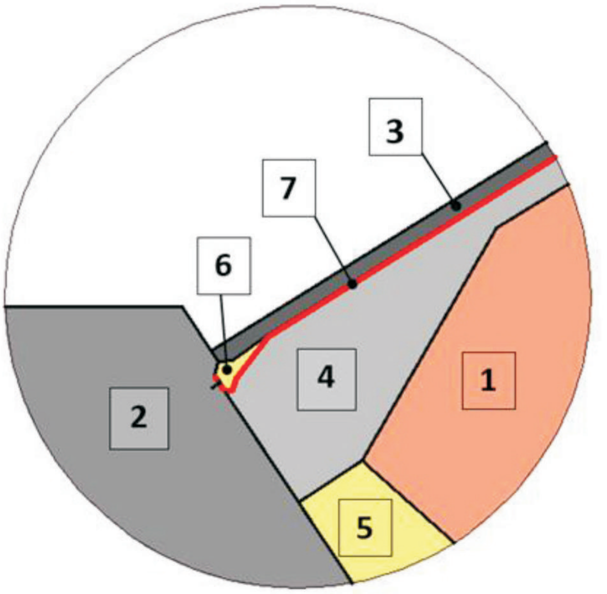

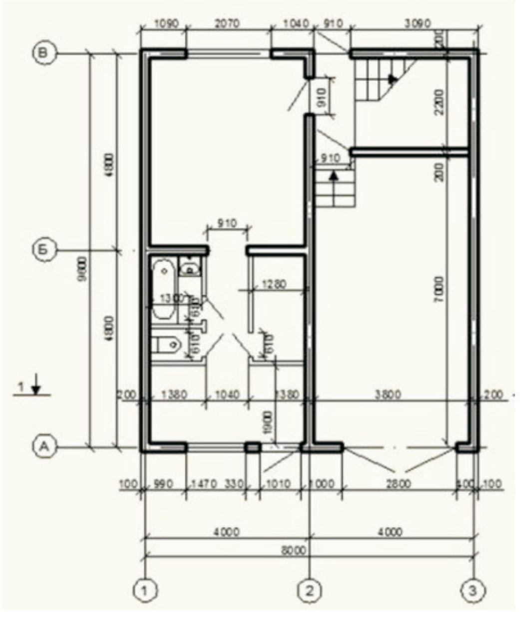

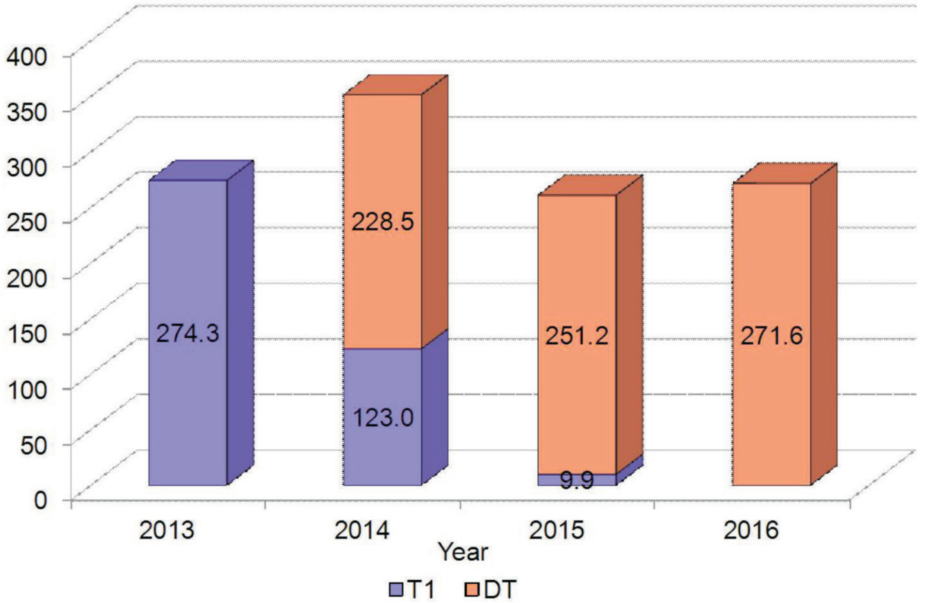

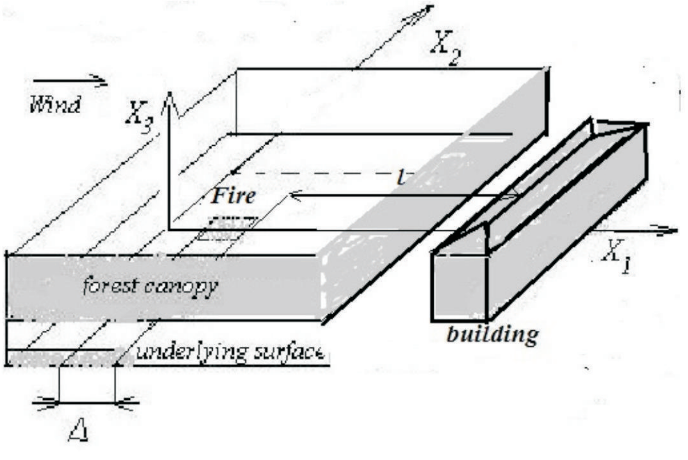

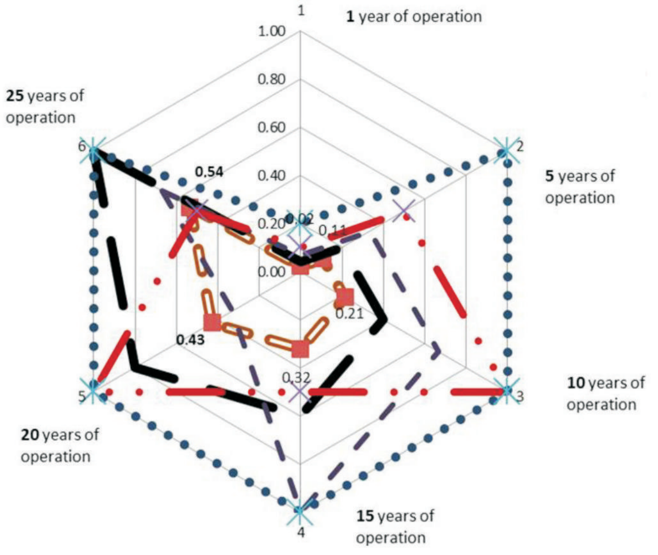

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Behaviour of concrete with a disperse reinforcement under dynamic loads

Поведение бетона с дисперсным армированием при динамических воздействиях

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Key words: disperse reinforcement; fiber concrete; combined reinforcement; experimental study; alternating dynamic highly intensive impact

Ключевые слова: дисперсное армирование; фибробетон; комбинированное армирование; фиброжелезобетон; экспериментальное исследование; знакопеременное динамическое воздействии большой интенсивности

Abstract. Disperse reinforcement of concrete greatly contributes to the properties of the latter. Most research has dealt with the properties of disperse reinforced concrete and influence of disperse reinforcement on structures under a static load of one sign or for a regular dynamic load that is not highly intensive. In practice there might well be alternating dynamic impacts that are highly intensive and over the calculated ones, e.g., seismic ones. The paper presents the results of an experiment study of beam structures with a disperse and combined reinforcement under an alternating highly intense dynamic impact. A method of performing an experimental study of beam elements with a disperse and combined reinforcement under an alternating dynamic highly intensive impact that is based on the use of a universal dynamic stand with extra equipment. The results of the experimental studies of cubes and prisms for static and dynamic compression are discussed. The outcomes of the study of the operation of beam elements with a disperse and combined reinforcement under an alternating dynamic highly intensive impact are presented. The operation of fiber concrete structures and ferroconcrete beam elements under similar impacts are compared. The presented results of the experimental studies allow us to conclude that a disperse reinforcement has a great influence on the operation of structures with an alternating dynamic highly intensive impact and the positive effect of a combined reinforcement of structures operating under such impacts. The use of a disperse reinforcement in structures operating under alternating dynamic highly intensive impacts would enable the resistance of structures to resist these impacts.

Аннотация. Дисперсное армирование бетона значительно улучшает свойства последнего. Значительная часть проводимых исследований посвящена изучению свойств дисперсно-армированных бетонов и изучение влияния дисперсного армирования на работу конструкций при статической нагрузке одного знака или при знакопеременной динамической нагрузке небольшой интенсивности. На практике периодически возникают знакопеременные динамические воздействия большой интенсивности, превышающие расчетные, например, сейсмические. В работе представлены результаты экспериментального исследования работы балочных конструкций с дисперсным и комбинированным армированием при знакопеременном динамическом воздействии большой интенсивности. Приведена методика проведения экспериментального исследования работы балочных элементов с дисперсным и комбинированным армированием при знакопеременном динамическом воздействии большой интенсивности, основанная на использовании дооборудованного универсального динамического стенда. Рассмотрены результаты экспериментальных исследований кубов и призм на статическое и динамическое сжатие. Приведены результаты исследования работы балочных элементов с

Николенко С.Д., Сушко Е.А., Сазонова С.А., Однолько А.А., Манохин В.Я. Поведение бетона с дисперсным армированием при динамических воздействиях // Инженерно-строительный журнал. 2017. № 7(75). С. 3–14.

дисперсным и комбинированным армированием на знакопеременном динамическом воздействии большой интенсивности. Выполнено сравнение работы фиброжелезобетонных конструкций с работой железобетонных балочных элементов при аналогичных воздействиях. Представленные результаты экспериментальных исследований позволяют сделать вывод о значительном влиянии дисперсного армирования на работу конструкций при знакопеременном динамическом воздействии большой интенсивности и целесообразности комбинированного армирования конструкций, работающих при таких воздействиях. Применение дисперсного армирования в конструкциях работающих при знакопеременных динамических воздействиях большой интенсивности позволит увеличить сопротивляемость конструкций таким воздействиям.

Introduction

In many industrial developed countries there is a lot of focus on studies of concrete reinforced with different fibres: steel, glass, basalt, polypropylene, etc. Such concretes are called disperse reinforced concretes or fibre concretes. According to the data, they have improved characteristics compared to a regular concrete and outperform it by 2 or 4 times in bending. In the standard literature [1, 2] steel fibreconcrete structures depending on their reinforcement are classed into structures with fibre reinforcement (fibreconcrete ones) that are reinforced using only steel fibre that is evenly distributed along the element; with a combined reinforcement (fibreferroconcrete ones) that are reinforced using steel fibres in combination with a steel rod or wire reinforcement.

Studies of the properties of disperse reinforced concretes are numerous. In [3] there are the results of studies of physical and mechanical characteristics of dispersing reinforced fine-grained concretes with multifunctional modifying additives. Studies of ceramsitefibreferroconcrete elements that has an extra rough basalt fibre reinforcement are described in [4]. In [5] the combined and individual effects of a polypropylene and glass fibre on the mechanical properties and rheological characteristics of a self-sealing concrete are investigated. The combined effect of a polypropylene and glass fibre improves the compressive, tensile and bending strength.

However, most studies have investigated a metal fibre. A full account of the influence of a disperse reinforcement on the properties of concrete can be found in [6]. It is concluded that the use of steel fibres for strong and durable structures and concrete reinforcement is agreed to be promising all over the world. 300 thousands of tonnes of fibre is used for concrete reinforcement, 50 % of them are steel [7].

The investigations described in [8] show the efficiency of steel fibres. In particular it was suggested that the minimum volume of steel, glass and polypropylene fibres in a concrete matrix that have the best performance is about 0.31 %, 0.40 % and 0.75 % respectively for each type. The efficiency of steel fibres in beam structures for pure bending is described in the papers by H.V. Dwarakanath, T.S. Nagaraj [9]. The results of the studies mentioned by Job Thomas, AnanthRamaswamy [10] indicate that the mutual effect of a metal fibre and concrete matrix largely contributes to the mechanical properties that are improved by a fibre introduced into a concrete matrix.

The results of static studies of fibreferroconcrete beams and their comparisons by other authors are accounted for in [11]. In [12] there are the results of experimental studies of fibreconcrete beams with three types of fibres: a steel fibre with a curved end, a fibre from wavy steel and polypropylene fibres.

In [13] according to the above studies, the introduction of a steel fibre into concrete is reported to improve the limit bending strength of a material. This bending strength goes up as does the amount of fibre in concrete. Quick ash is used as an additive. In [13] there are the results of a study of a fibreconcrete under a dynamic impact load. The outcomes of the investigation of the properties of fibreconcrete with different fibres are also described in [14]. It is also indicated that concrete must be designed so that it saturates the energy of natural forces such as earthquakes.

An extensive analysis of studies of disperse reinforced structures is presented in a monography by F.N. Rabinovich [15].In [16] the methods and results of experimental studies of beam fibreconcrete elements with a synthetic fibrecan be found.

Based on the theoretical and experimental studies of physical and mechanical characteristics of disperse reinforced concrete in a wide range of volumetric saturation with fibres, Yu.V. Pukharenko [17] contributed to the ideas about the structure of fibreconcrete. He also found that the main structural component of disperse reinforced concrete is fibre. In addition, the general method of designing the composition of fibreconcrete was developed and features of its use for different types of concrete reinforced with steel and non-metal fibresare investigated.

Nikolenko S.D., Sushko E.A., Sazonova S.A., Odnolko A.A., Manokhin V.Ya. Behaviour of concrete with a disperse reinforcement under dynamic loads. *Magazine of Civil Engineering*. 2017. No. 7. Pp. 3–14. doi: 10.18720/MCE.75.1.

The following papers deal with structures under a dynamic alternating impact. In [18] experimental studies of ferroconcrete beams reinforced with fibre and their effect on alternating dynamic loads are presented. The use of metal fibresis shown to have a positive influence of the energy capacity of a failure. In [19] the results of experimental studies of ferroconcrete beams that are statistically determined or not determined with a regular and pre-stressed reinforcement under alternating low-cycle loads of a high level, e.g., seismic ones are described.

The results of dynamic studies of beam elements of a transverse section sized 150 x 200 mm with a pure span of 1 m are identified in [20]. The samples contained metal fibres with the amount of 40 kg/m^3 and 80 kg/m^3. Fibres from stainless steel and carbonaceous steel wire were involved. The experiments were conducted under a static load up to 100 kN and dynamic load up to 80 KN. The results suggest that metal fibres contribute greatly to the properties of fibreconcrete elements under an alternating deformation.

Studies addressing improvement of crack resistance of ferroconcrete structures performed by N.I. Vatin [21, 22] seem promising. They also explore possible improvement of crack resistance of concrete and nanoconcrete in structures by applying pre-stresses. They are generated by placing a reinforced rope in a structure along the distribution of bending moments. The rope that is pulled is pre-stressed without the cohesion with concrete to allow this method to be employed in structures with small sections.

However, despite a lot of research and materials, most of them have to do with the operation of disperse reinforced construction under a static load of one sign or alternating loads that are not highly intensive. In practice most structures operate under alternating dynamic loads, e.g., under seismic loads that are highly intensive. The properties of the material of a structure are essential to the strength of buildings subjected to seismic loads.

It seems hardly possible to identify seismic forces and their directions on buildings as the Earth's movement during an earthquake depends on a number of factors. While selecting the intensity of an impact on sample structures it is important to note that actual intensity of earthquakes is significantly over the expected one. Therefore in the experiments a special shock stand was employed that allowed one to generate an alternating dynamic highly intensive load on the sample structure.

The objective was to evaluate the effect of a disperse reinforcement of concrete on the operation of structures under an alternating dynamic highly intensive load. For that experimental studies were planned of the operation of fibreconcrete and fibreferroconcrete beam elements under an alternating dynamic highly intensive load and their operation was compared with that of ferroconcrete beam elements.

Experimental sample

All of the sample structures were beams with the length of 1650 mm. Their transverse section was 100 x 100 mm. These sizes were chosen based on the capacity of the laboratory equipment, requirements for the absolute accuracy of measurements and geometric size of the model and actual structures. 4 series of beams were designed with the following reinforcement. The beams of series 1 were reinforced as shown in Figure 1. The beams of series 2 and series 4 had a combined reinforcement as shown in Figure 2, i.e. there was only a transverse operating reinforcement left in them and the rest was substituted with a disperse one at the rate of 1 % and 2 % in the volume respectively.

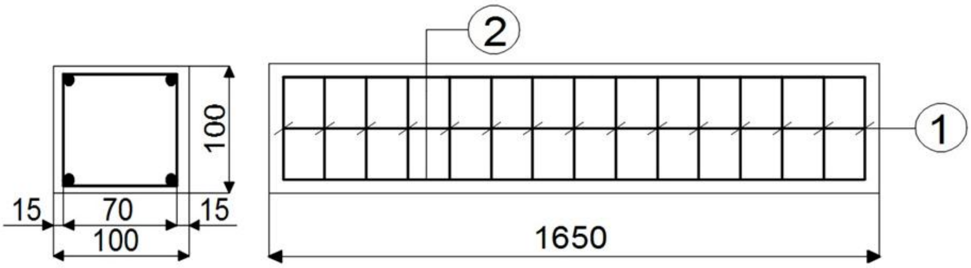

Figure 1. Scheme of the reinforcement of ferroconcrete beams:
1 – reinforcement A – III, diameter 6mm, length 80 mm, 18;
2 – reinforcement A – III, diameter 6mm, length 1620 mm, 4

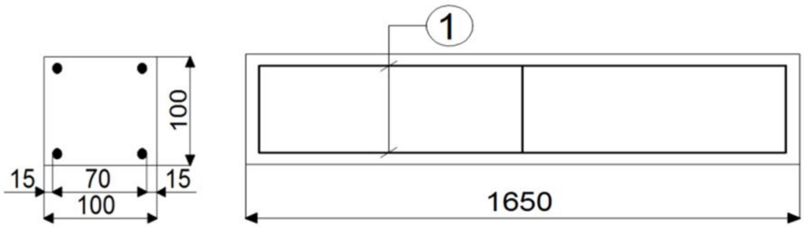

Figure 2. Scheme of reinforcement of fibreconcrete beams:
1 – reinforcement A – III, diameter 6mm, length 1620 mm, 4

The beams of series 3 and series 5 had no rod reinforcement but only a disperse one at the rate of 2 % and 1 % in the volume respectively. Hence the beams of series 3 and series 5 are not indicated in the schemes. Three sample beams were tested in each of the series.

Overall the amount of reinforcement in the beams of the first three series was approximately identical (Table 1). The beams of series 4 and series 5 had an increased and decreased amount of reinforcement respectively.

Table 1. Characteristics of reinforcement of the experimental beams

№ of the series	Names of the beams	Percentage of road reinforcement	Amount of rod reinforcement, kg	Percentage of fibre reinforcement	Amount of fibre reinforcement, kg	Total amount of reinforcement, kg
1	Fibreconcrete	0.565	2.5	-	-	2.5
2	Fibreferroconcrete with 1.0 % of fibres	0.565	1.5	1.0	1.2	2.7
3	Fibreconcrete with 2.0 % of fibres	-	-	2.0	2.4	2.4
4	Fibreferroconcrete with 2.0 % of fibres	0.565	1.5	2.0	2.4	3.9
5	Fibreconcrete with 1.0 % of fibres	-	-	1.0	1.2	1.2

In order to make the samples, a fine-grained cement mix with the composition Cement:Sand 1:2.5. The binder was Portland cement M400. The ratio Water/Cement was 0.4. For disperse reinforcement a fibre was used made from a regular low-carboneous wire of a with the diameter 0.8 mm, density 78.5 g/cm^3. Along with cutting the wire fibre on a special machine, their surface was made profiled using rollers. The scheme of the profile of the fibresis presented in Figure 3.

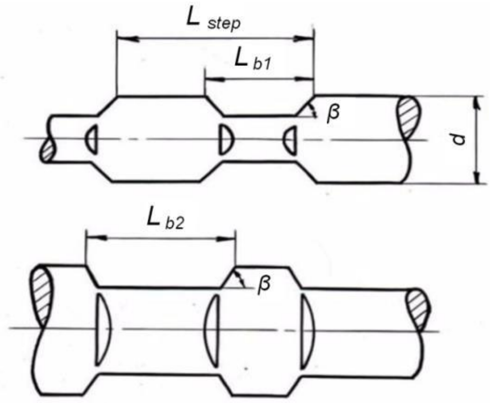

Figure 3. Scheme of the profile of the fibre in two planes:
L_{step} – profiling step of the fibre is 4 mm; L_{b1} – profiling length of the fibre in plane 1 is 1.9 mm;
L_{b2} – profiling length of the fibre in plane 2 is 2.4 mm; β – the bearing angle is 45^0;
***d* – diameter of the fibre**

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The length of the fibres was 36 mm due to being able to randomly arrange them in the experimental samples. For larger fibres for these sizes of the transverse sections of the experimental beams when the fibresare distributed in the beams, they are distinctively “squeezed” and there is hardly any volumetric reinforcement left. Hence there is no major advantage of disperse reinforcement any more either. The recommendations on the distance between rods of transverse reinforcements in structures with a combined reinforcement with no less than 1.5 fibre lengths, optimally no less than 2 fibre lengths.

Despite the fact that according to a lot of researchers, the accepted ratio of the fibre length and diameter (*l/d)* is not optimal and lately fibre with the *l/d* 45-60 has been used. E.g., fibre from a cut steel wire “Dramix” produced by the Belgian group Bekaert N.V. with the brand Dramix 3D 45/50BL has the length of 50 mm, diameter of 1.05 mm. *l/d* that is indicated in the fibre labelling is 45. The accepted geometric parameters of the fibre are in agreement with the recommendations by the Committee 544 [23]. In particular, the ratio of the fibre length to the diameter is from about 20 to 100, making a fibre by cutting of a regular wire with the diameter 0.25–1.00 mm. The results of the experiment also showed that for the ratio *l/d* a disperse reinforcement also shows its major advantages.

In addition, the advantages in the technology of producing structures with a fibre of such parameters are out of question. The introduction of such fibres into a concrete mix requires no extra equipment without leaving them clumped. When fibres with the ratio $l/d \geq 100$ are used, additional introduction of concrete is essential in order to avoid clumping of fibres.

A concrete mix was prepared first with no fibres and then they were gradually introduced.

In order to identify the strength characteristics of concrete and fibreconcrete in the beams and fibreconcrete in the beams of each mixing, three prisms and three cubes were made (100 x 100 x 400 mm) (150 x 150 x 150 mm). In order to get comparative characteristics for fibres of the accepted geometric parameters, prisms and cubes were additionally produced for dynamic studies.

Research method

As providing complete protection of buildings in the event of severe earthquakes hardly seems possible, the guidelines [24] make it possible to damage individual elements as long as people safety is guaranteed. Based on that the intensity and scheme of the experiments were chosen as follows (Figure 4).

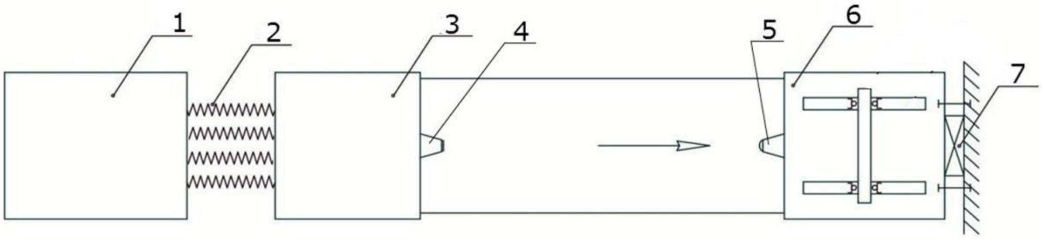

Figure 4. Scheme of dynamic studies (from the top):
1 – mechanism of trapping the shock cart and compressing the springs;
2 – springs of the stand;3 – shock cart; 4, 5 – scheme of centering the shock spot;
6 – operating cart with the tested beam; 7 – elastic filling

The tested structure was safely put on the operating cart (6) using a special structure of the beam clamping. Using the mechanism of trapping the shock cart and compression of springs (1) while using the hydraulics of the stand spring (2) was compressed down to a specified level. The shock cart (3) was set to motion due to the force of straightening of the springs. Applying some shock to the operating cart using the device for centering of the shock (4, 5), it impacted the tested beam by accelerating through the supports of a special structure. In order to obtain the second semi-wave of the acceleration of the supports of the beam between the operating cart and support, an elastic filling was provided (6).

Dynamic tests of the beam elements were conducted on a universal dynamic stand (Figure 5). The scheme of placing the beam and sensors on the operating cart is presented in Figure 6.

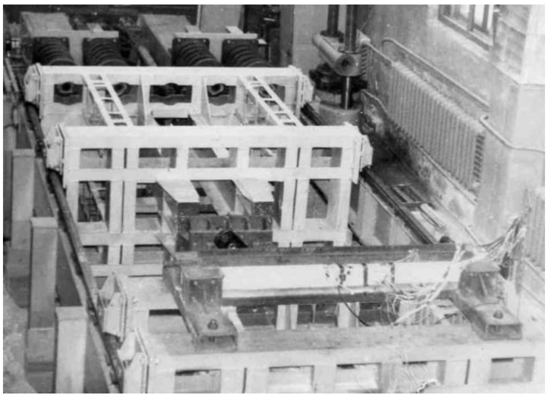

Figure 5. View of the universal dynamic stand with the tested structure

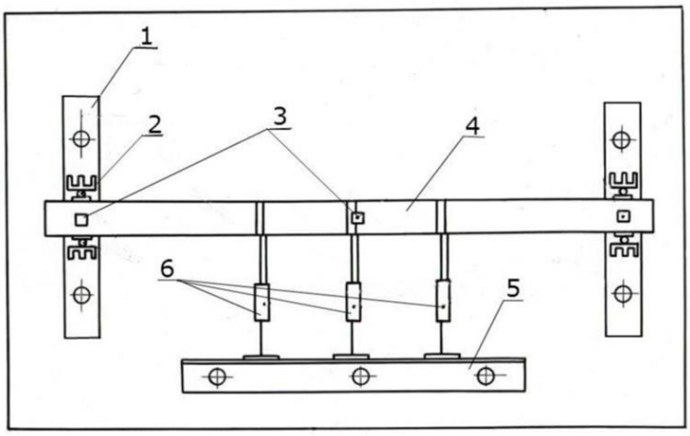

Figure 6. Scheme of placing the beam and sensors on the operating cart (from the top):
1 – the beam attached to the cart;2 – support;3 – acceleration sensors;4 – tested beam;
5 – angle for placing the displacement sensor;6 – displacement sensors

Throughout the experiment the bendings of the beam were measured at three spots in the middle of the span, accelerations on the support and the centre of the beam span, deformation of the reinforcement in the centre of the beam span.

The measurements were performed using acceleration and displacement sensors and the deformation of reinforcement using strain gauges. The resulting values were fixed by means of intensifiers and oscillographs. In order to observe cracks and their development, the surface of the beams was wetted with acetone.

Results and Discussion

The prisms and cubes were tested [25] under a static and dynamic load one by one. The test results for the cubes are in Table 2. The cubes were made without reinforcement (series 1), with disperse reinforcement with the fibre content of 1 % (series 2) and 2 % (series 3). The average cubic strength of concrete at the time of the tests was 31.4 MPa. The average cubic strength of concrete for the dynamic tests was 32.7 MPa. The strength of concrete during the dynamic tests was 4.2 % higher than that during the static tests. The average cubic strength of fibrecocnrete for beams with a combined reinforcement and those with a disperse one with 1.0 % of fibre reinforcement at the time of the tests was 32.6 MPa. The average strength of such cubes of fibreconcrete during the dynamic tests was 34.4 MPa.

The strength of cubes of fibreconcrete for the dynamic tests was 5.5% higher than that for the static ones.

Table 2. Results of the compression tests of cubes

№ of the series	Static		Dynamics	
	Sample labelling	Failure load, kN	Sample labelling	Failure load, kN
1	K-B-S-1	702	K-B-D-1	735
	K-B-S-2	709	K-B-D-2	739
	K-B-S-3	710	K-B-D-3	738
2	K-FB-S-1(1.0)	739	K-FB-D-1 (1.0)	776
	K-FB-S-2(1.0)	736	K-FB-D-2(1.0)	774
	K-FB-S-3(1.0)	730	K-FB-D-3 (1.0)	771
3	K-FB-S-1(2.0)	742	K-FB-D-1 (2.0)	780
	K-FB-S-2(2.0)	748	K-FB-D-2 (2.0)	785
	K-FB-S-3(2.0)	746	K-FB-D-3 (2.0)	778

In Table 2 there are the following denotations: К is a cube; B is concrete; FB is fibreconcrete; S is static;

D is dynamic; 1, 2, 3 is the number of a sample; (1.0), (2.0) is a percentage of a volumetric fibre reinforcement.

The average cubic strength of fibreconcrete for beams with combined reinforcement and those with a disperse one with 2.0 % of fibre reinforcement was 33.15 MPa. The average strength of such cubes of fibreconcrete during the dynamic tests was 34.7 MPa. The strength of cubes of fibreconcrete during the dynamic tests was 4.8 % higher than that during the static tests.

The compression strength of fibreconcrete was 4–6 % higher than that of standardized concrete depending on the percentage of fibre reinforcement.

The results of the tests of prisms are presented in Table 3. The prisms were made in series 1 with no reinforcement, in series 2 with a disperse reinforcement with 1 % of fibre and in series 3 with 2 % of fibre.

Table 3. Results of the compression tests of prisms

№ of the series	Static		Dynamics	
	Sample labelling	Failure load, kN	Sample labelling	Failure load, kN
1	P-B-S-1	258	P-B-D-1	285
	P-B-S-2	266	P-B-D-2	281
	P-B-S-3	262	P-B-D-3	280
2	P-FB-S-1(1.0)	280	P-FB-D-1(1.0)	299
	P-FB-S-2(1.0)	283	P-FB-D-2(1.0)	301
	P-FB-S-3(1.0)	276	P-FB-D-3(1.0)	308
3	P-FB-S-1(2.0)	290	P-FB-D-1(2.0)	315
	P-FB-S-2(2.0)	292	P-FB-D-2(2.0)	309
	P-FB-S-3(2.0)	286	P-FB-D-3(2.0)	313

In Table 2 there are the following denotations: P is a prisms; B is concrete; FB is fibreconcrete; S is static; D is dynamic; 1, 2, 3 is the number of a sample; (1.0), (2.0) is a percentage of a volumetric fibre reinforcement.

The average prism strength of concrete at the time of the tests was 26.2 MPa. The average strength of prisms of concrete during the dynamic tests was 28.2 MPa. The prism strength of concrete during the dynamic tests was 7.6 % higher than that during the static tests.

The average prism strength of fibreconcrete for beams with a combined reinforcement and those with a disperse one with 1.0 % of fibre reinforcement at the time of the tests was 27.9 MPa. The average strength of such prisms of fibreconcrete during the dynamic tests was 30.3 MPa. The strength of prisms of fibreconcrete during the dynamic tests was 8.6% higher than that during the static tests. The average prism strength of fibreconcrete for beams with a combined reinforcement and those with a disperse one with 2.0 % of fibre reinforcement at the time of the tests was 28.9 MPa. The average strength of such prisms of fibreconcrete during the dynamic tests was 31.2 MPa. The strenght of prisms of fibreconcrete during the dynamic tests was 8 % higher than that during the static tests.

The prism strength of fibreconcrete was 6.5–10 % higher than that of standardized concrete depending on the percentage of fibre reinforcement. The tests of prisms showed that the failure of prisms of fibreconcrete was viscous unlike fragile failure of concrete prisms. It should be noted that the distribution of fibre has a great influence on the nature of failure of prisms [26]. It is particularly the case for the percentage of volumetric reinforcement of 1 %.

Non-reinforced prisms were originally deformed elastically. Then microcracks occurred and transformed into localized microcracks that came together under a load and as a result, the prism lost its strength. During the dynamic tests concrete prisms were almost torn into pieces. In order to take a photograph of the samples, they had to be put together using a wire. In disperse reinforced prisms the failure was more viscous with its nature having to do with the amount of fibre in concrete. The comparison of the type of a failure of prisms during the dynamic tests is in Figure 7.

Figure 7. Typical view of failures of prisms following a dynamic impact: on the left are concrete prisms, on the right are fibreconcrete prisms

The impact on the tested beams was an impulse by applying a dynamic load to the operating cart. As a result of the impact, the resulting acceleration by using the supports of the tested structure was 18-22 g. The time of impact of each of the semiwaves of acceleration was 9-10 msec. The difference between the readings of the sensors of acceleration on different supports was 0.8-1.2%. The centre of the span of the tested beam started displacement following 2.5-3.5 msec after the supports started displacement. As a result, there was alternating loading and unloading occurring in the beams, which caused a change in their stress-strain state. These led to the accumulation of residual deformations and damages. The results of the tests of the experimental beams are presented in Table 4.

Table 4. Results of the tests of the beams using impacts such as “seismic” ones (the mean values in the series)

№ of the series	Sample labelling	Maximum displacement of the centre of the span, mm	Time of oscillations of the beam, msec	Maximum acceleration of the centre of the span, g	Notes
1	B-ZHB-D	9	210	48	
2	B-FZHB-D(1.0)	6	120	45	
3	B-FB-D(2.0)	12	60	12	Failure of the beam
4	B-FZHB-D(2.0)	5	105	40	
5	B-FB-D(1.0)	-	-	8	Failure of the beam

In Table 4 there are the following denotations: B is a beam; ZHB is ferroconcrete; FZHB is fibreferroconcrete; D is dynamic; (1.0), (2.0) is a percentage of the volumetric fibre reinforcement.

The ratio of the maximum acceleration on the support and in the centre of the span of the beam was 0.6–0.7. In Figure 8 there are graphs of the oscillations of the centre of the span of the beam under an alternating dynamic highly intensive impact.

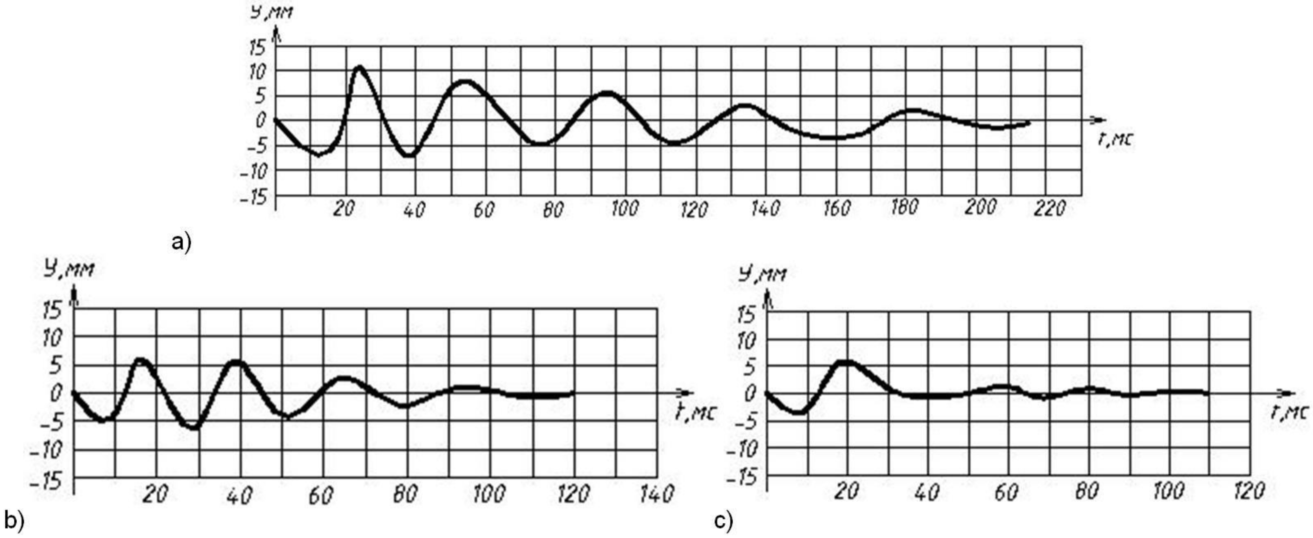

Figure 8. Graphs of the oscillations of the centre of the span of the beams under an alternating dynamic impact with the intensity 20 gbeams of series 1 (a), of series 2 (b), of series 4 (c)

The tests of the beams with a combined reinforcement (series 2 and series 4) showed that the amplitude of the oscillations in the centre of the span of such beams is almost two times less than in ferroconcrete ones (series 1). It can be accounted for with the fact that fibres additionally perceive some of compression forces and curb a bending increase. Damping of the oscillations of fibreferroconcrete beams was more rapid than in ferroconcrete ones and as the percentage of fibre reinforcement went up, it grew even more so. Rapid damping of the oscillations of the structure has a positive effect on the human behavior during earthquakes contributing to low levels of panic.

There were visible cracks all through the ferroconcrete beams and plastic deformations in the longitudinal rod reinforcement in the centre of the span. Under an identical impact in the beams of series 2 there were visible cracks with less opening but not all through them. In the beams of series 4 there were no visible cracks and in the longitudinal rod reinforcement in the centre of the beams there were no plastic deformations. This is due to a great influence of the dynamic strength of concrete of initial defects in the structure of concrete (e.g., microcracks). The use of a disperse reinforcement reduces this influence to a maximum.

The tests of fibreconcrete beams showed the following. In the beams of series 3 the total time of the oscillations was on average 60 msec, maximum bending of the beam was 12 mm with enough cracks in the beams but they retained their integrity as a self-sustaining element. However, it was not able to perceive the external impacts that followed. The beams of series 5 almost failed with a crack all through them. The fibreswere largely stretched from the matrix. Due to a constant stretching of the fibres, the failure was viscous.

Numerical Calculation of the Oscillations of the Beam

Numerical modelling of the oscillations of the beam under an impulse load was performed using the finite element method by means of a computational tool Structure CAD (SCAD). In the software in order to solve a dynamic task, an absolutely stablevatriant of the Newmark method was implemented in the form of the “predictor-corrector” algorithm [27].

The sizes, mass of the beam, type of an impulse and parameters of the damping of the oscillations were accepted according to the results of the experiment. A preliminary modal analysis of the dynamic model of the beam (Figure 9.) showed that the first frequency of the oscillations was f = 26.74 Hz (T_1 = 0.0374 sec). The time of the shock impulse obtained during the experiment was t_u = 0.018 sec. With a relative time of the impulse t_u/T_1 = 0.05 its time has no great influence on free oscillations of the beam [28] as shown in Figure 10.

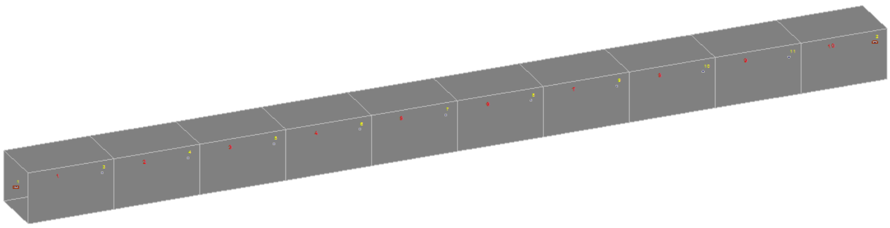

Figure 9. Finite-element model of the beam of a rectangular section

While comparing the calculated linear elastic operation of the material obtained in the assumption and experimental vibrorecord, it can be concluded that the nature of oscillations of the beam during the experiment was non-linear due to damages of the concrete and reinforcement. Cracks and plastic deformations in the reinforcement also contribute greatly to the saturation of the oscillation energy as shown in the experimental vibrorecord.

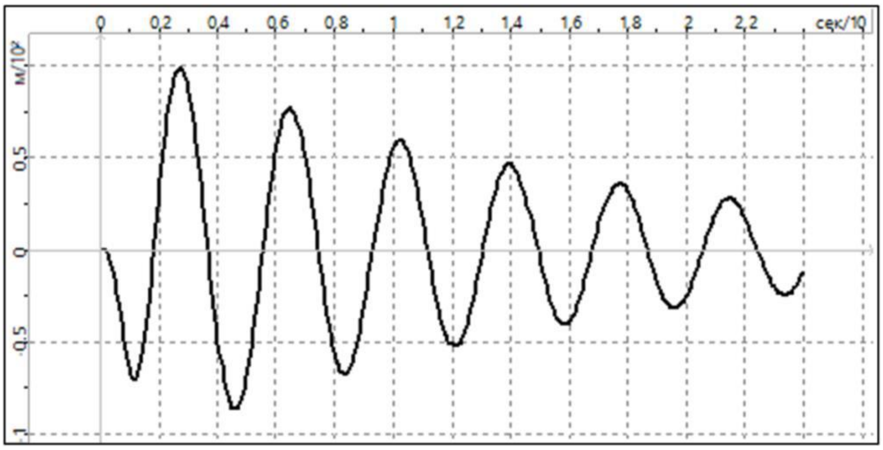

Figure 10. Calculationalvibrorecord of displacements in the middle of the span of the beam

The results of the studie sareinagreement withthoseobtained by otherre searchers [6, 8, 9, 10, 11, 12, 13, 14, 17, 18] and compliment them in terms of the use of disperse reinforcement with steel fibre with a small l/d ratio in structures under an alternating highly intensive dynamic load.

Conclusions

1. The use of a disperse reinforcement with the use of fibres with a small l/d ratio has a significant impact on the behaviour of structures with a combined reinforcement under an alternating dynamic highly intensive load thus resulting in an increase in the crack resistance as well as a decrease in the amplitude and time of oscillations of structures with a combined reinforcement.

2. Under alternating dynamic highly intensive loads it is reasonable to make use of a combined reinforcement since structures with only one fibre reinforcement are not capable of perceiving such loads.

3. During an alternating deformationfibre shows extra resistance to the opening of cracks not only due to cohesion with concrete but also thanks to the resistance in the transverse direction. If the resistance of fibres is in the axial direction, after being stretched out of the matrix they continue to resist due to the friction force along the surface of a resulting channel in the concrete.

4. Cubes and prisms with a disperse reinforcement has a higher compression strength compared to a non-reinforced concrete under a static and dynamic impact for the employed geometric parameters of fibres. An increase in the strength is due to “the ring effect” created by a disperse reinforcement that prevents transverse expansion.

References

1. *RTM–17–03–2005.* Rukovodyashchiye tekhnicheskiye materialy po proyektirovaniyu, izgotovleniyu i primeneniyu stalefibrobetonnykh konstruktsiy na fibre iz stalnoy provoloki [Russian regulatory document PTM–17–03–2005. Technical guides for the design, manufacture and application of steel fiber reinforced concrete structures on the fiber from a steel wire]. Moscow, 2005. 79 p. (rus)

Литература

1. РТМ–17–03–2005. Руководящие технические материалы по проектированию, изготовлению и применению сталефибробетонных конструкций на фибре из стальной проволоки. М., 2005. 79 с.
2. СП 52–104–2006. Сталефибробетонные конструкции. М., 2007. 68 с.

Nikolenko S.D., Sushko E.A., Sazonova S.A., Odnolko A.A., Manokhin V.Ya. Behaviour of concrete with a disperse reinforcement under dynamic loads. *Magazine of Civil Engineering*. 2017. No. 7. Pp. 3–14. doi: 10.18720/MCE.75.1.

2. *SP 52–104–2006*. Stalefibrobetonnyye konstruktsii [Russian Set of Rules SP 52-104-2006. Steel fiber concrete construction]. Moscow, 2007. 68 p. (rus)
3. Nizina T.A., Balykov A.S. Experimental-statistical models of properties of modified fiber-reinforced fine-grained concretes. *Magazine of Civil Engineering* 2016. No. 2. Pp. 13–25.
4. Mailyan L.R., Shilov A.V. *Izgibayemyye keramzitofibrozhelezobetonnyye elementy na grubom bazaltovom volokne* [Bending claydite-reinforced concrete elements on coarse basalt fiber]. Rostov-on-Don, 2001. 174 p. (rus)
5. Taheri Fard A.R., Soheili H., Ramzani Movafagh S., Farnood Ahmadi P. Combined effect of glass fiber and polypropylene fiber on mechanical properties of self-compacting concrete. *Magazine of Civil Engineering.* 2016. No. 2. Pp. 26–31.
6. Vikrant S. Vairagade, Kavita S. Kene. Introduction to Steel Fiber Reinforced Concrete on Engineering Performance of Concrete. *International Journal of Scientific & Technology Research.* 2012. Vol. 1. No. 4. Pp. 139–141.
7. Banthia N. FRC: Milestone in international Research and development. Proceedings of FIBCON. Nagpur, India. 2012. Pp. 48.
8. James J., Beaudoin. Handbook of fiber reinforced concrete. principles, properties, development and applications. Noyes Publications. New Jersey, United State of America. 1990. Pp. 57–63.
9. Dwarakanath H.V., Nagaraj T.S. Deformational behavior of reinforced fiber reinforced concrete beams in bending. Journal of Structural Engineering. 1992. No. 118(10). Pp. 2691–2698.
10. Thomas J., Ramaswamy A. Mechanical properties of steel fiber-reinforced concrete. *Journal of Material in Civil Engineering.* 2007. No. 19(5). Pp. 385–392.
11. Mr. Vikram Vijaysinh Balgude. Experimental study on crimped steel fiber reinforced concrete deep beam in shear. *IOSR Journal of Mechanical and Civil Engineering.* 2014. Vol. 11. No. 2. Ver. IV. Pp. 24–39.
12. Al-lami K.A. *Experimental investigation of fiber reinforced concrete beams.* A thesis submitted in partial fulfillment of the requirements for degree of Master of Science. Portland State University 2015. 106 p.
13. Chien Yet T., Hamid R., Kasmuri M. Dynamic stress-strain behaviour of steel fiber reinforced high-performance concrete with fly ash. *Advances in Civil Engineering.* 2012. Vol. 2012. Article No. 907431.
14. Xu Z., Hao H., Li H. Experimental study of dynamic compressive properties of fibre reinforced concrete material with different fibres. *Materials & Design.* 2012. Vol. 33. Pp. 42–55.
15. Rabinovich F.N. *Kompozity na osnove dispersnoarmirovannykh betonov. Voprosy teorii i proyektirovaniya, tekhnologiya, konstruktsii* [Composites based on dispersed reinforced concrete. Issues of theory and design, technology, design]. Moscow, 2004. 560 p. (rus)
16. Rusanov V.Ye. Opredeleniye klassa fibrobetona po ostatochnoy prochnosti na rastyazheniye pri izgibe [Determination of the class of fiber-reinforced concrete by residual tensile strength at bending] *Transportnoe stroitel'stvo.* 2014. No. 1. Pp. 16–18. (rus)
17. Morozov V.I., Pukharenko Yu.V. Effektivnost primeneniya fibrobetona v konstruktsiyakh pri dinamicheskikh vozdeystviyakh [Efficiency of fiber reinforced concrete application in structures subjected to dynamic effects]. Vestnik of MGSU. 2014. No. 3. Pp. 189–196. (rus)
18. Hameed R., Duprat F., Turatsinze A., Sellier A., Siddiqi Z.A. Behaviour of reinforced fibrous concrete beams under reversed cyclic loading. *Pak. J. Engg. & Appl. Sci.* 2011. Vol. 9. Pp. 1–12.
19. Kurnavina S.O. *Dinamicheskiy raschet zhelezobetonnykh*

3. Низина Т.А., Балыков А.С. Экспериментально-статистические модели свойств модифицированных дисперсно-армированных мелкозернистых бетонов // Инженерно-строительный журнал. 2016. № 2(62). С. 13–25.
4. Маилян Л.Р., Шилов А.В. Изгибаемые керамзитофиброжелезобетонные элементы на грубом базальтовом волокне. Ростов-на-Дону, 2001. 174 с.
5. Тахери Фард А.Р., Сохели Х., Рамзани Мовафах С., Фарнуд Ахмади П. Совместное действие стеклянной и полипропиленовой фибры на механические свойства самоуплоутняющихся бетонов // Инженерно-строительный журнал. 2016. № 2(62). С. 26–31
6. Vikrant S. Vairagade, Kavita S. Kene. Introduction to steel fiber reinforced concrete on engineering performance of concrete // International Journal of Scientific & Technology Research. 2012. Vol. 1. № 4. Pp. 139–141.
7. Banthia N. FRC: Milestone in international Research and development // Proceedings of FIBCON. Nagpur, India, 2012. Pp. 48.
8. James J., Beaudoin. Handbook of Fiber Reinforced Concrete. Principles, properties, Development and Applications // Noyes Publications. New Jersey, United State of America, 1990. Pp. 57–63.
9. Dwarakanath H.V., Nagaraj T.S. Deformational behavior of reinforced fiber reinforced concrete beams in bending // Journal of Structural Engineering. 1992. № 118(10). Pp. 2691–2698.
10. Thomas J., Ramaswamy A. Mechanical properties of steel fiber-reinforced concrete // Journal of Material in Civil Engineering. 2007 № 19(5). Pp. 385–392.
11. Mr. Vikram Vijaysinh Balgude. Experimental study on crimped steel fiber reinforced concrete deep beam in shear // IOSR Journal of Mechanical and Civil Engineering. 2014. Vol. 11. № 2. Ver. IV. Pp. 24–39.
12. Al-lami K.A. Experimental investigation of fiber reinforced concrete beams // A thesis submitted in partial fulfillment of the requirements for degree of Master of Science. Portland State University, 2015. 106 p.
13. Chien Yet T., Hamid R., Kasmuri M. Dynamic stress-strain behaviour of steel fiber reinforced high-performance concrete with fly ash // Advances in Civil Engineering. 2012. Vol. 2012. Article № 907431.
14. Xu Z., Hao H., Li H. Experimental study of dynamic compressive properties of fibre reinforced concrete material with different fibres // Materials & Design. 2012. Vol. 33. Pp. 42–55.
15. Рабинович Ф.Н. Композиты на основе дисперсноармированных бетонов. Вопросы теории и проектирования, технология, конструкции. М., 2004. 560 с.
16. Русанов В.Е. Определение класса фибробетона по остаточной прочности на растяжение при изгибе // Транспортное строительство. 2014. №1. С. 16–18.
17. Морозов В.И., Пухаренко Ю.В. Эффективность применения фибробетона в конструкциях при динамических воздействиях. // Вестник МГСУ. 2014. № 3. С. 189–196.
18. Hameed R., Duprat F., Turatsinze A., Sellier A., Siddiqi Z.A. Behaviour of reinforced fibrous concrete beams under reversed cyclic loading // Pak. J. Engg. & Appl. Sci. 2011. Vol. 9. Pp. 1–12.
19. Курнавина С.О. Динамический расчет железобетонных конструкций с учетом упругопластических деформаций арматуры и бетона по сечениям, совпадающим с полем направлений трещин: дис. … канд. техн. наук: 05.23.10. М:, 1999. 193 с.
20. Абаканов М.С. Прочность железобетонных конструкций при малоцикловых нагружениях типа сейсмических // Сейсмостойкое строительство. Безопасность сооружений. 2013. № 5. С. 30–34.
21. Кишиневская Е.В., Ватин Н.И., Кузнецов В.Д.

konstruktsiy s uchetom uprugoplasticheskikh deformatsiy armatury i betona po secheniyam, sovpadayushchim s polem napravleniy treshchin [Dynamic calculation of reinforced concrete structures taking into account elastoplastic deformations of reinforcement and concrete by sections that coincide with the field of crack directions] The thesis of a Cand. Tech. Sci.: 05.23.10. Moscow, 1999. 193 p. (rus)

20. Abakanov M.S. Prochnost zhelezobetonnykh konstruktsiy pri malotsiklovykh nagruzheniyakh tipa seysmicheskikh [Strength of reinforced concrete structures under low-cycle loads such as seismic]. *Sejsmostojkoe stroitel'stvo. Bezopasnost' sooruzhenij.* 2013. No. 5. Pp. 30–34. (rus)
21. Kishinevskaya Ye.V., Vatin N.I., Kuznetsov V.D. Perspektivy primeneniya nanobetona v monolitnykh bolsheproletnykh rebristykh perekrytiyakh s postnapryazheniyem [Prospects for the use of nanobeton in monolithic large-span ribbed ceilings with post-tension]. *Magazine of Civil Engineering.* 2009. No. 2. Pp. 54–58. (rus)
22. Kishinevskaya Ye.V., Vatin N.I., Kuznetsov V.D. Usileniye stroitelnykh konstruktsiy s ispolzovaniyem postnapryazhennogo zhelezobetona [Strengthening of building structures using post-stressed reinforced concrete]. *Magazine of Civil Engineering.* 2009. No. 2. Pp. 29–32. (rus)
23. *ACI 544 1R-96 (2002).* State-of-the-art-report on Fibre Reinforced Concrete. Reported by ACI Committee 544, 2002. 66 p.
24. *SP 14.13330.2011.* Stroitelstvo v seysmicheskikh rayonakh [Russian Set of Rules SP 14.13330.2011. Construction in seismic areas]. Moscow, 2011. 88 p. (rus)
25. *GOST 10180–2012.* Betony. Metody opredeleniya prochnosti po kontrolnym obraztsam [Russian State Standard GOST 10180-2012. Concretes. Methods for strength determination using reference specimens]. Moscow, 2013. 36 p. (rus)
26. Tkachenko A.N., Nikolenko S.D., Fedulov D.V. Theoretical estimation of fiber distribution in fiber reinforced concretes. *Russian Journal of Building Construction and Architecture.* 2011. No. 3(11). Pp. 36–41.
27. Hughes T.J.R., Belytschko T. A precis of development in computational methods for transient analysis. *Journal of Applied Mechanics.* 1983. Vol. 50. Pp. 1033–1041.
28. Klaf R., Penziyen Dzh. *Dinamika sooruzheniy* [Dynamics of structures]. Moscow, 1979. 320 p. (rus)

Перспективы применения нанобетона в монолитных большепролетных ребристых перекрытиях с постнапряжением // Инженерно-строительный журнал. 2009. № 2. С. 54–58.

22. Кишиневская Е.В., Ватин Н.И., Кузнецов В.Д. Усиление строительных конструкций с использованием постнапряженного железобетона // Инженерно-строительный журнал. 2009. №2. С. 29–32.
23. ACI 544 1R–96 (2002). State-of-the-art-report on Fibre Reinforced Concrete. Reported by ACI Committee 544, 2002. p.66.
24. СП 14.13330.2011. Строительство в сейсмических районах. М., 2011. 88 с.
25. ГОСТ 10180–2012. Бетоны. Методы определения прочности по контрольным образцам. М., 2013. 36 с.
26. Tkachenko A.N., Nikolenko S.D., Fedulov D.V. Theoretical estimation of fiber distribution in fiber reinforced concretes // Russian Journal of Building Construction and Architecture. 2011. № 3(11). Pp. 36–41.
27. Hughes T.J.R., Belytschko T. A precis of development in computational methods for transient analysis // Journal of Applied Mechanics. 1983. Vol. 50. Pp. 1033–1041.
28. Клаф Р., Пензиен Дж. Динамика сооружений. Москва, 1979. 320 с.

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Impact of forest fires on buildings and structures

Воздействие лесных пожаров на здания и сооружения

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Key words: mathematical model; buildings; constructions; forest fire; equation; finite volume; numerical method

Ключевые слова: математическая модель; здания, сооружения; лесной пожар; уравнение; конечный объем; численный метод

Abstract. The mathematical modeling of forest fires actions on buildings and structures has been carried out to study the effects of fire intensity and wind speed on possibility of ignition of buildings. The crown forest fire is introduced as a heat and mass source defined by the empirical values of average crown fire temperature and vertical gas velocity at the top crown surface dependent on fire intensity. The hydrodynamic and thermal interactions between plume, wind flow and building are analyzed. The modeling approach is based on the use of standard non-stationary three-dimensional conservation equations that are solved numerically under the input conditions specific for large crown forest fires.

Аннотация. Проведено математическое моделирование действий лесных пожаров на зданиях и сооружениях с целью изучения влияния интенсивности огня и скорости ветра на возможность возгорания зданий. Верховой лесной пожар вводится как источник тепла и массы, определяемый эмпирическими значениями средней температуры верхового пожара и вертикальной скорости газа на верхней поверхности кроны в зависимости от интенсивности огня. Анализируются гидродинамическое и тепловое взаимодействия между шлейфом, потоком ветра и зданием. Модельный подход основан на использовании стандартных нестационарных трехмерных уравнений сохранения, которые решаются численно при входных условиях, характерных для крупных верховых лесных пожаров.

Introduction

This paper addresses the development of a mathematical model for fires in the wildland-urban intermix**.** The forest fire is a very complicated phenomenon. At present, fire services can forecast the danger rating of, or the specific weather elements relating to, forest fire. There is need to understand and predict forest fire initiation, behaviour and impact of fire on the buildings and constructions. This paper's purposes are the improvement of knowledge on the fundamental physical mechanisms that control forest fire behavior. A great deal of work has been done on the theoretical problem of forest fires. Crown fires are initiated by convective and radiative heat transfer from surface fires. However, convection is the main heat transfer mechanism. Crown fires a more difficult to control than surface. The first accepted method for prediction of crown fires was given by Rothermal [1] and Van Wagner [3]. The semi-empirical models [1-2] allow to obtain a quite good data of the forest fire rate of spread as a function of fuel bulk and moisture, wind velocity and the terrain slope. But these models use data for particular cases and do not give results for general fire conditions. Also crown fires initiation and hazard have been studied and modeled in detail (eg: Alexander [3], Van Wagner [3], Xanthopoulos, [4], Van Wagner, [5], Cruz [6], Albini [7], Scott, J. H. and Reinhardt, E.D. [8].The discussion of the problem of modeling forest fires is provided by a group of co-workers at Tomsk University (Grishin [9], Grishin et al [10]). A mathematical model of forest fires was obtained by Grishin [9] based on an analysis of known and original experimental data [9, 11], and using concepts and methods from reactive media mechanics. The physical two-phase models used in [12] may be considered as a development and extension of the formulation proposed by Grishin [9]. However, the investigation of crown fires thermal impacts on buildings and constructions has been limited mainly to cases of using simple models [13–16]. But in Russia and other countries these kinds of WUI (wildfire urban interface) models are developed very intensive [17-29]. The purpose of this paper is to study the thermal impact of forest fires on buildings and constructions. For the solution of this problem,

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it is necessary to solve the following problems: to develop a mathematical model for the spread of forest fire, to obtain a numerical solution, and to investigate the thermal effect of forest fires on buildings in order to determine the safe distances from combustion sites to buildings and structures. The present mathematical model and results of calculation are used to study the forest fire fronts interaction with buildings of different sizes.

Physical and mathematical model

It is assumed that the forest during a forest fire can be modeled as 1) a multi-phase, multistoried, spatially heterogeneous medium; 2) in the fire zone the forest is a porous-dispersed, two-temperature, single-velocity, reactive medium; 3) the forest canopy is supposed to be non – deformed medium (trunks, large branches, small twigs and needles), which affects only the magnitude of the force of resistance in the equation of conservation of momentum in the gas phase, i.e., the medium is assumed to be quasi-solid (almost non-deformable during wind gusts); 4) let there be a so-called "ventilated" forest massif, in which the volume of fractions of condensed forest fuel phases, consisting of dry organic matter, water in liquid state, solid pyrolysis products, and ash, can be neglected compared to the volume fraction of gas phase (components of air and gaseous pyrolysis products); 5) the flow has a developed turbulent nature and molecular transfer is neglected; 6) gaseous phase density doesn't depend on the pressure because of the low velocities of the flow in comparison with the velocity of the sound. Let the point x_1, x_2 , x_3= 0 is situated at the centre of the surface forest fire source at the height of the roughness level, axis $0x_1$ directed parallel to the Earth's surface to the right in the direction of the unperturbed wind speed, axis $0x_2$directed perpendicular to $0x_1$ and axis $0x_3$ directed upward (Fig. 1). The building is situated on the right part of the picture.

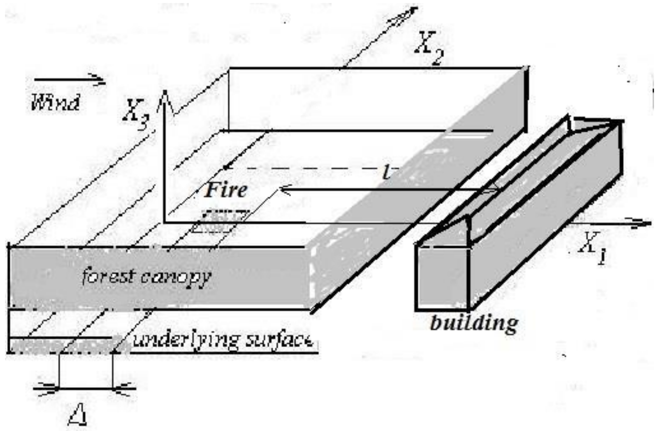

Figure 1. The scheme of computational domain

Problem formulated above reduces to the solution of systems of equations (1)–(7):

$$\frac{\partial \rho}{\partial t}+\frac{\partial}{\partial x_j}(\rho v_j)=Q,\ j=1,2,3; \quad (1)$$

$$\rho\frac{dv_i}{dt}=-\frac{\partial p}{\partial x_j}+\frac{\partial}{\partial x_j}(-\rho\overline{v_i'v_j'})-\rho s c_d v_i\,|\vec{v}\,|-\rho g_i-Qv_i,\ i=1,2,3; \quad (2)$$

$$\rho c_p\frac{dT}{dt}=\frac{\partial}{\partial x_j}(-\rho c_p\overline{v_j'T'})+q_5R_5-\alpha_v(T-T_s); \quad (3)$$

$$\rho\frac{dc_\alpha}{dt}=\frac{\partial}{\partial x_j}(-\rho\overline{v_j'c_\alpha'})+R_{5\alpha}-Qc_\alpha,\ \alpha=1,3; \quad (4)$$

$$\frac{\partial}{\partial x_j}\left(\frac{c}{3k}\frac{\partial U_R}{\partial x_j}\right)-k(cU_R-4\sigma T_S^4)=0; \quad (5)$$

$$\sum_{i=1}^{4}\rho_i c_{pi}\varphi_i\frac{\partial T_S}{\partial t}=q_3R_3-q_2R_2+k(cU_R-4\sigma T_S^4)+\alpha_V(T-T_S); \quad (6)$$

$$\rho_1 \frac{\partial \varphi_1}{\partial t} = -R_{1s}, \rho_2 \frac{\partial \varphi_2}{\partial t} = -R_{2s}, \rho_3 \frac{\partial \varphi_3}{\partial t} = \alpha_C R_{1s} - \frac{M_C}{M_1} R_{3w}, \rho_4 \frac{\partial \varphi_4}{\partial t} = 0; \quad (7)$$

$$\sum_{\alpha=1}^{3} c_\alpha = 1, P_e = \rho RT \sum_{\alpha=1}^{3} \frac{c_\alpha}{M_\alpha}, \vec{v} = (v_1, v_2, v_3),\ \vec{g} = (0, 0, g).$$

The system of equations (1)–(7) must be solved taking into account the initial and boundary conditions:

$$t = 0: v_1 = 0, v_2 = 0, v_3 = 0, T = T_e, c_\alpha = c_{\alpha e},\ T_s = T_{se}, \varphi_i = \varphi_{ie}; \quad (8)$$

$$x_1 = 0: v_1 = V, v_2 = 0, v_3 = 0, T = T_e, c_\alpha = c_{\alpha e}, -\frac{c}{3k}\frac{\partial U_R}{\partial x_1} + \frac{c}{2} U_R = 0; \quad (9)$$

$$x_1 = x_{1e}: \frac{\partial v_1}{\partial x_1} = 0, \frac{\partial v_2}{\partial x_1} = 0, \frac{\partial v_3}{\partial x_1} = 0, \frac{\partial T}{\partial x_1} = 0, \frac{\partial c_\alpha}{\partial x_1} = 0, \frac{c}{3k}\frac{\partial U_R}{\partial x_1} + \frac{c}{2} U_R = 0; \quad (10)$$

$$x_2 = -x_{2e}: \frac{\partial v_1}{\partial x_2} = 0, \frac{\partial v_2}{\partial x_2} = 0, \frac{\partial v_3}{\partial x_2} = 0, \frac{\partial T}{\partial x_2} = 0, \frac{\partial c_\alpha}{\partial x_2} = 0, -\frac{c}{3k}\frac{\partial U_R}{\partial x_2} + \frac{c}{2} U_R = 0; \quad (11)$$

$$x_2 = x_{2e}: \frac{\partial v_1}{\partial x_2} = 0, \frac{\partial v_2}{\partial x_2} = 0, \frac{\partial v_3}{\partial x_2} = 0, \frac{\partial T}{\partial x_2} = 0, \frac{\partial c_\alpha}{\partial x_2} = 0, \frac{c}{3k}\frac{\partial U_R}{\partial x_2} + \frac{c}{2} U_R = 0; \quad (12)$$

$$\begin{aligned} &x_3 = 0:\ v_1 = 0, v_2 = 0, \frac{\partial c_\alpha}{\partial x_3} = 0, -\frac{c}{3k}\frac{\partial U_R}{\partial x_3} + \frac{c}{2} U_R = 0, \\ &\rho v_3 = \rho_0 \omega_0, T = T_0, |x_1| \le x_0, |x_2| \le x_0, \\ &\rho v_3 = 0, T = T_e, |x_1| > x_0, |x_2| > x_0; \end{aligned} \quad (13)$$

$$x_3 = x_{3e}: \frac{\partial v_1}{\partial x_3} = 0, \frac{\partial v_2}{\partial x_3} = 0, \frac{\partial v_3}{\partial x_3} = 0, \frac{\partial T}{\partial x_3} = 0, \frac{\partial c_\alpha}{\partial x_3} = 0, \frac{c}{3k}\frac{\partial U_R}{\partial x_3} + \frac{c}{2} U_R = 0. \quad (14)$$

Here and above $\frac{\mathrm{d}}{\mathrm{d}t}$ is the symbol of the total (substantial) derivative; α_V is the coefficient of phase exchange; ρ - density of gas – dispersed phase, t is time; v_i– the velocity components; T, T_S, – temperatures of gas and solid phases, U_R – density of radiation energy, k – coefficient of radiation attenuation,

P – pressure; c_p – constant pressure specific heat of the gas phase, c_{pi}, ρ_i, φ_i – specific heat, density and volume of fraction of condensed phase (1 – dry organic substance, 2 – moisture, 3 – condensed pyrolysis products, 4 – mineral part of forest fuel), R_i – the mass rates of chemical reactions, q_i – thermal effects of chemical reactions; k_g, k_S – radiation absorption coefficients for gas and condensed phases; T_e – the ambient temperature; c_α – mass concentrations of α – component of gas – dispersed medium, index α = 1,2,3, where 1 corresponds to the density of oxygen, 2 – to carbon monoxide CO, 3 – to carbon dioxide and inert components of air; R – universal gas constant; M_α, M_C, and M molecular mass of α –components of the gas phase, carbon and air mixture; g is the gravity acceleration; $c_d$is an empirical coefficient of the resistance of the vegetation, s is the specific surface of the forest fuel in the given forest stratum. To define source terms which characterize inflow (outflow of mass) in a volume unit of the gas-dispersed phase, the following formulae were used for the rate of formulation of the gas-dispersed mixture $\dot{m}$, outflow of oxygen R_{51}, changing carbon monoxide R_{52}

$$Q = (1-\alpha_c)R_1 + R_2 + \frac{M_c}{M_1} R_3, R_{51} = -R_3 - \frac{M_1}{2M_2} R_5,$$
$$R_{52} = \nu_g (1-\alpha_c)R_1 - R_5, R_{53} = 0.$$

Here ν_g – mass fraction of gas combustible products of pyrolysis, α_4 and α_5 – empirical constants. Reaction rates of these various contributions (pyrolysis, evaporation, combustion of coke and volatile combustible products of pyrolysis) are approximated by Arrhenius laws whose parameters (pre-exponential constant k_i and activation energy E_i) are evaluated using data for mathematical models [9, 10].

$$R_1 = k_1\rho_1\varphi_1 \exp(-E_1/RT_S),\; R_2 = k_2\rho_2\varphi_2 T_S^{-0.5}\exp(-E_2/RT_S),$$
$$R_3 = k_3\rho\varphi_3 S_\sigma c_1 \exp(-E_3/RT_S),$$
$$R_5 = k_5 M_2\left(\frac{c_1 M}{M_1}\right)^{0.25}\left(\frac{c_2 M}{M_2}\right)T^{-2.25}\exp(-E_5/RT)\,.$$

The initial values for volume of fractions of condensed phases are determined using the expressions:

$$\varphi_{1e} = \frac{d(1-\nu_z)}{\rho_1}, \varphi_{2e} = \frac{dW}{\rho_2}, \varphi_{3e} = 0,$$

where *d* – bulk density for surface layer, ν_z – coefficient of ashes of forest fuel, *W* – forest fuel moisture content. It is supposed that the optical properties of a medium are independent of radiation wavelength (the assumption that the medium is “grey”), and the so-called diffusion approximation for radiation flux density were used for a mathematical description of radiation transport during forest fires. To close the system (1)–(7), the components of the tensor of turbulent stresses, and the turbulent heat and mass fluxes are determined using the local-equilibrium model of turbulence (Grishin, [9]). The system of equations
(1)–(9) contains terms associated with turbulent diffusion, thermal conduction, and convection, and needs to be closed. The components of the tensor of turbulent stresses $\rho\overline{v_i' v_j'}$, as well as the turbulent fluxes of heat and mass $\overline{\rho v_j' c_p T'}$, $\overline{\rho v_j' c_\alpha'}$ are written in terms of the gradients of the average flow properties using the formulas:

$$-\rho\overline{v_i v_j} = \mu_t\left(\frac{\partial v_i}{\partial x_j} + \frac{\partial v_i}{\partial x_i}\right) - \frac{2}{3}K\delta_{ij},$$

$$-\rho\overline{v_j c_p T'} = \lambda_t \frac{\partial T}{\partial x_j},\; -\rho\overline{v_j c_\alpha'} = \rho D_t \frac{\partial c_\alpha}{\partial x_j},$$

$$\lambda_t = \mu_t c_p/\mathrm{Pr}_t,\; \rho D_t = \mu_t/Sc_t,\; \mu_t = c_\mu \rho K^2/\varepsilon,$$

where μ_t, λ_t, D_t are the coefficients of turbulent viscosity, thermal conductivity, and diffusion, respectively; Pr_t, Sc_t are the turbulent Prandtl and Schmidt numbers, which were assumed to be equal to 1. The thermodynamic, thermophysical and structural characteristics correspond to the forest fuels in the canopy of a different (for example pine [1, 9, 10]) type of forest.

Result and Discussion

The boundary-value problem (1)–(14) is solved numerically. Buildings were set using a method of fictitious areas [19].In order to efficiently solve this problem in a reactive flow the method of splitting according to physical processes was used. The basic idea of this method is based on the information that the physical timescale of the processes is great than chemical. In the first stage, the hydrodynamic pattern of flow and distribution of scalar functions was calculated. Then the system of ordinary differential equations of chemical kinetics obtained as a result of splitting was then integrated. The time step for integrating each function has to be smaller than the characteristic time of physical process to ensure the convergence of the numerical method. The time step was selected automatically. A discrete analog was obtained by means of the control volume method using the SIMPLE like algorithm (Patankar [30]). Difference equations that arise in the course of sampling were resolved by the method of SIP [31]. The accuracy of the program was checked by the method of inserted analytical solutions. Analytical expressions for the unknown functions were substituted in (1)–(14) and the closure of the equations were calculated. This was then treated as the source in each equation. Next, with the aid of the algorithm described above, the values of the functions used were inferred with an accuracy of not less than 1%. The effect of the dimensions of the control volumes on the solution was studied by diminishing them.

Fields of temperature, velocity, component mass fractions, and volume fractions of phases were obtained numerically. The first stage is related to increasing maximum temperature in the place of ignition with the result that a crown fire source appears. At this process stage over the fire source a thermal wind is formed a zone of heated forest fire prolysis products which are mixed with air, float up and penetrate into the crowns of trees. As a result, forest fuels in the tree crowns are heated, moisture evaporates and gaseous and dispersed pyrolysis products are generated. Ignition of gaseous pyrolysis products of the crown occurs at the next stage, and that of gaseous pyrolysis products in the forest canopy occurs at the last stage. As a result of heating of forest fuel elements of crown, moisture evaporates, and pyrolysis occurs accompanied by the release of gaseous products, which then ignite and burn away in the forest canopy. At the moment of ignition the gas combustible products of pyrolysis burns away, and the concentration of oxygen is rapidly reduced. The temperatures of both phases reach a maximum value at the point of ignition. The ignition processes is of a gas-phase nature. At $V_e \neq 0$, the wind field in the forest canopy interacts with the gas-jet obstacle that forms from the forest fire source and from the ignited forest canopy and burn away in the forest canopy. The isotherms of gas phase components moved in the forest canopy by the action of wind. It is concluded that the forest fire begins to spread. The results of the calculation give an opportunity to consider forest fire spread for different wind velocity, canopy bulk densities and moisture forest fuel. It is considered the effect of forest fire front on the building which is situated near from the forest. The influences of wind velocity and distance between forest and building on ignition of building are studied numerically. The results of calculations can be used to evaluate the thermal effects on the building, located near from the forest fires. The wind and temperature fields interact with the obstacle – building (Figure 2 a) and b)).

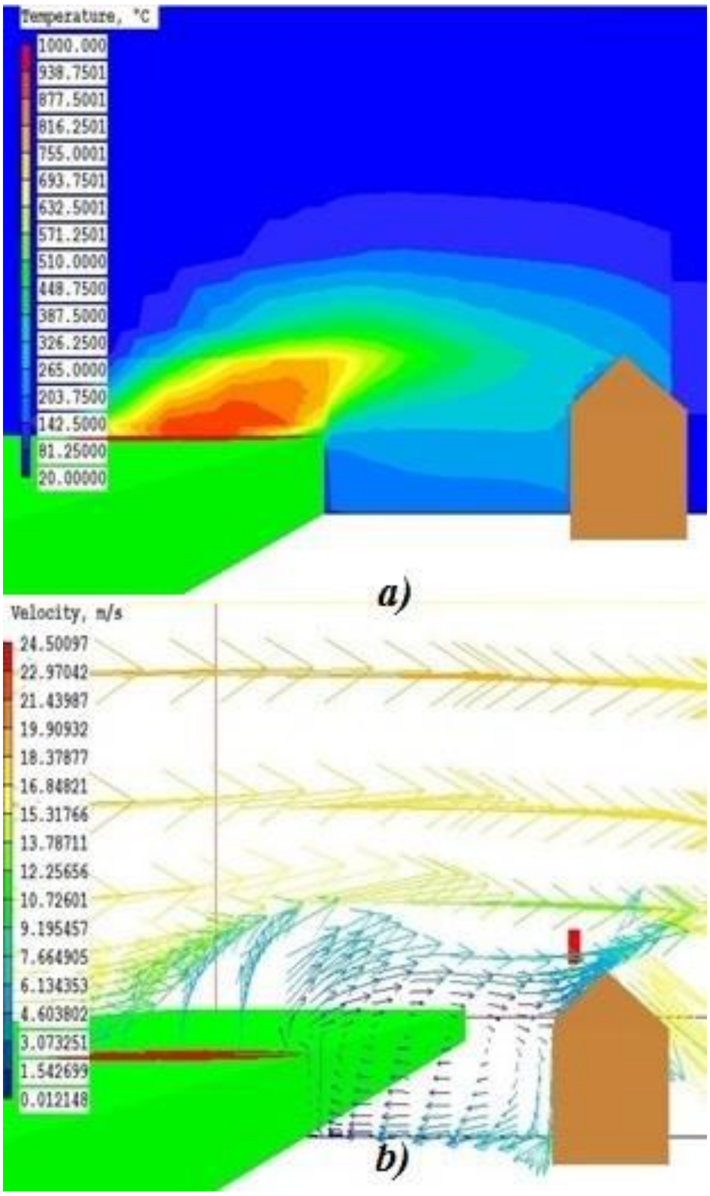

Figure 2. The distributions of temperature a) and velocity b) near from the building

Figure 2 shows the simulation results of the wall temperatures at different distances between crown forest fire and wooden building (20×50×20 meters) for different values of wind speeds from 3 to 15 m/s. An analysis of this dependence (Figure 3) shows the following:

Fire wood construction with a wind speed of 3 m/s will start at a distance of 15-16 meters from the forest fires;

Fire wood construction with a wind speed of 5 m/s will start at a distance of 26-27 meters from the forest fires ;

Fire wood construction with a wind speed of 10 m/s will start at a distance of 39-40 meters from the forest fires;

Fire wood construction with a wind speed of 15 m/s will start at a distance of 46-47 meters from the forest fires.

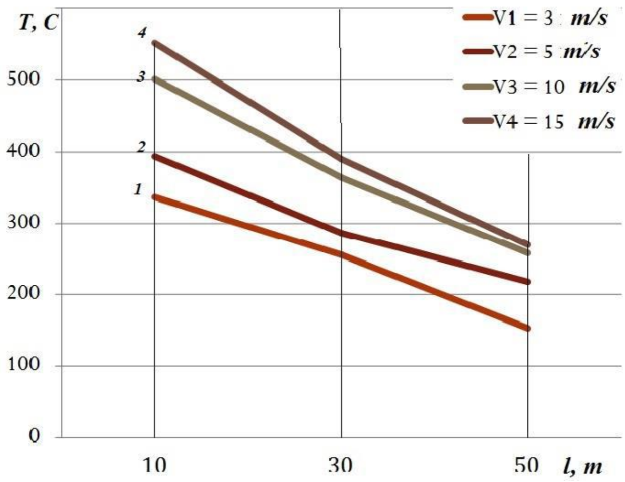

Figure 3. The dependence of the temperature on the walls of a wooden house for different wind speeds (3-15m/s) for the different distances *l* (10-50 m)

Then using the results of calculation it is plotted the temperature on the walls of a wooden building (12×15×12 meters) for wind speeds from 3 to 15 m/s. The results are shown in Figure 4.

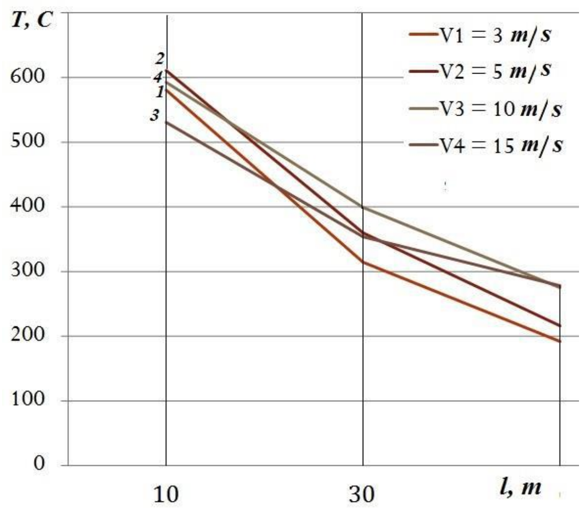

Figure 4. The dependence of the temperature on the walls of a wooden house for different wind speeds (3-15m/s) for the distances *l* (10-50 m)

An analysis of this dependence (Figure 4) shows the following:

1. Fire wood construction with a wind speed of 3 m/s will start at a distance of 32–33 meters from the forest fires;

Fire wood construction with a wind speed of 5 m/s will start at a distance of 38–39 meters from the forest fires;

Fire wood construction with a wind speed of 10 m / s will start at a distance of 43–44 meters from the forest fires;

Combustion of wooden buildings at a wind speed of 15 m/s will start at a distance of 42–43 meters from the forest fires.

Conclusion

The model proposed there gives a detailed picture of the change in the velocity, temperature and component concentration fields with time. It allows to investigate the dynamics of the impact of forest fires on buildings under the influence of various external conditions: a) meteorology conditions (air temperature, wind velocity etc.), b) type (various kinds of forest combustible materials) and their state (load, moisture etc.). The calculations let to get the maximum distance from the fire to the building in which the object possible ignition.

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References

1. Rothermal R.C. *Predicting behaviour and size of crown fires in the Northern Rocky Mountains.* Res.Pap. INT-438. Ogden. UT: US Department of Agriculture, Forest Service, Intermountain Forest and Range Experiment Station. 1972. 46p.
2. Van Wagner C.E. Conditions for the start and spread of crown fire. *Canadian Journal of Forest Research.* 1977. Vol. 7. Pp. 23–34.
3. Alexander M.E. Crown fire thresholds in exotic pine C.E. *10th conference on fire and forest meteorology.* Ottawa. Ontario. 1979. Pp. 207–212.
4. Xanthopoulos G. *Development of a wildland crown fire initiation model.* PhD thesis. University of Montana. Montana. 1990.
5. Van Wagner C.E. Prediction of crown fire behavior in two stands of jack pine. *Canadian Journal of Forest Research*. 1999. Vol. 23. Pp. 445–449.
6. Cruz M.G. et al. Predicting crown fire behavior to support forest fire management decision-making. *IV International conference on forest fire research.* Luso-Coimbra. Portugal. 2002. Vol. 11 [CD-ROM].
7. Albini F.A. et al. Modeling ignition and burning rate of large woody natural fuels. *Int. Journal of Wildland Fire.* 1995. Vol. 5. Pp. 81–91.
8. Scott J.H. et al. *Assessing crown fire potential by linking models of surface and crown fire behavior. USDA Forest Service, Rocky Mountain Forest and Range Experiment Station.* Fort Collins: RMRS-RP-29, (USA). 2001.
9. Grishin A.M. *Mathematical Modeling Forest Fire and New Methods Fighting Them.* Tomsk. Publishing House of Tomsk University. 1997.
10. Grishin A.M., Perminov V.A. Mathematical modeling of the ignition of tree crowns. *Combustion, Explosion, and Shock Waves.* 1998. Vol. 34. Pp. 378–386.
11. Konev E.V. *Fizicheskiye osnovy goreniya rastitel'nykh materialov* [The physical foundation of vegetative materials combustion]. Nauka. Novosibirsk (Russia). 1977. (rus).
12. Morvan D., Dupuy J.L. Modeling the propagation of wildfire through a Mediterranean shrub using a multiphase formulation. *Combustion and Flame*. 2004. Vol. 138. Pp. 199–210.
13. Cohen J.D. Preventing disaster: home ignitability in the wildland-urban interface. *Journal Forestry.* 2000. No. 98. Pp. 5–21.
14. Cohen J.D. Relating flame radiation to home ignition using modeling and experimental crown fires. *Canadian Journal of Forest Research.* 2004. Vol. 34. Pp. 1616–1626.
15. Porterie B, Consalvi J-L, Loraud J-C, et al. Dynamics of wildland fires and their impact on structures. *Combust and Flame.* 2007. No. 149. Pp. 314–328.
16. Mell W., McNamara D., Maranghides A., McDermott R., Forney G., Hoffman C., Ginder M. Computer modelling of wildland-urban interface fires. *Fire & Materials.*

Литература

1. Rothermal R.C. Predicting behaviour and size of crown fires in the Northern Rocky Mountains. Res.Pap. INT-438. Ogden. UT: US Department of Agriculture, Forest Service, Intermountain Forest and Range Experiment Station. 1972. 46 p.
2. Van Wagner C.E. Conditions for the start and spread of crown fire // Canadian Journal of Forest Research. 1977. Vol. 7. Pp. 23–34.
3. Alexander M.E. Crown fire thresholds in exotic pine C.E. // 10th conference on fire and forest meteorology. Ottawa. Ontario. 1979. Pp. 207–212.
4. Xanthopoulos G. Development of a wildland crown fire initiation model. PhD thesis. University of Montana. Montana. 1990.
5. Van Wagner C.E. Prediction of crown fire behavior in two stands of jack pine // Canadian Journal of Forest Research. 1999. Vol. 23. Pp. 445–449.
6. Cruz M.G. et al. Predicting crown fire behavior to support forest fire management decision-making // IV International conference on forest fire research. Luso-Coimbra. Portugal. 2002. Vol. 11 [CD-ROM].
7. Albini F.A. et al. Modeling ignition and burning rate of large woody natural fuels // Int. Journal of Wildland fire. 1995. Vol. 5. Pp. 81–91.
8. Scott J.H. et al. Assessing crown fire potential by linking models of surface and crown fire behavior. USDA Forest Service, Rocky Mountain Forest and Range Experiment Station. Fort Collins: RMRS-RP-29, (USA). 2001.
9. Grishin A.M. Mathematical Modeling Forest Fire and New Methods Fighting Them. Tomsk. Publishing House of Tomsk University. 1997.
10. Grishin A.M., Perminov V.A. Mathematical modeling of the ignition of tree crowns // Combustion, Explosion, and Shock Waves. 1998. Vol. 34. Pp. 378–386.
11. Конев Э.В. Физические основы горения растительных материалов. Новосибирск: Наука, 1977.
12. Morvan D., Dupuy J.L. Modeling the propagation of wildfire through a Mediterranean shrub using a multiphase formulation // Combustion and Flame. 2004. Vol. 138. Pp. 199–210.
13. Cohen J.D. Preventing disaster: home ignitability in the wildland-urban interface // Journal Forestry. 2000. № 98. Pp. 5–21.
14. Cohen J.D. Relating flame radiation to home ignition using modeling and experimental crown fires // Canadian Journal of Forest Research. 2004. Vol. 34. Pp. 1616–1626.
15. Porterie B, Consalvi J-L, Loraud J-C, et al. Dynamics of wildland fires and their impact on structures // Combust and Flame. 2007. № 149. Pp. 314–328.
16. Mell W., McNamara D., Maranghides A., McDermott R., Forney G., Hoffman C., Ginder M. Computer modelling of wildland-urban interface fires // Fire & Materials. International conference. San Francisco, CA. 2011.

International conference. San Francisco, CA. 2011.
17. Mell W.E., Manzello S.L., Maranghides A, Butry D., Rehm R.G. The wildland-urban interface problem – current approaches and research needs. *International J. Wildland Fire.* 2010. No. 19. Pp. 238–251.
18. David R. Weise and Mike Wotton, Wildland–urban interface fire behavior and fire modelling in live fuels. *International Journal of Wildland Fire.* 2010. No.19. Pp.149–152.
19. Cohen J.D. Relating flame radiation to home ignition using modeling and experimental crown fires. *Canadian Journal of Forest Research.* 2004. No. 34. Pp. 1616–1626.
20. Golner M.J. A review of pathways for building fire spread in the wildland-urban interface part I: exposure conditions // Fire Technology. 2016.
21. Alexander Maranghides and William Mell, A case study of a community affected by the witch and Guejito wildland fires. *Fire Technology.* 2011. No. 47. Pp. 379–420.
22. Syphard AD, Brennan TJ, Keeley JE. The role of defensible space for residential structure protection during wildfires. *Int J Wildland Fire.* 2014. Vol. 23(8). Pp.1165–1175.
23. Rossi, JL, Simeoni, A, Moretti, B & Leroy-Cancellieri V., An analytical model based on radiative heating for the determination of safety distances for wildland fires. *Fire Safety Journal.* 2015. Vol. 46. No. 8. Pp. 520–527.
24. Collins K.M., Price O.F., Penman T.D. Spatial patterns of wildfire ignitions in south-eastern Australia. *Int J Wildland Fire.* 2015. Vol. 24(8). Pp. 1098–2108.
25. Michael John Golner, A Review of Pathways for Building Fire Spread in the Wildland-Urban Interface Part I: Exposure Conditions // Fire Technology, 4, 2016.
26. Raquel S.P Hakes, Sara E. Caton, Daniel J. Gorham and Michael J. Gollner1 A review of pathways for building fire spread in the wildland urban interface Part II: Response of Components and Systems and Mitigation Strategies in the United States // Fire Technology,5, 2016.
27. Grishin A.M. Pugacheva P.V. Analiticheskoye resheniye zadachi o zazhiganii steny derevyannogo doma v rezul'tate deystviya fronta lesnogo pozhara [Analytical solution of the problem of ignition of the wooden house wall as a result of the action of the forest fire front]. *Tomsk State University Journal of Mathematics and Mechanics*. 2010. No. 3(11). Pp. 88–94. (rus).
28. Kuznetsov V.T, Filkov A.I. Vosplameneniye razlichnykh vidov drevesiny potokom luchistoy energii [Inflammation of various kinds of wood by a stream of radiant energy]. *Fizika goreniya i vzryva.* 2011. Vol. 47, No. 1. Pp. 74-79. (rus).
29. Grishin A.M., Yakimov A.S. Matematicheskoye modelirovaniye protsessa zazhiganiya drevesiny [Mathematical modeling of the process of ignition of wood]. *Thermophysics and Aeromechanics.* 2013. Vol. 20. No. 4. Pp. 473–486. (rus).
30. Loboda E.L., Kasymov D.P., Yakimov A.S. Modelirovaniye teplofizicheskikh protsessov pri zazhiganii derevyannoy doshchechki [Modeling of thermophysical processes during the ignition of a wooden plaque]. *Journal of Engineering Physics and Thermophysics.* 2015. Vol. 88. No. 1. Pp. 111–121. (rus).
31. Patankar S.V. *Numerical Heat Transfer and Fluid Flow. Hemisphere Publishing Corporation*. New York. 1981.
32. Stone H.L. Iterative solution of implicit approximations of multidimensional partial differential equations. *SIAM Journal of Numerical Analysis.* 1968. No. 5. Pp. 530–558.

17. Mell W.E., Manzello S.L., Maranghides A, Butry D., Rehm R.G. The wildland-urban interface problem – current approaches and research needs // International J. Wildland Fire. 2010. № 19. Pp. 238–251.
18. David R. Weise and Mike Wotton, Wildland–urban interface fire behavior and fire modelling in live fuels // International Journal of Wildland Fire. 2010. №19. Pp.149–152.
19. Cohen J.D. Relating flame radiation to home ignition using modeling and experimental crown fires // Canadian Journal of Forest Research. 2004. № 34. Pp. 1616–1626.
20. Golner M.J. A review of pathways for building fire spread in the wildland-urban interface part I: exposure conditions // Fire Technology. 2016.
21. Alexander Maranghides and William Mell, A case study of a community affected by the witch and Guejito wildland fires // Fire Technology. 2011. № 47. Pp. 379–420.
22. Syphard AD, Brennan TJ, Keeley JE. The role of defensible space for residential structure protection during wildfires // Int J Wildland Fire. 2014. Vol. 23(8). Pp.1165–1175.
23. Rossi, JL, Simeoni, A, Moretti, B & Leroy-Cancellieri V., An analytical model based on radiative heating for the determination of safety distances for wildland fires // Fire Safety Journal. 2015. Vol. 46. № 8. Pp. 520–527.
24. Collins K.M., Price O.F., Penman T.D. Spatial patterns of wildfire ignitions in south-eastern Australia // Int J Wildland Fire. 2015. Vol. 24(8). Pp. 1098–2108.
25. *Gollner M.J. A review of pathways for building fire spread in the wildland-urban interface. Part I: exposure conditions // Fire Technology. 2016. Vol. 4.*
26. Raquel S.P Hakes, Sara E. Caton, Daniel J. Gorham and Michael J. Gollner1 A review of pathways for building fire spread in the wildland urban interface Part II: Response of components and systems and mitigation strategies in the United States // Fire Technology,5, 2016.
27. Гришин А.М. Пугачева П.В. Аналитическое решение задачи о зажигании стены деревянного дома в результате действия фронта лесного пожара // Вестник Томского государственного университет. Математика и механика. 2010. № 3(11). С. 88–94.
28. Кузнецов В.Т., Фильков А.И. Воспламенение различных видов древесины потоком лучистой энергии // Физика горения и взрыва. 2011. Т. 47. № 1. С. 74−79.
29. Гришин А.М., Якимов А.С. Математическое моделирование процесса зажигания древесины // Теплофизика и аэромеханика. 2013. Т. 20. № 4. С. 473–486.
30. Лобода Е.Л., Касымов Д.П., Якимов А.С. Моделирование теплофизических процессов при зажигании деревянной дощечки // Инженерно-физический журнал. 2015. Т.88. № 1. С. 111–121.
31. Patankar S.V. Numerical Heat Transfer and Fluid Flow. Hemisphere Publishing Corporation. New York, 1981.
32. Stone H.L. Iterative solution of implicit approximations of multidimensional partial differential equations // SIAM Journal of Numerical Analysis. 1968. № 5. Pp. 530–558.

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Vertical transport: resource by the criterion of safety

Вертикальный транспорт: корректировка ресурса по критерию безопасности

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Ключевые слова: вертикальный транспорт; лифты; эксплуатация; отказ; риск; безопасность

Abstract. The issue of engineering systems safety in particular of vertical transportation is revealed. Inconsistency in the implementation of theoretical developments in practice is defined because of the probabilistic calculation of the parameters modeled by monotone-logical functions, when real systems are non-monotone functions. The duality of the results of the processes theoretical description is revealed. A typical algorithm of safety analysis is based on the deductive abilities of a researcher when drawing up a scenario of possible hazardous situations, their development and possible consequences. Critical analysis of the regulated risk assessment procedures FMEA / FMECA was carried out when compiling the criticality level matrix of the event or process. Conceptually, the risk analysis is represented by a sequence of logical steps that provide a systematic approach to the identification of hazards associated with the operation of vertical transportation. It is suggested to supplement the methodology for data records of the loss time during the nonproduction downtime with the safety parameter assessment. The condition of the vertical transportation systems and the parameter deviation vector in the dual risk-safety system are established by introducing a variable value of the parameter in the probabilistic polynomial of the simulation event model. The obtained result is universally applicable, which allows us to approach to the value of the simulated criticality of the parameter through the variability of the calculations. The developed method imposes constrains on the compilation of a logical chain of assumptions in the program of the experimental research. It also allows creating adequate conditions for the operating of the physical model of the system. The modified methodology suggests table compiling of the parameter variation limits ranging the hazard rate and calculating the corresponding values of hazard factors. It is suggested to apply the developed methodology as the supplement to the existing general methodology for risk assessment at all stages of the service time of vertical transportation. The example of the implementation of the modified procedure. The developed service life parameter adjusting method reduces operation costs, ensures safety and stability of the public mobile movement abilities when using vertical transportation.

Аннотация. Раскрыта проблема решения задач обеспечения безопасности технических систем, в частности вертикального транспорта. Установлена несогласованность реализации теоретических разработок в практике из-за вероятностного исчисления параметров, моделируемых монотонно-логическими функциями, когда реальные системы представляют собой немонотонные функции. Выявлена дуальность результатов теоретического описания процессов. Типовой алгоритм анализа безопасности, базируется на дедуктивных способностях исследователя при составлении сценария возможных аварийных ситуаций, их развития и возможных последствий. Выполнен критический анализ регламентированных методик оценки риска FMEA / FMECA при составлении матрицы критичности события или процесса. Концептуально анализ риска представлен последовательностью логических шагов, обеспечивающих системный подход к установлению опасностей, связанных с эксплуатацией вертикального транспорта. Предлагается дополнить методику учета потерь времени при непроизводственных простоях, оценкой показателя безопасности. Состояние систем вертикального транспорта и вектор отклонения показателя в дуальной системе риск-безопасности устанавливается путем введения вариативного значения параметра в вероятностный полином на имитационной модели события. Получаемый результат универсален. позволяет приблизиться к

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значению условной критичности показателя через вариативность исчисления. Разработанный метод накладывает ограничения при составлении логической цепочки допущений в программе экспериментальных исследований, позволяет создать адекватные условия функционирования физической модели системы. Модифицированная методика предполагает построение таблиц пределов изменения параметров с установлением тяжести проявления опасности и вычислением значений соответствующих им величин поражающих факторов. Предлагается применить разработанную методику как дополнение к уже имеющейся общей методологии при оценке риска на всех стадиях жизни вертикального транспорта. Составлен типовой алгоритм обеспечения безопасности компонентов с сопоставлением срока службы элементов лифта. Дан пример реализации модифицированной методики при установлении остаточного ресурса элементной базы лифта. Разработанный метод корректировки ресурсного показателя снизит затраты на эксплуатацию обеспечит безопасность и стабильность мобильного перемещения граждан при использовании вертикального транспорта.

Introduction

The technical regulations of the Customs Union state the service life of the various technical systems life cycle as an important factor in determining the probability of the hazardous event under analysis (Federal Law No. 184-FL of 27.12.2002 "On Technical Regulation"; Technical Regulation of Customs Union (TR CU) 011/2011 Safety of Elevators). Statutory regulation meets the requirements of the following documents: Russian State Standard GOST R 53387-2009 (ISO / CU 14798: 2006) Elevators, escalators and passenger conveyors. Methodology for analysis and risk reduction; Russian State Standard GOST R 54999-2012 (EN 13015: 2001) Elevators. General requirements for the instructions for technical maintenance of elevators; Russian State Standard GOST R 55964-2014 Elevators. General safety requirements for operational service.

TR CU 011/2011 "Safety of elevators" provide for the formulation of the scenario, including hazards, hazardous situation and the causes, as well as possible consequences, i.e. the identification of damage probability. In many cases risk characteristics cannot be precisely defined, only their level can be determined. That primarily applies to determining the probability of possible damage. TR CU 011/2011 "Safety of elevators" state four (1-4) levels of severity of possible damage, whereas while performing the risk analysis six (A-F) levels of damage probability are determined.

The design and maintainability risk assessment of elevators allows us to evaluate the safety level of the equipment, its components and related control procedures. The regulations of ISO / CU 14798: 2006 state elevators, escalators and passenger conveyors risk analyses subject as the following: complete elevator, escalator, passenger conveyor; components or systems of equipment; people dealing with the equipment; processes associated with the equipment and its components [1, 2].

The regulation establishes the stages of both analysis and risk reduction procedure. One of the steps is determination of the risk analysis subject and analysis-related factors (TR CU 011/2011 Safety of elevators, Russian State Standard GOST R 53387-2009 (ISO / CU 14798: 2006) Elevators, escalators and passenger conveyors. The methodology for analysis and risk reduction, Russian State Standard GOST R 55964-2014 Elevators. General safety requirements for operational service).

The scientists studying this problem put the notion of "acceptable risk" to use as a compromise solution. The main part of the conclusions and recommended methodologies is based on the statistical data compilation, on the acceptance of a certain set of assumptions, on the apriority of the proposed scenarios for the situation development and on the relativity of the results. Additional research, refinement of the previously obtained results lead to inconsistency, i.e. the scientist makes assumptions based on the statistical data of the previous studies which already include certain percentage of assumptions. These result in paradox – that is, an attempt to clarify the result of the previous studies leads to an increase in the logic-probabilistic influence as well as to an increase in the subjectivity of the suggested logical function. The article [3] presents a methodology for quantifying the risk of failure of lifting equipment, based on logic and probabilistic risk analysis techniques and the method of expert estimates. The article [4] gives an account and analysis of the accuracy of assembly variations in the parameters of the stress-strain state – according to values of assembly (initial) efforts. The author suggests constructive solutions for joining rods and installation method for coatings, which are aimed at increasing their load-bearing capacity, longevity and assemblability. The article [5] defines the ways of lifting unbalanced loads. Discusses the equipment for lifting is not balanced cargo. The article [6] gives an overview and analysis of the models of the damage accumulation in monolithic materials when exposed to prolonged and repeatedly applied load. The application of the principle of equivalence stress in the continuous and damage body allowed introducing into the strength criterion of Pisarenko-Lebedev and the three-parameter plasticity condition of Coulomb-Mohr the measures of the theory to accumulate damage in capacity of which the damage of Y.N. Rabotnova and the continuity of L.M. Kachanov are used. It was found that when exposed to repeated load, the process of the reduction of the continuity and the increase in the damage is hereditary. Therefore, to predict changes in these measures under the action of cyclic loads the integral equations of the theory of heredity are applied. In [7, 8], synergetic principles and mathematical apparatus of catastrophe theory were used to model the processes of destruction of polymeric materials. In polymeric materials, the process of accumulation of damages at various scale levels is proposed to be taken into account through the synergetic effect, the calculation of which in this work was carried out using the mathematical apparatus of catastrophe theory. It was found that a structure of any constructional material as a mechanical system possessed spatial and time properties and to study them a transition from the material structure to the cybernetic one was done. A formation process description of a new structure of the cybernetic system was suggested to do, using the information theory instrument. The authors [9] present a research of the opportunity to construct a sustainability model of life support systems under different emergency situations in respect of modern current trends in the development of information-analytical systems and principles of systems engineering approach.

It should be noted that the calculation methods imply the logic-probabilistic calculation of parameters, where the function arguments are both dual and Boolean variables. Mathematical calculi simulate monotone logic functions, although in reality the systems are non-monotone functions.

The implementation of the concept of complete or "absolute" human safety in the production sphere is an insurmountable task due to a number of reasons:

– engineering tools for performing work processes are technologically imperfect;

– the research methods that do not exclude randomness of hazardous situations during the operation of technical equipment and machinery are theoretically probabilistic.

The purpose of the research is to ensure the stability of the public mobile movement abilities while using vertical transportation by adjusting service life parameter of the elevator elements in accordance with the criteria of failure risk and safety throughout the entire period of their operation.

Research problems:

1. To carry out assessment of efficiency of application of techniques of assessment of risk of FMEA/FMECA making matrixes of criticality of an event in case of the decision of tasks of safety of vertical transport.

2. To consider the object and parametric characteristic of elevators, escalators and passenger pipelines taking into account factors of influence on origin of risk of a failure.

3. To set a level of variability of a resource index of vertical transport by development of scenarios of possible alert conditions

4. To develop the modified technique of assessment of safety of the elevator allowing to adjust conditions of technical maintenance and the requirement to the load modes of an element basis.

5. To make a safety algorithm with comparison of service life of elements of the elevator.

6. To give a technique example of implementation in case of establishment of a residual resource of an element basis of the elevator and change of an index of probability risk failure.

Basic aspects of the logic of compiling the mathematical model of the non-monotone function in the engineering system safety assessment.

From the point of view of reliability and safety, the processes occurring in the engineering systems when operating machinery and equipment can definitely be classified as irreversible.

The scheme of service support offered by manufacturers is aimed at maintaining the normative level of operability throughout the service life period.

From the point of view of the process theoretical description, the function arguments of the system are dual Boolean variables, i.e. carrying out the research suggests obtaining double results.

For elevator equipment, which is a technically complex system, danger is manifested in the form of failure of various structural elements with different levels of influence on safety. From now forth, it is accepted to characterize the possibility of a dangerous situation as failure, and the research is aimed at the establishment of the criteria throughout the entire operation period of elevators.

During the machine or equipment operation a failure is probable (P), whereas the safety of the process is ensured by a normative safety level (E) for a single situation under the specific conditions of both external and internal environment factor influence typical only to the above single situation. ((E = 1- P) it is assumed to present these values ranging from 0 to 1). One of the boundary values has the form P (E = 1) represented by a probabilistic polynomial of n variables P (x_1), ... P (x_n) [2, 10, 11].

Relying on the logic of the speculations on duality, the domain of the system existence is in two boundary states:

– "zero probability of a failure" - "absolutely safe system", P = 0: 1 ↔ E = 1: 0, the Boolean algebra allows presenting it as a logical expression P ↑ ↓ E. This idealized state of the system cannot be implemented.

– the scheme "danger of the situation - complete lack of safety" refers to the category of hazardous situations.

In the given example the duality is also inherent to the criterion assessment of the risk of failure in the design of the elevator, its different elements.

The logic of reasoning implies the presence of intermediate parameters, which many calculation methodologies rely on.

The basic methodologies of reliability and risk theories allow us to use the computational method to perform tasks connected with risk, reliability and the systems safety assessment. The standard safety assessment algorithm is commonly known, it is based on the deductive abilities of the researcher while drawing up the scenario for possible hazardous situations, their development and possible consequences. Simulating the event on the computational model by introducing a variable value of the parameter into the probabilistic polynomial, the state of the system and the deviation vector of the parameter are established in the dual risk-safety system. It is assumed that both physical and simulation modeling has a certain spread of data due to misperception of the phenomena by the researcher.

Methods

Analysis of the main statements of the regulation

According to FMEA or FMECA methodologies, functional safety is assessed by the criticality rating of the event or processes. The criticality rating is established according to the matrix compiled for each case on the basis of expert analysis. It should be noted that assessing the total risk, FMECA suggests the ranking system of the event contribution with the description of the factor interaction focusing on the acceptability parameter. For the systems with high risk or high complexity of the design it is recommended to use probabilistic risk analysis [7, 11–13].

The classical models of the quantitative evaluation are based on the mathematical models of both cost calculations and identification of the nondimensional value of the composite parameters. In particular, the reliability composite parameters are identified as failure free performance, durability and maintainability quantitatively represented by generalized formulas including loss factor as an indicator of a separate property:

$$k_f = \frac{1}{\left(1+\frac{t_{atf}}{t_{atbt}}\right)}, \tag{1}$$

where k_f – is the loss factor due to elimination of technical failures,

t_{atf} – is average time to eliminate one failure (design or production defect) during working hours, hour;

t_{atbf} – is average time between failures (design or production defect), hour.

The average time for elimination of one failure during working hours is directly dependent on the maintainability of the machine; organizational and production characteristics and the means of performing service works as well as indirect parameters of climatic conditions.

Technical failure due to the design or production defect depends primarily on the quality of the design and calculation works performed at the design stage and their implementation in production. It should be noted that a defect is inherently a failure having a random character and it is probabilistically difficult to define.

It is suggested to supplement the methodology for recording the time loss data during the non-productive downtime with the safety parameter record which is quantitatively expressed by the loss factor because of the elimination of the consequences of the hazardous situation:

$$k_f^{es} = \frac{1}{\left(1 + \frac{t_{atec}}{t_{adti}}\right)}, \tag{2}$$

where k_f^{es} is the loss factor due to elimination of the consequences of the abnormal or emergency situations

t_{atec} – is average time to eliminate the consequences of the hazardous situation, hour,

t_{adti} – is average duration of technical preventative measures taken against the hazardous situation for one technological machine, hour.

Further theoretical development is aimed at the refinement of the main aspects of the methodology for assessing the risks of elevators, escalators and passenger conveyors.

Conceptually, risk analysis is represented by a sequence of logical steps that ensure a systematic approach to the determination of hazards associated with the operation of vertical transportation.

This methodology regulates the events that can lead to damage of the different level, regulated in ISO / CU 14798: 2006.

The next requirement for the risk assessment is the effectiveness of all the available information records and the collection of data that allow qualitative hazard analysis, which is in good agreement with the previously suggested theoretical approaches to the influence factor consideration and the equipment element contribution to the overall safety concept of the operation facility.

Methods and Results

The development of the modified risk assessment methodology

The basic methodology for the reliability assessment of the machine / equipment determins the interrelationships of the probabilities of the operational state $P = P\left(T_{P\gamma}\right) = \frac{\gamma}{100}$ of its assembly units $P_{ij} = P\left(T_{\gamma ij}\right) = \frac{\gamma_{ij}}{100}$ within the operation period [1, 2].

As the limiting state of the engineering system is determined by the simultaneous limiting state of the assembly units of the system, the influence factor determined by the type of element connection becomes significant: successive connection ($P \le \prod_{i=1}^{J} P_i$); parallel connection ($P \le 1 - \prod_{j=1}^{k_i} \left(1 - P_{ij}\right)$).

Considering vertical transportation as a complex technically unsafe system with various factors of influence on risk-failure within the random operational period, the concept of "time lag" is introduced. This concept is mathematically described by Boolean algebra tools as a unit monotone function: $y_1^i\left(X_m\right) = y\left(x_1,..., x_{i-1}, 1, x_{i+1},..., x_m\right)$.

Inherent characteristic of the logical function is variability, the function arguments are Boolean variables (x_1).

The variability of the equipment elements state is represented by "zero", "physical" (real) and "unit" function having a general form $[y(X_m)]$. Mutual influence within a certain operational period is expressed by the equation: $[y_0^i(X_m)] \subseteq [y(X_m)] \subseteq [y_1^i(X_m)]$, in which the elevator is represented as a functional system consisting of m elements.

Using Boolean tools, we preset the safety parameter via the conjunction operator and the risk parameter via the disjunction operator. The inversion operator shows the compatibility of the hazard - safety levels: $E \leftrightarrow 1 \subseteq P \leftrightarrow 0$.

The obvious consequence of the above logical-mathematical steps is the function of the hazardous situation P_c probability during the operation of vertical transportation: $P_c = P(y(X_m) = 1)$.

Considering the factors of the influence on the hazardous situation, a conditional parameter characterizing "the event contribution" to safety is introduced:

$$B_i = P_i \xi_i \,, \tag{3}$$

where B_i is conditional indicator characterizing the "event contribution" to the element safety of the system;

$\xi_i = P(\Delta_x q(X_m))$ is the partial derivative of the probability of the system dangerous operation.

$q_i = \sum_{i=1}^{k} 2^{-(r_i-1)} - \sum_{j=1}^{e} 2^{-(r_i-1)}$ is the level of the deviation vector of the "risk-safety" system probabilistic state for the elements of the complex engineering system, in our case, elevators, as the most mass representatives of the vertical transportation types;

e – is a number;

r_i – is a rank.

Taking into account the previous influence factors, the model for the elevator safety assessment within the actual operational period is presented by the following expression:

$$\xi_i = P_{C_1}^i - P_{C_0}^i \,, \tag{4}$$

where P_{C1i} is the probability of a dangerous situation throughout the operational period under observation, considering one of the failure criteria;

P_{C0i} is the probability of a dangerous situation in the initial operation phase considering one of the failure criteria.

– "contribution of the event":

$$B_i = P_C - P_{C_0}^i \quad , \tag{5}$$

– relative contribution:

$$b_i = \frac{\xi_i P_i}{P_C} \,. \tag{6}$$

Using the given methodology, you can adjust the conditions of technical operation, the requirements for load modes which ensure safety without design modifications.

Furthermore, this methodology allows us to ensure the safety of the system through the main parameter of the components performance in the system such as a set of energy data

$$\overline{F_{эн}} = \frac{m'' - m_{Nom}}{r \cdot m''} \,. \tag{7}$$

where m '' – is the minimum severity of danger;

m_{Nom} is the nominal level of the danger severity.

Table 1. Danger severity aftereffects classification of failures and failure frequency (100 elevators) (in accordance with the analogue SAE J1739)

Danger severity aftereffects	**failure criterion**	**Rank**	**Frequency of failure**
Missing	No aftereffects	1	Up to 0.010
Very slight	Finish of the object does not meet the requirements (noise). The defect is noticed by picky users of the elevator (less than 25%)	2	0.1
Minor	Finish of the object does not meet the requirements (noise). The defect is noticed by 50% of users of the elevator.	3	0.5
Very low	Finish of the object does not meet the requirements (noise). The defect is noticed by most users (over 75%).	4	1
Low	The elevator is operative, but comfort/convenience system is poor and ineffective. Users of the elevator are quite unsatisfied.	5	2
Medium	Elevator is operative, but comfort/convenience system is inoperative. Users of the elevator feel uncomfortable (There is probability for people to get injured, that may pose harm of medium severity to human health).	6	5
High	Elevator is operative, but the efficiency is low. Elevator users are very dissatisfied (There is probability of serious injury to humans).	7	10
Very high	Elevator is inoperative (loss of primary function) (There is probability of either severe injury to humans or fatal outcome).	8	20
Dangerous with danger warning	There is a very high level of severity when a potential failure affects safe operation of the elevator and/or leads to the safety standards discrepancy warning of danger (There is risk of severe injury/fatal outcome)	9	50
Dangerous without danger warning	There is a very high level of danger severity when a potential failure affects safe operation of the elevator and/or leads to the safety standards discrepancy without warning of the danger (There is risk of severe injury/fatal outcome).	10	100

Characterizing the severity of danger, it is necessary to come from the qualitative characteristics to the quantitative analogue. We will assess the severity by the possible damage to the person. It is logical to classify a scenario as of high severity if it results in the person's social opportunity restriction in realizing his or her potential, i.e. infliction of health harm, which cannot be recuperated morally and / or physically.

The parameter of the empirical minimum value of danger severity is determined and the proportionality coefficient having the dimension of the damaging factor is justified.

It seems to be correct to come from categorizing the danger severity as "death" to such notion as "loss of social level" which means the person is alive, but with physical or moral limitations with discreteness of 1x1000:

1 is high = 0.5 * 10-3 (the person is alive, but is able to function 50%);

2 is average = 0.25 * 10^{-3} (reversible incapacitation requiring a certain period of rehabilitation);

3 is low = 0.15 * 10^{-3} (transfer of danger to a stressful situation, rapid recovery);

4 is negligible = 0.

The parameter is accepted on the basis of the concept of the dangerous situation criticality assessment considering human beings, according to the methods developed in the national standard of the Russian Federation "Risk Management. Analysis method of the types and failure aftereffects", which was introduced by Decree of the Federal Agency for technical regulation and metrology on December 27, 2007. article No. 572, which is modified in relation to the international standard IEC 60812:2006 “Analysis methods of system reliability. The analysis method of failure types and aftereffects (FMEA)".

The deviation of parameters from the standard ones, in terms of the danger severity and the risk of a mortal danger for humans is expressed respectively:

$$\Delta_m = \frac{m''_{av.calc}}{m''_{nom}} \qquad \Delta_p = \frac{P_{calc}}{P_{nom.}}. \tag{8}$$

where $m''_{av.calc}$ is the actual value of the danger severity of the lethality risk demonstration;

$P_{calc;\ nom}$ is the probability of the system failure situation with lethality risk, calculated and nominal respectively.

It should be noted that the shifting coefficient of the middle degree of the danger severity plays an important role

$$b = 1 - \eta \ln P, \tag{9}$$

where η – is the shifting coefficient.

P – is the probability of the system failure with the risk of fatal outcome.

By varying the severity limit value (upper - lower limits) we can determine the shifting coefficient, and by making the infogram, we can visually assess the consequences of lowering the limits, which in turn will allow determining the weight limits of the system taken as a whole.

Block diagram of the algorithm for ensuring the safety of components with comparison of the service life of the of the elevator elements is presented in Figure 1.

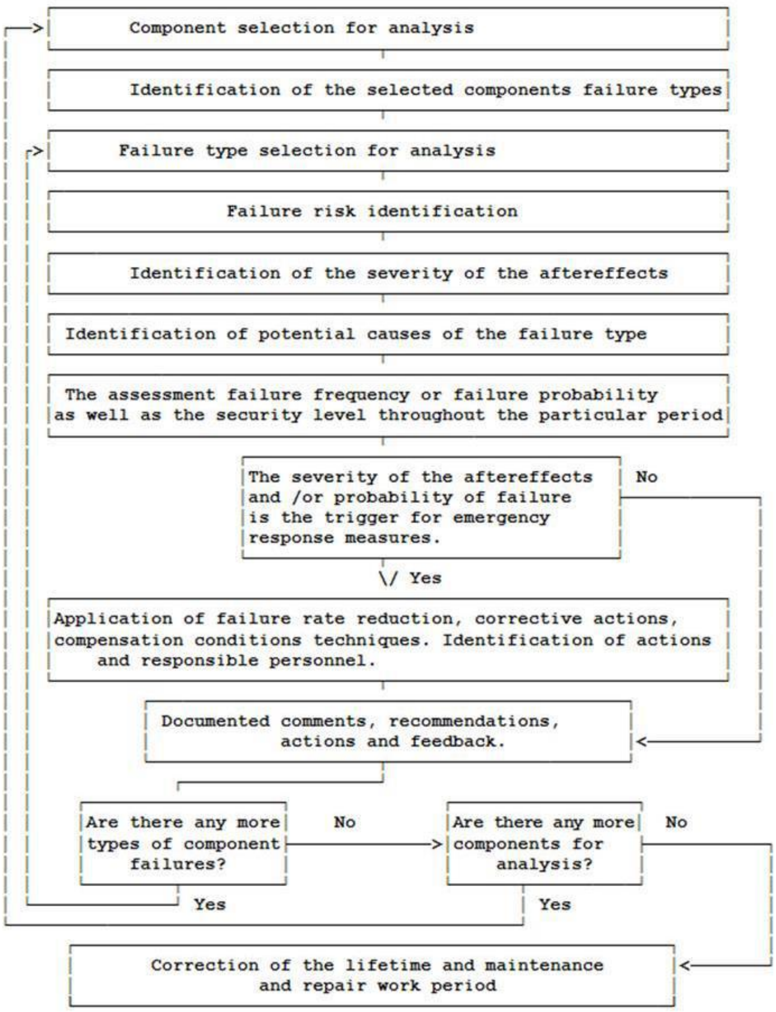

Figure 1. Block diagram of the algorithm for ensuring the safety of components with comparison of the service life of the of the elevator elements

As a result, it becomes possible to determine the safety conditions according to the various criteria.

The typical algorithm of components security with comparison of the service life of the components is made on the basis of the risk assessment scheme methods FMEA or FMECA. As a result, it becomes possible to determine the safety conditions according to the various criteria.

The modified methodology implies compiling the tables of limits of parameter range with determining danger severity as well as calculating the corresponding values of the damaging factors.

It is suggested to apply the developed methodology as the supplement to the existing general methodology for the risk assessment at all operation stages of elevators, escalators and passenger conveyors.

The example of the implementation of the modified methodology when determining the residual operation life of the elevator elements

The initial stage of the elevator safety assessment requires quantifying the gamma-percentile life value

$$T_{\gamma ij} = \frac{T_{\gamma} k_{ij}}{N_{ij}}, \tag{10}$$

where T_{γ} is gamma-percentile life of the equipment. It is taken according to the manufacturer's operational documentation;

k_{ij} is coefficient that allows for the use of assembly units / elements according to the time (0 < k_{ij} <1)/

This parameter for a specific component is determined on the basis of the technological cycle analysis of the equipment operation. So, for the engine, control elements and other assembly units of the elevator $k_i = 1$, we take $k_i = 0.75$ considering the time factor of usage, the intervals of the elements operation, and we take $k_i = 0.15$ allowing for the frequency of the backup system activation.

N_{ij} is the multiplicity of the replacement of the *i*-jth component before the operation period is finished, which corresponds to the limiting state of the equipment. The acceptance of any given value is based on the provision that the elevator components are operated until they reach the limiting state with repetitive replacement until the elevator reaches the limiting state and it is written off the inventory [8, 10–12, 14–16].

Using manufacturers data, the whole elevator design is assessed and the elevator elements are grouped according to the service life parameter : 7 objects – 25 years; 2 objects – 15 years; 16 objects – 12.5 years; 1 objects – 10 years; 6 objects – 5 years (fig.2, top-left box).

As statistical analysis shows, most times elevator structural elements fail in the mechanism, which plays a key role and is the most loaded, that is elevator drive, its elements have different service life parameters specified by the manufacturer, they are Electric engine – 15 years; Reducer, Braking device – 12.5 years; the outlet box – 10 years; Rope driving pulley – 5 years (fig.2, top-right box).

According to the manufacturer initial data, ensuring the components safety algorithm with the comparison of the elevator elements service time is carried out (Fig. 2).

The authors compiled statistics on the elevator failures in residence buildings with different operating periods in Moscow and Moscow region within the framework of expert evaluation of residual operation life. A fragment of the calculations is presented in table 2, where the sample elevator components making a significant contribution to predicting the risk of failure were chosen (the state of danger and guaranteed safety) [2, 11, 14].

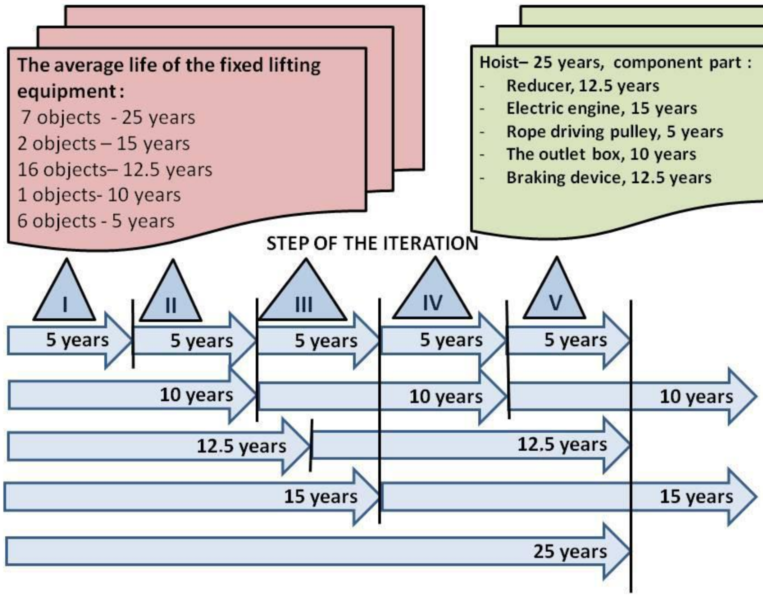

Figure 2 Algorithm for ensuring the safety of components with a comparison of the lifetime of the elements of the elevator

Table 2. A fragment of the calculations, where the sample elevator components making a significant contribution to predicting the risk of failure were chosen

Components of elevator equipment	Conditional indicator which characterizes "the contribution of events" to the security for the elevator element, Bi	Inversion operator (comparison of the levels of risk and safety)	
		$E \leftrightarrow 1$	$P \leftrightarrow 0$
Cabin and shaft doors	0.387	0.880	0.120
Lifting mechanism equipment	0.064	0.990	0.010
Shaft equipment	0.129	0.980	0.020
Drive system	0.096	0.996	0.004
Control system	0.291	0.900	0.100
Alarm unit	0.032	0.999	0.001

At the initial stage of implementation of the developed technique, elevator elements uniform wear is assumed, which is expressed in a 4 % decrease in efficiency per year of normal operation. It is calculated that if the maximum elevator service life is 25 years, in accordance with the manufacturer consideration, service life loss for a year will not exceed 4 %, on condition that the requirements of the qualitative normal operation are observed, repair and maintenance work is carried out with high standard of quality and on time. (The beginning of operation is 100 % – full service life, the end of operation is in 25 years, which is 0% of the service life, the calculations give the value of 100 %:25 years = 4 % for 1 year).

At the initial stage of the developed methodology implementation, it is assumed that the elevator equipment elements wear is the same; it is expressed by 4 % performance degradation per year of normal operation.

An additional requirement for safety is establishing the interdependence of the elements. This is justified by the fact that the elevator design allows for the stand-by system of the individual elements to reduce the risk of a hazardous situation. At the same time, when the backup system is activated, the elevator passes into the mode of abnormal operation conditions requiring the delivery of the passengers to the nearest floor and ensuring the possibility for their evacuation from the cabin. It is logical to conclude that this state is also included in the list of non-safe conditions [2, 3, 9, 14, 17–21].

During the entire service life, the residual operation life decreases taking into account the repetitive replacement of life expired elements, and so, the elevator safety level is calculated:

$$E = \sum_{i=1}^{n} \left(1 - q_i\right) \tag{11}$$

where q – is the level of the deviation vector of the probabilistic state of the system "risk – safety" for the complex technical system elements, in the proposed design methodology it presents a reduction factor of the service life, defined as the ratio of the actual period of operation to the stated service life;

i – is the number of structural elements

n – is the actual period of the operation of the whole equipment.

Statistical data on the elevator elements failure for 1 year of operation [18] are taken as initial data. The level of safety during the operation of the elevator for 1 year is calculated according to the initial data at 4 % wear of the elements.

E_{1yo}=(7*(1-0.04/25)+2(1-0.04/15)+16(1-0.04/12.5)+1(1-0.04/10)+6(1-0.04/5)))/36=0.885.

Similar calculations are performed for the periods of operation taking into account the replacement of the expired service life elements, the results obtained are grouped and presented in the form of the diagram (Fig. 3).

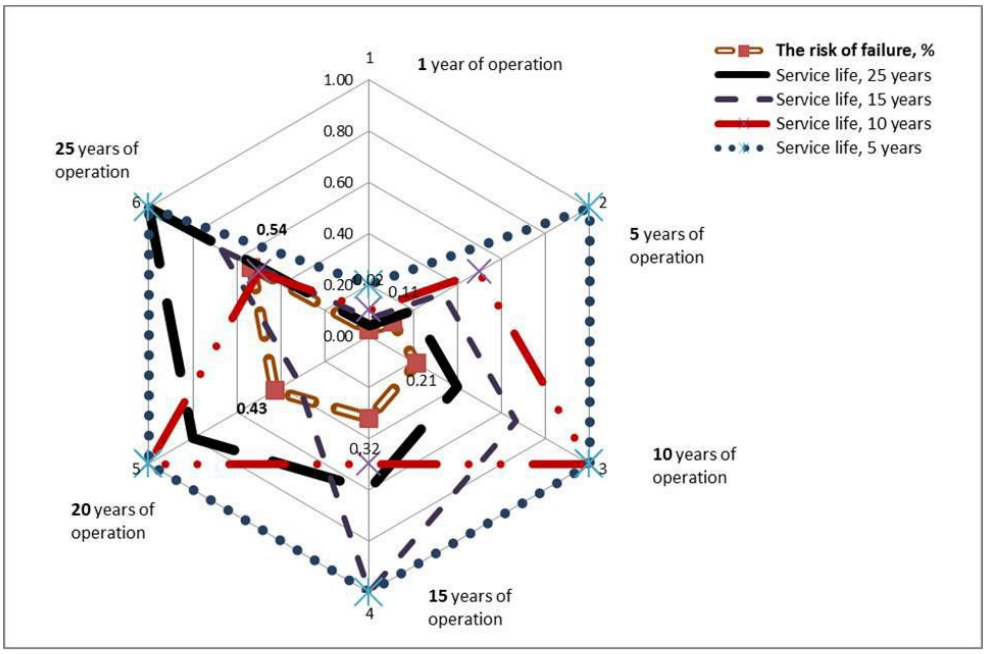

Figure 3 Diagram of the influence of the operating period of the elevator on the change in the probability of risk-failure

Table 3. The results of calculations of the influence of the operating period of the elevator on the change in the probability of risk-failure

The number of objects in the design	Resource	The period of operation					
1		1	5	10	15	20	25
7	25	0.040	0.200	0.400	0.600	0.800	1.000
2	15	0.067	0.333	0.667	1.000	0.330	0.670
1	10	0.100	0.500	1.000	0.500	1.000	0.500
6	5	0.200	1.000	1.000	1.000	1.000	1.000
The percentage of loss		2.14	10.71	21.42	32.13	42.83	53.54

The diagram shows the expired service life elevator components / elements replacement periods and the corrected service life parameter: the safety level 98 % is taken as a reference (this value may vary from 99.9 % to 95 % in accordance with TR CU Safety of elevators); for 1 year operation period the safety level is reduced from the stated 98 % to 88.5 %, which refers to the reduction of the safety level by 2.14 %; after a 10-year period losses increase up to 21.42 %; by the end of the operation period safety level decreases by 53.54 %.

Analysing the obtained results, it can be seen that the risk-failure value increases beginning from the year of operation regardless of the replacement of the time-expired elements. The wear of the remaining currently operated elements affects the risk-failure value.

Discussion

The suggested methodology allows performing similar assessment based on the results of the current condition records of the elevator elements within any period of operation, fixing the current performance parameters specified in the design model exactly for particular elevator [9, 10], which allows us to have quantitative value of the safety level in real time.

The social importance issue of safety insurance when operating vertical transportation has not yet attracted adequate attention of broad scientific camps [18, 22–26].

At the same time the available statistics data on the hazardous situations show the significance of the work performed. With high-rise construction, vertical transportation becomes an integral engineering system for ensuring the public comfort and safety.

The authors believe that the interdisciplinary, systematic approach to the conducted research ensures the stability of the public mobile movement ability when using vertical transportation and at the same time it reduces capital investments in the equipment operated [2, 10, 22, 25, 26].

Conclusion

1. The analysis of the scientific part as well as of the safety measures in the production sphere is performed.
2. Theoretical and practical inconsistency in the issue of taking safety measures in the production sphere has been identified from the ethical-social point of view.
3. It is revealed that the basic theories suggest a probabilistic computation of the risk parameters by modeling monotone logic functions, although in reality the systems are non-monotone functions.
4. It is suggested taking into account the duality of the computational arguments of the function of the engineering systems and expressing the risk-failure and safety parameters by Boolean variables using the example of vertical transportation. All these allow reducing errors in the development of possible hazardous situations scenarios.
5. The modified model of the cost calculation of time losses in non-production downtime of the elevators is suggested by recording the safety parameter quantitatively expressed by the factor of loss due to the elimination of the consequences of the abnormal or hazardous situation.
6. Characteristics of the elevators, escalators and passenger conveyors have been carried out for the risk and corresponding influence factors assessment.
7. The modified methodology for safety assessment of the elevator within the real operational period is worked out. The previous influence factors and the contribution of the event have been taken into account. The methodology allows us to make adjustments both in the conditions of technical operation and in the requirements for under-load operation of the elements ensuring safety without design improvements. It also allows ranging weight limits of the system elements integrally.
8. The example of the implementation of the modified methodology is given when determining the residual life of the elevator elements. The diagram of the effect of the operating period of the elevator on the change of the risk probability parameter is constructed.
9. The presented method of adjusting the service life parameter of vertical transportation according to the failure risk and safety criteria allows us to predict the safety of vertical transportation operations in accordance with the service lifetime as well as to identify the most problematic elements and to prepare a set of spare parts for restoring serviceability in advance.

References

1. Polyakov Yu.I. Teoriya proyavleniya opasnosti: osnovnyye zadachi strategii tekhnicheskogo regulirovaniya bezopasnosti tekhnicheskimi reglamentami [Theory of manifestation of danger: the main tasks of the strategy of technical regulation of safety by technical regulations]. *Vestnik nauchnogo tsentra.* 2008. No. 2. Pp. 154–164. (rus)
2. Sevryugina N.S. *Teoriya formirovaniya tekhnicheskoy bezopasnosti polnogo zhiznennogo tsikla transportnykh i tekhnologicheskikh mashin: monografiya* [Theory of formation of technical safety of the complete life cycle of transport and technological machines: monograph]. Belgorod: Izd-vo BGTU. 2012. 179 p. (rus)
3. Antsev V.Yu., Tolokonnikov A.S., Gorynin A.D. Avtomatizatsiya rascheta riskov vozniknoveniya otkazov gruzovykh kranov [Automation of the calculation of the risks of crane failure]. *Izvestiya TulGU. Tekhnicheskiye nauki.* 2013. No. 7. P. 1. Pp. 214–220. (rus)
4. Bondarev A.B., Yugov A. Otsenka montazhnykh usiliy v metallicheskom pokrytii s uchetom sborki [Evaluation of installation efforts in metal coatings, allowing for assembly process]. *Magazine of Civil Engineering*. 2015. No. 4(56). Pp. 28–37. (rus)
5. Ushakov L.S., Panichkin A.A., Grebenchenko D. Yu., Zhukov M.I. Analiz sposobov i sredstv provedeniya podyemno-transportnykh rabot krupnogabaritnykh gruzov so smeshchennym tsentrom tyazhesti [Analysis of ways and tools of carrying out lifting and transport operations for bulky goods with a shifted center of gravity]. *Mir transporta i tekhnologicheskikh mashin.* 2014. No. 1(44). Pp. 50–54. (rus)
6. Aleksandrova N.P., Chysow V.V. The usage of integral equations hereditary theories for calculating changes of measures of the theory of damage when exposed to repeated loads. *Magazine of Civil Engineering.* 2016. No. 2. Pp. 69–82. doi: 10.5862/MCE.62.7 (rus)
7. Baurova N.I., Zorin V.A., Prikhodko V.M. Description of scenarios of transition of material from an operable to inoperable state using an equation of fold catastrophe theory. *Polymer Science.* Series D. 2015. Vol. 8. No 1. Pp. 1–5.
8. Baurova N.I., Zorin V.A., Prikhodko V.M. Manifestation of a synergistic effect in technological heredity. *Polymer Science.* Series D. 2016. Vol. 9. No 2. Pp. 209–211.
9. Volkov A.A., Roytman V.M., Shilova L.A. Model of stability of life support systems in emergency situations. *International Journal of Applied Engineering Research.* 2016. Vol. 11. No. 3. Pp. 1666–1669.
10. Bogomolov A.A., Bunin M.V., Sevryugina N.S. *Struktura i semantika variatsionnoy optimizatsii transportnykh mashin i tekhnologicheskikh protsessov v obshchey teorii sistem: monografiya* [Structure and semantics of variational optimization of transport vehicles and technological processes in the general theory of systems: monograph]. Bel-gorod: Izd-vo BGTU, 2009. 83 p.
11. Stepanov M.A., Mechiyev A.V. Modeling the system of technical maintenance of elevators fleet. *Scientific review.* 2016. No. 3. Pp. 27–32. (rus)
12. Verigin Yu.A., Tolstenev S.V. *Sinergeticheskiye osnovy protsessov i tekhnologiy: monografiya* [Synergetic bases of processes and technologies: monograph]. Barnaul: Izd-vo AltGTU, 2007. 158 p.
13. Petrik A.I., Bykhovets A.N., Sokharev V.A., Perein V.N., Serdyukov A.L. Modernization of mineral raw materials base in the long-term development strategy of kovdor ore dressing and processing enter-prise. *Eurasian Mining.* 2012. No. 2. Pp. 7–11.
14. Sevryugina N.S., Stepanov M.A., Mechiyev A.V. Obshchemetodologicheskiy podkhod k otsenke riska obespecheniya konstruktivnoy bezopasnosti liftov [The general methodological approach to assessing the risk of

Литература

1. Поляков Ю.И. Теория проявления опасности: основные задачи стратегии технического регулирования безопасности техническими регламентами // Вестник научного центра. 2008. № 2. С. 154–164.
2. Севрюгина Н.С. Теория формирования технической безопасности полного жизненного цикла транспортных и технологических машин: монография. Белгород: Изд-во БГТУ, 2012. 179 с.
3. Анцев В.Ю., Толоконников А.С., Горынин А.Д. Автоматизация расчета рисков возникновения отказов грузовых кранов // Известия ТулГУ. Технические науки. 2013. № 7. Ч.1. С. 214–220.
4. Бондарев А.Б., Югов А.М. Оценка монтажных усилий в металлическом покрытии с учетом сборки // Инженерно-строительный журнал. 2015. № 4(56). С. 28–37.
5. Ушаков Л.С., Паничкин А.А., Гребенченко Д. Ю., Жуков М.И. Анализ способов и средств проведения подъемно-транспортных работ крупногабаритных грузов со смещенным центром тяжести // Мир транспорта и технологических машин. 2014. № 1(44). С. 50–54.
6. Александрова Н.П., Чусов В.В. Применение интегральных уравнений наследственных теорий для расчета изменения мер теории поврежденности при воздействии повторных нагрузок // Инженерно-строительный журнал. 2016. № 2(62). С.69–82.
7. Baurova N.I., Zorin V.A., Prikhodko V.M. Description of scenarios of transition of material from an operable to inoperable state using an equation of fold catastrophe theory // Polymer Science. Series D. 2015. Vol. 8. № 1. Pp. 1–5.
8. Baurova N.I., Zorin V.A., Prikhodko V.M. Manifestation of a synergistic effect in technological heredity // Polymer Science. Series D. 2016. Vol. 9. № 2. Pp. 209–211.
9. Volkov A.A., Roytman V.M., Shilova L.A. Model of stability of life support systems in emergency situations // International Journal of Applied Engineering Research. 2016. Vol. 11. № 3. Pp. 1666–1669.
10. Богомолов А.А., Бунин М.В., Севрюгина Н.С. Структура и семантика вариационной оптимизации транспортных машин и технологических процессов в общей теории систем: монография. Бел-город: Изд-во БГТУ, 2009. 83 с.
11. Степанов М.А., Мечиев А.В. Моделирование системы технического обслуживания парк лифтов // Научное обозрение. 2016. № 3. С. 27–32
12. Веригин Ю.А., Толстенев С.В. Синергетические основы процессов и технологий: монография. Барнаул: Изд-во АлтГТУ, 2007. 158 с.
13. Petrik A.I., Bykhovets A.N., Sokharev V.A., Perein V.N., Serdyukov A.L. Modernization of mineral raw materials base in the long-term development strategy of kovdor ore dressing and processing enter-prise // Eurasian Mining. 2012. № 2. Pp. 7–11.
14. Севрюгина Н.С., Степанов М.А., Мечиев А.В. Общеметодологический подход к оценке риска обеспечения конструктивной безопасности лифтов // Механизация строительства. 2017. Т. 78. № 4. С. 24–29.
15. Volkov A.A., Chulkov V., Kazaryan R. Innovative representation of normative support in high-rise construction // Procedia Engineering. 2014. Vol. 91. Pp. 373–376.
16. Шарапов Р.Р., Мамедов А., Степанов М.А., Харламов Е.В. Шагающее ходовое оборудование стрелового самоходного крана грузоподъемностью свыше 100 т повышенной проходимости и мобильности // Механизация строительства. 2016. № 77. С. 5–11.
17. Горопашная А.В. Оценка важности аргументов немонотонных логических функций при логико-вероятностном анализе безопасности сложных

constructive safety of elevators]. *Mechanization of Construction*. 2017. Vol. 78. No. 4. Pp. 24–29.

15. Volkov A.A., Chulkov V., Kazaryan R. Innovative representation of normative support in high-rise construction. *Procedia Engineering.* 2014. Vol. 91. Pp. 373–376.
16. Sharapov R.R., Mamedov A., Stepanov M.A., Kharlamov Ye.V. Walking running equipment of small self-propelled cranes with a lifting capacity of over 100 tons of terrain and mobility. *Mechanization of Construction*. 2016. No. 77. Pp. 5–11.
17. Goropashnaya A.V. Otsenka vazhnosti argumentov nemonotonnykh logicheskikh funktsiy pri logiko-veroyatnostnom analize bezopasnosti slozhnykh tekhnicheskikh sistem [Estimation of the importance of the arguments of nonmonotonic logic functions in the logic-probabilistic analysis of the safety of complex technical systems]. *Vestnik of Saint Petersburg University.* Ser.10. 2009. No. 1. Pp. 19–32. (rus)
18. Telichenko V.I., Zavalishin S.I., Khlystunov M.S. Globalnyye riski i novyye ugrozy bezopasnosti otvetstvennykh stroitelnykh obyektov megapolisa [Global risks and new threats to the security of the responsible construction sites of the metropolis]. Sb. dokladov tematicheskoy nauchno-prakticheskoy konferentsii «Gorodskoy stroitelnyy kompleks i bezopasnost zhizneobespecheniya grazhdan». Moscow: MGSU, 2005. Pp. 211–218. (rus)
19. Baurova N.I., Zorin V.A., Prikhod'ko V.M. Influence of structural defects in carbon fibers on their sensor properties evaluated using scanning electron microscopy. *Fibre Chemistry.* 2015. Vol. 46. No. 5. Pp. 283–287.
20. Kozlov V.V. The dynamics of systems with servoconstraints. *Regular and Chaotic Dynamics.* 2015. Vol. 20. No. 3. Pp. 205–224.
21. Sharapov R.R., Sevryugina N.S., Kapyrin P.D. The solution to the problem of training qualified personnel in the industry by pairing the requirements of technical regulations of the customs Union "Safety of lifts", professional standards and educational programs. *Mechanization of Construction*. 2016. No. 10. Pp. 57–62. (rus)
22. Sharmanov V.V., Simankina T.L., Mamaev A.E. BIM in the assessment of labor protection. Magazine of Civil Engineering. 2017. No. 1. Pp. 77–88. doi: 10.18720/MCE.69.7.
23. Nekliudova E.A., Semenov A.S., Melnikov B.E., Semenov S.G. Experimental research and finite element analysis of elastic and strength properties of fiberglass composite material. *Magazine of Civil Engineering.* 2014. No.3(47). Pp. 25–39. (rus)
24. Zhabin A.B., Fomichev A.D., Naumov Ju.N., Solovyh D.Ja. Results of studies of shaft boring machine operation in vertical shaft construction at upper kama potash deposit. *Eurasian Mining.* 2016. No. 1(25). Pp. 29–32.
25. Volkov A.A., Lysenko D.A., Doroshenko A.V., Sedov A.V., Chelyshkov P.D. Computer modelling in evaluating standard projects of school buildings. *International Journal of Applied Engineering Research.* 2015. Vol. 10. No 22. Pp. 43269–43272.
26. Volkov A., Chelyshkov P., Lysenko D. Information management in the application of bim in construction. Stages of construction. *Procedia Engineering*. 2016. Vol. 153. Pp. 833–837.

технических систем // Вестник Санкт-Петербургского университета. Сер.10. 2009. № 1. С. 19–32.

18. Теличенко В.И., Завалишин С.И., Хлыстунов М.С. Глобальные риски и новые угрозы безопасности ответственных строительных объектов мегаполиса // Сб. докладов тематической научно-практической конференции «Городской строительный комплекс и безопасность жизнеобеспечения граждан». М.: МГСУ, 2005. С. 211–218.
19. Baurova N.I., Zorin V.A., Prikhod'ko V.M. Influence of structural defects in carbon fibers on their sensor properties evaluated using scanning electron microscopy // Fibre Chemistry. 2015. Vol. 46. No. 5. Pp. 283–287.
20. Kozlov V.V. The dynamics of systems with servoconstraints // Regular and Chaotic Dynamics. 2015. Vol. 20. № 3. Pp. 205–224.
21. Шарапов Р.Р., Севрюгина Н.С., Капырин П.Д. Решение проблемы подготовки квалифицированных кадров отрасли путем сопряжения требований ТР ТС «Безопасность лифтов», профессиональных стандартов и образовательных программ // Механизация строительства. 2016. Т. 77. № 10. С. 57–62.
22. Шарманов В.В., Симанкина Т.Л., Мамаев А.Е. BIM технологии в оценке уровня охраны труда // Инженерно-строительный журнал. 2017. № 1(69). С. 77–88
23. Неклюдова Е.А., Семенов А.С., Мельников Б.Е., Семенов С.Г. Экспериментальное исследование и конечно-элементный анализ упругих и прочностных свойств стекловолоконного композиционного материала (англ.) // Инженерно-строительный журнал. 2014. № 3(47). С. 25–39.
24. Zhabin A.B., Fomichev A.D., Naumov Ju.N., Solovyh D.Ja. Results of studies of shaft boring machine operation in vertical shaft construction at upper kama potash deposit // Eurasian Mining. 2016. No. 1(25). Pp. 29–32.
25. Volkov A.A., Lysenko D.A., Doroshenko A.V., Sedov A.V., Chelyshkov P.D. Computer modelling in evaluating standard projects of school buildings // International Journal of Applied Engineering Research. 2015. Vol. 10. No. 22. Pp. 43269–43272.
26. Volkov A., Chelyshkov P., Lysenko D. Information management in the application of bim in construction. Stages of construction // Procedia Engineering. 2016. Vol. 153. Pp. 833–837.

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Sevryugina N.S., Stepanov M.A. Vertical transport: resource by the criterion of safety. *Magazine of Civil Engineering*. 2017. No. 7. Pp. 23–36. doi: 10.18720/MCE.75.3.

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Spun concrete properties of power transmission line supports

Свойства центрифугированного бетона опор линий электропередач

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Key words: spun concrete; physical and mechanical properties of the fresh concrete, mechanical properties, hardened concrete, power transmission tower

Ключевые слова: центрифугированный бетон, физико-механические свойства свежеуложенного бетона, прочностные характеристики бетона, опоры линий электропередач

Abstract. The article presents results of investigation study of physical and mechanical properties of spun and vibrated samples. Heterogeneity of freshly placed concrete was assessed for change in water to cement proportion, residual water content, density throughout the spun sample height, as well as changes in mechanical properties of hardened concrete. Analysis of experimental data showed a significant change (up to 4%) in the average density of concrete throughout the spun sample, while the overall voids content in the sludge layer increased by almost 10%, and strength of concrete changes by 18-25% along the lift height. By using the method of least squares, a consistent change in the strength of concrete along the lift height of samples was observed. An experiment, assessing the bearing capacity of spun pylons, accounting for the resulting changes in strength of concrete along the wall height, was carried out.

Аннотация. В статье приведены результаты экспериментальных исследований физико-механических свойств центрифугированных и вибрированных образцов. Проведена оценка неоднородности свойств свежеуложенной бетонной смеси для определения изменения водоцементного отношения (В/Ц), остаточного водосодержания и плотности по толщине центрифугированного образца, а также изменение прочностных свойств затвердевшего бетона. Анализ экспериментальных данных показал, что наблюдается значительное изменение средней плотности бетона по толщине центрифугированного образца (до 4%), при этом общая пористость в шламовом слое увеличивается почти на 18%, а прочность бетона изменяется по толщине на 18-25%. Используя метод наименьших квадратов, получена закономерность изменения прочности бетона по толщине образцов. Проведен численный эксперимент по исследованию несущей способности центрифугированных опор линий электропередач с учётом полученной зависимости изменения прочности бетона по толщине стенки изделия.

Introduction

Reinforced concrete structures of annular cross section attract researchers' attention in the beginning of the last century. Prof. Schule F [1] conducted one of the first studies in the Swiss laboratory; results of which were published in 1908.

Interest in such structures especially increased with introduction of spun manufacturing products. In 1933–1935, engineer P.A. Abeles [2, 3] conducted experiments on bends of reinforced concrete elements of annular cross section of different diameter, under varying value of reinforcement percentage. At the same period prof. V.V. Mikhailov [4] and prof. S.A. Dmitriev conducted research of different spun structures and offered calculation methods as well as means of prestressing in laboratory and semi-industrial conditions.

Spun reinforced-concrete poles and masts [5, 6] were widely used in 50–70th of the last century in Russia for high-voltage power transmission line (PTL) supports, due to the rapid development of power construction America and the European countries.

Дедух Д.А., Щуцкий В.Л., Кузьменко А.А. Свойства центрифугированного бетона опор линий электропередач // Инженерно-строительный журнал. 2017. № 7(75). С. 37–51.

I.N. Akhverdov [7, 8], S.A. Dmitriev [9], V.M. Batashev[10, 11], R.R. Valduga [12–14], A.P. Kudzis [15–22], R. Kliukas [23], V. Sh. Kalandadze [24], E.E. Mikhelson [25], V.I. Soroker [26–28] and other scholars studied strength, crack resistance and deformability of PTL supports as well as physical and mechanical properties of the spun concrete in different years.

Methods of vibrated concrete strength measurement are well studied and rated. Physical and mechanical properties of the spun concrete are less studied, and there is no standard method of strength measurement in normative documents.

Offers of different studiers relating to spun concrete strength measurement can be divided into following groups:

- according to the results of vibrated samples with initial water to cement proportion $(W/C)_{init}$ having regard to conversion factor testing [13, 14];

- according to the results of vibrated samples with residual water to cement proportion $(W/C)_{res}$ testing [27, 28];

- according to the results of spun cubes testing [11, 17, 27];

- according to the results of annular cross section spun samples testing[12–14, 17, 26];

- according to the results of cubes (samples) cut out from spun hollow circles testing [24].

Each of the listed methods has special features and reflects the actual strength of spun concrete in manufactured item with various reliability. For example, determining the strength of spun concrete from the test results of vibrated cubes of concrete mixture with initial water/cement ratio, can result in significant error, because it does not account for the influence of the structure of the spun concrete wall along the height, the mode of centrifugation and some other factors.

I.N. Akhverdov has conducted the most extensive research in the field of spun concrete structure in the Russian Federation. He notes that the pressing pressure changes during the manufacturing of the product, and has the greatest value at the surface. Therefore, water is pressed optimally from an outer layer of a wall. Besides, a large number of filtration channels are found along the wall height, section and quantity of these channels increases in wall inner surface direction. It leads to concrete density and strength change along the manufactured item height.

When assessing the strength of spun concrete from the results of vibrated cubes of concrete with water/cement residual ratio [27], the actual structure of concrete wall is not considered, and the process of concrete mix selection is very time consuming.

Annular cross section samples testing provides the most exact data about spun concrete strength in the manufactured item, but samples are unwieldy, tests are time consuming and demand pressure equipment of high power [10, 11].

The results of sample (cubes, prisms, hollow circles) cut out from natural structures testing can provide the actual spun concrete strength with adequate accuracy. But all the preparatory works before testing are highly time consuming and demand special equipment.

The strength of spun concrete is most frequently determined by taking into account the conversion factors. But these factors considerably differ in works of different authors. R.R. Valduga and A.P. Kudzis [12, 14] suggest conversion factor value equal to 1.18, V.I. Soroker [27, 28] – 1.5–1.7, V.Sh. Kalandadze [24] – 1.37, E.E. Mikhelson [25] – 1.35.

Taking into account the data indicated below we can assume, that methods of spun concrete mechanical properties measuring require further research. And the regularity of concrete mechanical properties change throughout the manufactured item wall is almost not studied.

Methodology of the research

Methodology of the material and concrete research

Rationale for the raw materials for the experiment

Normative documents for the manufacturing of centrifuged reinforced concrete structures define main requirements for quality of raw materials for concrete mixing.

Use of Portland cement without additives or with mineral additives of cement grade 400 and higher as the binding agent is acceptable; and granulated blast-furnace sludge can be used as a mineral admixture in an amount no greater than 20 % of the cement mass. Use of Portland sulfate-resisting cement and Portland cement that is meant for concrete surfacing of roads is acceptable.

These requirements to the cement are associated with the fact that the material composition of the used Portland cement changes in the centrifugation process, since light floured additives are pressed to the inner cavity of the product and go to the sludge.

It is noted that cement paste normal consistency (CPNC) is one of the main factors, which influences the strength and uniformity of centrifuged concrete. It should not be more than 28 %.It is specified in the studies of different authors [8, 14, 26], that changing of CPNC from 24 to 28 % increases centrifugation durability by 1.3 times. Increasing the strength of spun concrete by increasing the consumption of cement over its optimal content does not produce a proportional effect. Besides, it was determined that increasing spending of cement over 500 kg/m3 doubles heat-shrinking deformation.

According to normative documents, the use of coarse- and average-grained natural and crushed sands as the fine aggregate is acceptable. In the case when fine sand is used, it is necessary to increase centrifugation durability [28]. Increased water requirement of fine sand mixes leads to increment of initial water /cement ratio $(W/C)_{init}$ and demands extra cement to warranted concrete strength. For example, increasing of the initial water /cement ratio $(W/C)_{init}$ from 0.35 to 0.41 leads to concrete strength reduction on the average by 28 %, in such case transition coefficient value changes from vibrated samples strength to centrifuged samples strength.

Use of gravel or crushed gravel of hard rock and freeze-proof rock is acceptable as the coarse aggregate. Gravel strength should be as much as twice the concrete strength. Coarse aggregate size regulations are especially specified. It is recommended to perform gauging of two fractions of size 5–10 mm and 10–20 mm separately by proportion of 1:1.5 between them and at maximum allowable voids ratio of mix as much as 40 %.

In our research [29, 30] we used fine-grained mixes a model concrete mix, consisting of stone screening dust of fractions size 2.5–5 mm, refined glass sand of fractions size 0.14–2.5 mm and Portland cement without admixtures made by Novorossisk cement plant “Oktyabr”. The choice of raw materials completely corresponded to the requirements of centrifuged concrete normative documents described above.

Fine-grained concrete mix segregation in the process of centrifugal consolidation will surely be less expressive than segregation of ordinary concrete mix, but we were limited by the size of the formed sample and the necessity of sample fragmentation for physical and mechanical properties measurement of the centrifuged concrete.

For this very reason all the mentioned tests were conducted on fine-grained concrete mixes.

Quality evaluation of the used raw materials.

Properties measurement of the used Portland cement was conducted according to the Russian State Standard GOST procedure 310.1…310.3-76 “Concretes: Test methods. General requirements. Methods of grind fineness, normal consistency, setting up time and soundness measurement.” And quality evaluation was conducted as per Russian State Standard GOST 10178-85 “Portland cement and Portland blast-furnace sludge cement. Technical regulations.”

Properties measurement of glass sand of fractions size 0.14–2.5 mm and stone screening dust of fractions size 2.5–5 mm was conducted according to the Russian State Standard GOST procedure 8735-88 “Sand for construction activity. Test methods.” And their quality evaluation was conducted as per Russian State Standard GOST 26633-2015 “Heavy concrete. Aggregates technical requirements.” Potable mains water, which complies with the requirements of Russian State Standard GOST 23732-2011 “Water for concrete and mortar. Technical regulations.” was used as mixing water.

Basic properties of the used raw materials (concrete, glass sand and stone screening dust) as well as their quality evaluation are presented in Table 1.

By these means fine-grained concrete mix components, which are used in tests, comply requirements of the normative documents and can be used for centrifuged concrete mixing.

Table 1. Properties of the initial raw materials

Name of the component	Basic properties	Compliance with the requirements of normative documents
Portland cement without admixtures made by Novorossisk cement plant "Oktyabr"	Cement fineness (rest on a sieve 008) – 9 %. Setting up time: - initial set 2h. 10 min. - final set 3 h. 50 min. Soundness – bears the activity of Rц = 43.5 MPa Flexural strength of 28 days – 5.65 MPa	Complies GOST Standard 10178-85*and has M400 grade
Refined glass sand of the Volzhsky occurrence	Average bulk density 1510 kg/m^3 Density – 2.66 g/cm^3 Void ratio – 43.2 % Fineness modulus = 1.8 Content of flour and clay particles – are absent Content of organic impurities – are absent	Complies GOST Standard 8736-93* "Sand for construction activity. Technical regulations."
Granit crushing riddlings Pavlovsky quarry (fractions size 2.5–5 mm)	Average bulk density 1300 kg/m^3 Density – 2.67 g/cm^3 Void ratio – 51% Content of flour and clay particles – are absent Content of organic impurities – are absent	Complies GOST Standard 8736-93* "Sand for construction activity. Technical regulations."

Investigation of centrifuged concrete mix properties

The analysis of the experimental studies [29, 30] of heterogeneity and centrifuged concrete strength along the product wall was conducted. The experiments were conducted on test cylinders with a diameter of 6.5 cm and height of 8 cm in laboratory condition with compliance with GOST Standard 18105-2010. The possibility of the laboratory setup, Centrifuge, to form the samples is responsible for the choice of the former. Measurement assurance of the experimental results validity was reached by means of parallel tests on 5 set of vibrated and centrifuged samples with 5 twin samples in each. Low-slump fine-grained concrete mix (Cone Slump = 3-4sm) was used for investigations as well as for production of the actual centrifuged reinforced concrete supports. Initial concrete mix composition per 1 m^3: cement – 500 kg, water – 225 l, sand – 745 kg, stone screening dust – 915 kg for an average density of the concrete mix – 2385 kg/m^3.

The evaluation of centrifuged concrete heterogeneity was conducted in two stages. The freshly-placed concrete mix properties were studied at the first stage, the properties of the hardened concrete - at the second one.

The production of the cylinder samples was made in special individual forms with removing bottom plate and the sample consolidation was made by using laboratory Centrifuge.

Before centrifugation the weighted concrete mix was placed into a mold in two layers with 15 times rodding of each layer. Then the next centrifugal consolidation mode for the experimental samples according to the GOST Standard 22687.0-85 and GOST Standard 22687.3-85 [6] was set on.

- accelerating to the 300 r/min rotation speed – 2 min;

- cure by n=300 r/min – 1 min;

- accelerating to the 500 r/min speed - 2 min;

- cure by n= 500 r/min – 15 min;

TOTAL: 20 min.

As-formed samples were weighted once again to measure pressed sludge quantity, the molds were removed, and as-formed samples were divided into three parts along their height.

Thereupon, each piece was divided into two portions. The first portion was to measure residual water of mixing. The second one was washed off through the 0.071 mm sieve and dried to fixed-mass. The measurement of cement paste content in each layer of the formed sample became possible after dry mineral components were weighted. The quantity of mixing water ΔB, that passed to sludge by centrifugal consolidation of the experimental samples, determines from the formula:

$$\Delta B = \frac{(M_H - M_K)}{V}; \tag{1}$$

where $M_н$ – concrete mix mass before consolidation, g;

$M_к$ – concrete mix mass after consolidation, g;

V – sample volume, cm^3.

To determine sample volume, we measured its height with caliper with an accuracy of 0.1 mm straight after pressing-out.

To determine the quantity of retained mixing water we used the following dependence:

$$B_{ост} = \frac{M_{vl} - M_c}{M_{vl}} \cdot \rho_{bc}^{c}; \tag{2}$$

where M_{vl} – mass of the wet concrete mix portion, g;

M_c – mass of the dry concrete mix portion, g;

ρ_{bc}^{c} – an average density of the concrete mix after the centrifugal consolidation, g/cm^3.

To determine ρ_{bc}^{c} we measured as-formed sample mass and volume.

Investigation of the hardened concrete strength

Solidification of the formed samples was performed in two stages: curing in the laboratory curing room and supplementary standard curing.

Molded samples in forms after 2–3 hours of air storage were placed in the laboratory curing room with automatic control of the cycle mode and were steam cured according to the regime:

– temperature rise up to 80° C – 2 hours;

– isothermal warming at a temperature of 80° C – 10 hours;

– cooling in natural conditions.

Demolding operation and marking of the cured samples was carried out after cooling. Then they were placed in special capacities and matured up to the age of 28 days in wet scrubs (relative humidity of the atmosphere 90–100 %) at the temperature of 20 ± 5° C.

When reaching the age of 28 days the samples were deplaned out from curing room and dried to fixed-mass. Then they were placed in special plastic bags and were kept there until the test.

Samples preparation constituted of dimension measurement, then they were weighted and scanned by ultrasonic device. The samples of each set were divided into two parts. The first part that consisted of three centrifuged cylinders, was tested on compression.

The second part that consisted of two centrifuged cylinders, was exposed to layer-by-layer cutting into three pieces along height.

Then the edges of each piece were trimmed extensively. After dimension measuring, weighting and scanning by ultrasonic device the pieces were tested on compression.

The special test for the purpose of showing “Concrete strength to ultrasonic sound propagation velocity” calibration curve was conducted for actual control of concrete strength by ultrasonic sound propagation velocity. The test procedure and its elaboration corresponded to GOST Standard 17624-2012 “Concretes. Ultrasonic method of strength measurement.” Instrument inverters were placed against the samples sides due to the fact that in order to measure ultrasonic travel time in the samples we used through-scanning method.

3 sets of cube-samples, 10 cm on edge (6 samples in each set), were produced for the purpose of this test. Planned concrete grades in each set were M400; M500 and M600. The samples had been dried for 28 days after concrete hardening, then the ultrasonic travel time in each sample was measured.

The ultrasonic travel time in each sample was measured at least three times. The measurement result of the ultrasonic travel time in the sample was ignored in the calculation of the ultrasonic travel time in a given sample set, if there was a departure of more than 5 % of measurement result of the single ultrasonic travel time measurement in each sample from the mean arithmetical value of the measurement results for a particular sample.

The concrete samples strength was determined by compression testing, by applying the compression apparatus according to the GOST Standard 18105-2010 “Concretes. Strength testing rules”. Then we made the results equal to standard sample strength value by multiplying the obtained result by scaling factor, that is equal to 0.91.

The primary investigation of the obtained results has shown a little data spread within each set; this has made it possible to start data computing.

The obtained results with their statistical analysis are represented in Table 2.

The arithmetical average of strength, speed and ultrasonic travel time in set samples were taken as a unit value of these properties in determining the “Concrete strength to ultrasonic sound propagation velocity” calibration curve.

The abnormal results processing of the testing of single samples in a set followed the rule: the test result of the single sample will be considered to be abnormal and will be ignored in calculating of the average result in the set, if “T” value, which is determined from the formula (3), is a subject to the condition:

Table 2. Main results of the test

Set and number of samples	Ultrasonic travel time, us					Breaking strength, kg-f	
	At the top of a sample	In the middle part of a sample	At the bottom of a sample	Average in a sample	Average in a set	A single sample	Average in a set
1-1	26.66	25.7	25.6	25.9	25.52	48300	48250
1-2	25.9	25.5	25.5	25.7		46625	
1-3	25.5	25.0	25.2	25.2		49625	
1-4	25.8	25.5	25.4	25.7		48250	
1-5	25.8	25.6	25.6	25.7		48875	
1-6	25.0	25.5	25.0	25.2		47850	
2-1	24.3	24.0	23.9	24.1	24.18	64250	66810
2-2	24.1	24.2	24.0	24.1		64250	
2-3	24.8	24.4	24.3	24.5		67125	
2-4	24.4	24.1	24.0	24.2		68750	
2-5	24.0	24.3	23.8	24.0		68500	
2-6	24.3	24.4	24.1	24.3		68000	
3-1	23.6	23.5	23.4	23.5	23.73	72250	70810
3-2	23.3	23.3	23.2	23.3		74625	
3-3	24.0	23.9	24.0	24.0		71250	
3-4	24.0	24.0	23.9	24.0		67500	
3-5	24.2	24.0	23.7	24.0		71000	
3-6	23.6	24.0	23.5	23.7		68250	

$$T = \frac{(X_1 - X_J)}{S} \geq 1.74, \tag{3}$$

where X_J is an average strength or ultrasonic sound propagation velocity (travel time) of the set of samples;

S – mean root square deviation of strength or sound propagation velocity (travel time);

$$S = \frac{\sum_{J=1}^{N}(X1_{max} - X1_{min})}{1.69N}, \tag{4}$$

where $X1_{max}$ and $X1_{min}$ – maximal and minimal test results of samples in a set;

N – number of sample sets used in the test.

Average concrete density, concrete voids content, ultrasonic sound propagation velocity; compression strength determined from “Concrete strength to ultrasonic sound propagation velocity” calibration curve, actual compression strength were evaluated as outcome parameters.

Concrete voids content calculation was determined from the formula:

$$\Pi_o = \left(1 - \frac{\rho_0^c}{\rho^c}\right) \cdot 100; \quad (5)$$

where ρ_0^c – average concrete density, g/cm³;

ρ^c – actual concrete density, g/cm³;

Results and Discussion

Results of investigation of concrete mix

Comparative data on heterogeneity of the properties of fresh concrete are shown in Table 3

Table 3. Average density and water/cement ratio of concrete mix

Layer of concrete mix in a sample	Average density		Actual residual flow of water in mix		Actual residual content of cement in mix		Residual water/cement ratio
	kg/m³	variation,%	l/m³	variation,%	kg/m³	variation,%	
top	2282	1.48	223	2.06	477	0.96	0.47
middle	2295	0.44	218	3.03	485	1.16	0.45
bottom	2296	0.58	219	2.01	488	1.19	0.45
whole sample	2291	1.07	220	1.41	484	0.871	0.46
inner	2290	2.92	142	8.6	495	0.47	0.287
middle	2338	2.10	133	5.5	478	0.50	0.277
outer	2375	1.51	123	5.3	466	1.23	0.263
whole sample	2334	1.07	133	5.0	480	0.99	0.275

Experimental graphs of variance of compacted concrete physical properties along a sample height are represented in Figure 1.

Analysis of the received experimental dependence represents a significant difference in quantity of residual mixing water and cement in each layer of the compacted concrete sample. The average density of the inner layer decreased by 6-7%, but consumption of residual water and cement increased. In such manner the conducted test provides not only an illustration of concrete mix segregation along samples depth by centrifugal consolidation, but allows to quantify this heterogeneity of properties as well. Different values of $(W/C)_{res}$ in each sample layer are critically important conditions in determination of physical and mechanical properties of hardened concrete. $(W/C)_{res}$ value of compacted concrete mix in the inner layer (towards centrifuge axis) of concrete is 10-12% greater than in the outer layer.

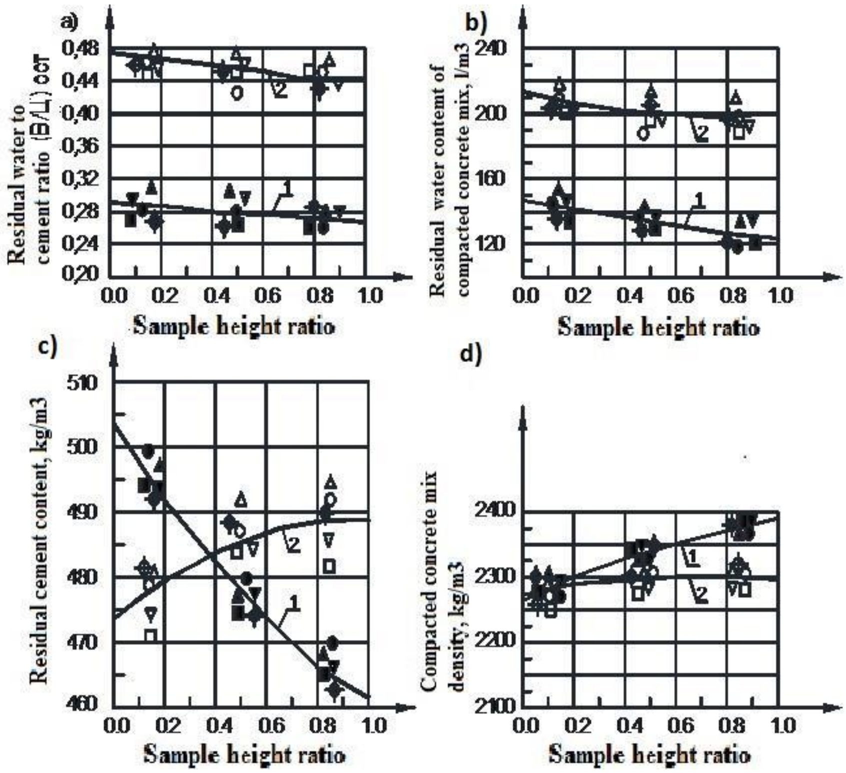

Figure 1. Variance of compacted concrete physical properties along a sample height (a – residual water/cement ratio; b – residual content of water; c – residual cement content; d – average density).

1 – centrifuged samples

2 – vibrated samples

▲ △ - the 1st set
● ○ - the 2nd set
■ □ - the 3rd set
▼ ▽ - the 4th set
⊕ - the 5th set

Results of investigation of concrete strength along a wall height

Comparative data on evaluation of structural heterogeneity of hardened concrete is represented in Table 4.

Table 4. Average density, porosity and strength of concrete

Layer	Average concrete density		Concrete porosity		Compressive strength		
	kg/m^3	variation,%	%	variation,%	Determined from the calibration curve, MPa	actual	
						MPa	variation, %
a) vibrated samples							
top	2260	0.96	16.72	4.7	50.66	51.28	4.03
middle	2268	1.28	16.42	6.4	52.06	52.08	4.84
bottom	2268	1.08	16.42	5.9	51.20	51.72	3.02
whole sample	2265	1.40	16.52	4.9	51.31	51.69	0.80
b) centrifuged samples							
inner	2270	0.51	16.23	2.10	59.22	60.26	3.51
middle	2317	0.31	14.40	1.62	64.93	65.04	2.64
outer	2356	0.33	13.30	2.78	72.78	73.89	4.40
whole sample	2314	0.20	14.64	1.62	65.64	66.39	1.91

Experimental graphs of variance of average concrete strength and compressive strength through sample height are represented in Figure 2.

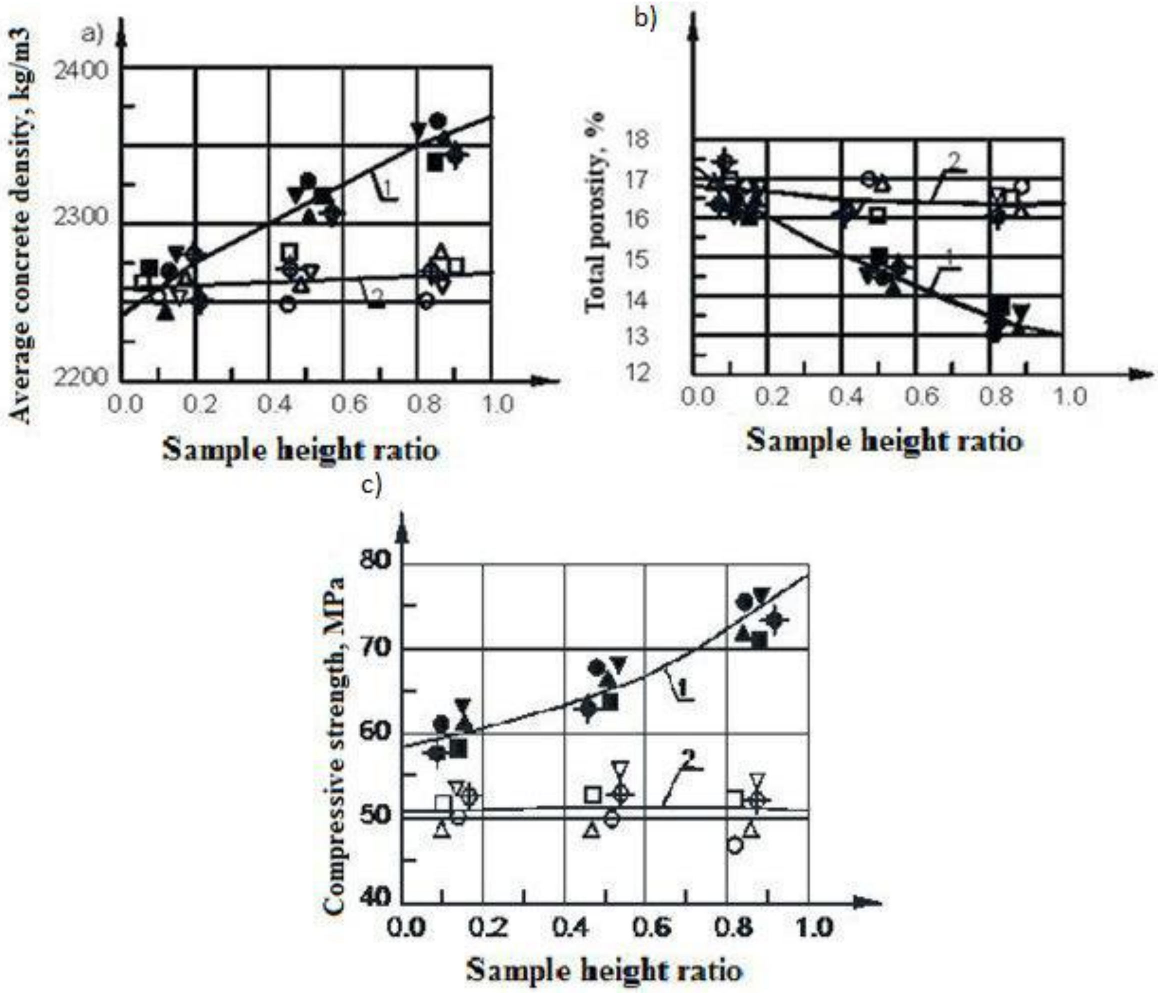

Figure 2. Variance of physical and mechanical properties of concrete along a sample height.
(a – average concrete density; b – total porosity; c – compressive strength)

1 – centrifuged samples

2 – vibrated samples

▲ △ - the 1st set
● ○ - the 2nd set
■ □ - the 3rd set
▼ ▽ - the 4th set
⯁ ⌖ - the 5th set

Analysis of the received data revealed that centrifuged concrete properties in samples sawn by layers (fragments) change considerably. The average concrete density change is observed along the height (up to 4 %), the total porosity in sludge layer is up almost by 18 %, and concrete strength in the outer layer is up by 18–25 %. Gross data of property distribution along a sample height conforms fully to analysis results of fresh-placed concrete mix properties.

A mathematical treatment of the received experimental data was conducted to determine a dependence of centrifuged concrete strength change along a product wall. Parabolic, hyperbolic and linear relations were taken as approximating functions.

Different regression equations describing the concrete strength change dependence along the height were obtained by the least square method (Fig. 3).

Particularly for parabolic function the following regression equations is provided:

$$R_b = R_{b1} + 2{,}064 \cdot \delta_x + 18{,}382 \cdot \delta_x^2; \quad (6)$$

where R_{b1} – concrete strength on the inner surface of a centrifuged product (MPa);

δ_x – wall height ratio

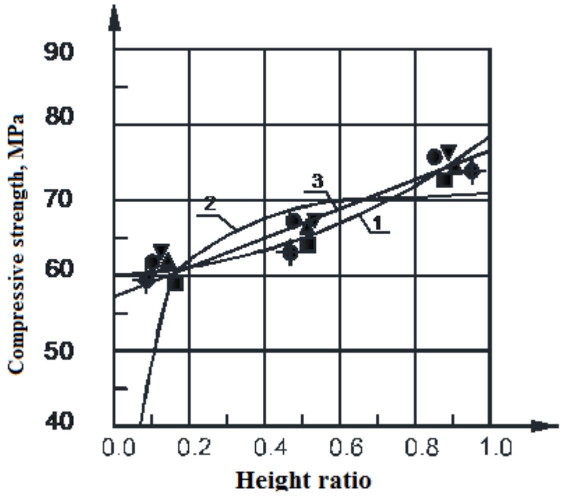

Figure 3. Comparison of mathematical relationships describing the patterns of change in the strength of centrifuged concrete

1 – parabolic relation

2 – hyperbolic relation

3 – linear relation

▲ - the 1st set

● - the 2nd set

■ - the 3rd set

▼ - the 4th set

◆ - the 5th set

Analysis of the experimental data revealed that the relationship of centrifuged concrete strength on the inner surface R_{b1} to the average strength along the height $R_{bк}$ was changing within the following limits $R_{b1}/R_{bк}$=0.87-0.94.

To assess the reliability of the obtained dependence (6) of concrete strength change a numerical experiment, investigating the bearing capacity of power transmission line supports was conducted [31–34].

Tower bodies of high-voltage power transmission lines were taken as study samples according to the GOST Standard 22687.0-85, 22687.3-85. The recommendations and directions presented in design codes Eurocode 2 and ACI 318-05 are not perfect and not fully formulated. This can lead to groundless overestimation or underestimation of reliability of designed and executed spun and vibrated concrete tubular structures [21, 23].

General constant and variable parameters of cylindrical tower bodies are represented in Table 5:

Table 5. General constant and variable parameters of cylindrical tower bodies

Parametrs of tower bodies (unit of measurement)	Code number and height of tower bodies (m)		
	CC20.0	CC22.2	CC26.4
Constant			
- outer diameter (m)	0.80	0.56	0.56
- inner diameter (m)	0.63	0.43	0.44
- flexibility	37.5	60.7	75
- grade of concrete	B45	B40	B40
- reinforcement grade	A600	A600	A600
-pretensioning level of reinforcement (σ_{sp}/R_{sk})	0.7	0.7	0.9
Variable			
Total reinforcement ratio, $\mu_{s,tot}$,%	2.5 3.5 4.5	2.7 3.7 4.7	2.7 3.7 4.7
The ratio of prestressing reinforcement to the whole area, ($A_{sp}/A_{s,tot}$),%	0;25; 50;75; 100	0;25; 50;75; 100	0;25; 50;75; 100
The ratio of vertical load moment to the	0;	0;	0;

full moment, (M_v/M),%	20; 40	20; 40	20; 40

In the numerical experiment total reinforcement ratio coefficient $\mu_{s,tot}$% for all types of tower bodies was changing withing the limits: 2.5÷4.5% (for CC=20.0 m), 2.7÷4.7% (for CC= 22.2), 2.7÷4.7% (for CC=26.4).

The ratio of prestressing reinforcement area to the whole area ($A_{sp}/A_{s,tot}$) and the ratio of vertical load moment to the full moment (M_v/M) was changing in each samples set. Bearing capacity for each tower bodies was determined in accordance with three conditions:

1 – according to strength conditions (V);

2 – according to target crack width (M_{acrc});

3 – according to limit deflection (M_f).

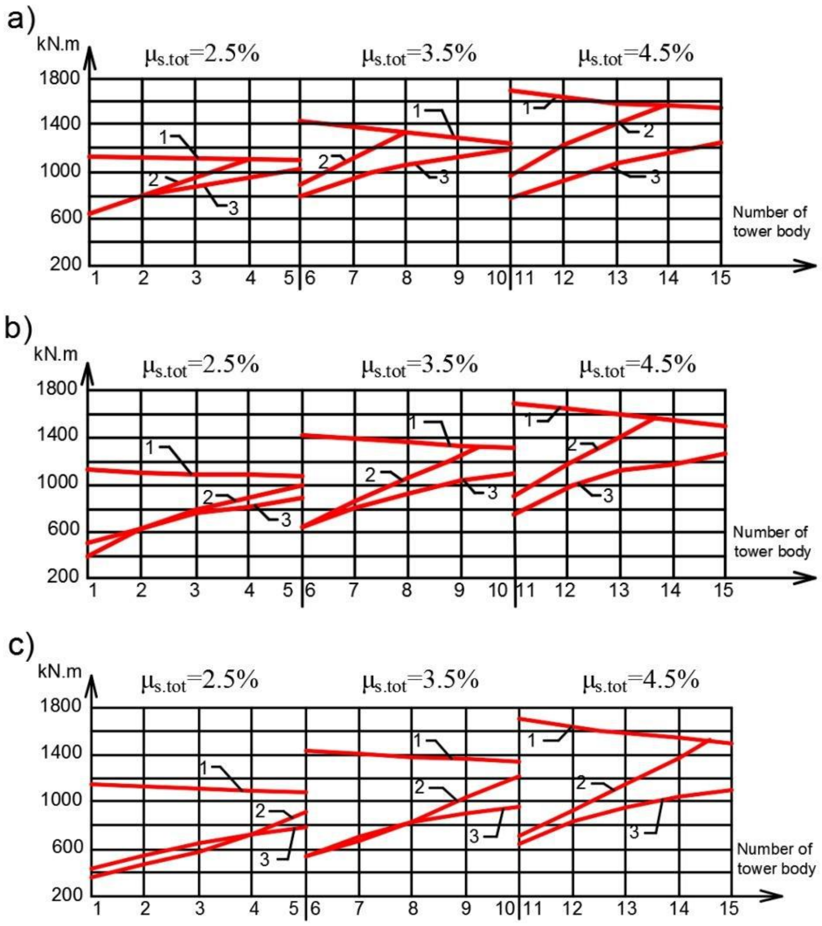

Figure 4. Changing of bearing capacity of CC20 tower bodies when
a) MV/M=0; b) MV/M=0.2; c) MV/M=0.4
1 – according to strength conditions (V); 2 – according to target crack width (Macrc);
3 – according to limit deflection (Mf).

Graphs of changing of bearing capacity of 45 CC20 tower bodies are depicted in Figure 4. Similar dependence is observed for other series of tower bodies (CC22, CC25).

Analysis of the received results demonstrates, that bearing capacity of tower bodies by uniform cross-section of concrete increases when total reinforcement ratio $\mu_{s,tot}$ increases. However, by constant $\mu_{s,tot}$, when ratio of prestressing reinforcement area A_{sp} to the whole area $A_{s,tot}$ increases, a modulated (close to linear) reduction of bearing capacity (Fig. 4) appears. This value goes up when percentage of reinforcement increases.

Similar changing of bearing capacity is observed in others series of tower bodies (CC22, CC25). The reason for this is connected with a symmetrical distribution of prestressing reinforcement according to the parameter of the annular cross section and can be explained by the earlier destruction of a compression area as a result of increasing prestressing reinforcement content.

It is interesting to compare the carrying capacity of tower bodies on the procedure adopted in the standards and methodology. The numeral comparison by different procedures is demonstrated in the Table 6.

Analysis of these results revealed, that standards inflate the bearing capacity for all the cylindrical series of tower bodies, besides the more reinforcement ratio is, the more the overestimation is.

Analysis of the graphs (Fig. 4) of changing of calculated strength of tower bodies according to crack width condition and limit deflection shows, that strength according to crack width condition for all types of tower bodies, as a rule, is higher than strength according to limit deflection.

Table 6. The numeral comparison of bearing capacity of tower bodies by different procedures

Type of tower body	Total reinforcement ratio $\mu_{s,tot}$	Bearing capacity, kN/M				(V3-V1)/V3*100,%	(V4-V2)/V4*100,%
		Author's procedure		Standards procedure			
		Asp/As.tot=0 V1	Asp/As.tot=1 V2	Asp/As.tot=0 V3	Asp/As.tot=1 V4		
CC20	2.5	1129.8	1066.1	1179.5	1115.4	4.2	4.4
	3.5	1409.9	1289.0	1555.9	1406.8	9.4	8.4
	4.5	1651.7	1468.9	1899.1	1632.1	12.8	10
CC22	2.7	416.9	388.3	456.5	429.0	8.6	9.5
	3.7	510.7	458.6	589.5	528.8	13.4	13.2
	4.7	594.2	516.2	710.4	605.7	16.3	14.7
CC26	2.7	390.7	357.7	428.0	401.2	8.7	10.8
	3.7	478.3	415.2	552.4	492.6	13.4	15.7
	4.7	556.2	463.1	665.2	561.8	16.3	17.5

Conclusion

1. Investigation of fresh-placed concrete mix properties determined a substantial segregation of its components by centrifugal consolidation. Density of the inner layer of the as-formed concrete is by 6–8 % less, but increased content of water and cement. Besides, a residual water content $(W/C)_{res}$ in the inner layer was 10–12 % greater, than in the outer layer.

2. Centrifuged concrete has a considerable heterogeneity of physical and mechanical properties along the wall of a formed product. The difference in average density of the outer and inner layer in our test varies within the limits of 4–6 %, and porosity of the inner layers was higher than porosity of the outer layers by over 18 %.

3. Actual compressive resistance varies within wide limits. In the analyzed tests on the inner surface the actual compressive resistance was 60.3 MPa in average and on the outer surface it was 74 MPa.

4. Parabolic dependence of change of centrifuged concrete strength from the wall height ratio of the product annular cross section was provided. Values of empirical coefficient were also determined.

5. Investigation of the bearing capacity of cylindrical power transmission line supports by using the provided dependence of concrete strength changing along the wall height (6) showed a satisfactory repeatability with calculation of tower bodies by standard procedure. Besides, bearing capacity of tower bodies, according to standard procedure, has an inflated value by 4.4–10 % for CC20 tower bodies with flexibility – 53.5, and for CC26 tower bodies with flexibility 75 by 10.8–15.7 %in comparison with the procedure provided by the author.

The authors are planning to continue the investigation of crack resistance and deformability of power transmission line supports with running values of the total reinforcement ratio, the relation between prestressing and nonprestressed reinforcement and different flexibility.

References

1. Schule F. Versuche und Berechnung von Eisenbetonmasten mit ringformigen Quersehnitt. *Beton und Eisen.* 1908. No. 4. Pp. 41–44.
2. Abeles P. Schleuderbetonmaste fur Bahn-Fahrleitungen. *Beton und Eisen*. 1935. No. 15. Pp. 27–30.
3. Abeles P. Mast und Mastfusse aus Schleuder beton. *Zeitschrift O.J.A.V.* 1961. No. 25/26. Pp. 13–17.
4. Mikhailov V.V. *Teoriya i praktika centrifugirovannogo napryazhenno armirovannogo betona* [Theory and practice of centrifugated prestressed reinforced concrete]. Moscow: JSC "Publishing house GosStroiizdat". 1939. (rus)
5. Korner C. Universell verwedbare Spannbetonmaste. Entwicklung eines neuen sortimentes. *Schriftenreihen der Bauforschung, Reihe stahlbeton.* 1967. №6. Pp. 108–112.
6. *Russian State Standard GOST 22687.0-85, GOST 22687.3-85.* Stojki zhelezobetonnye centrifugirovannye dlya opor vysokovoltnyx linij elektroperedach [Reinforced concrete centrifugated power bodies for high-voltage power transmission line supports]. (rus)
7. Akhvedov I.N. *Vysokoprochnyj beton* [High-strength concrete]. Moscow: GosStroiizdat 1961. 162 p. (rus)
8. Akhvedov I.N. *Zhelezobetonnye napornye centrifugirovannye truby* [Reinforced concrete pressure centrifuged pipes]. Moscow: JSC "Publishing house GosStroiizdat", 1967. (rus)
9. Dmitriev S.A. Raschet zhelezobetonnyx elementov kolcevogo secheniya [Salculation of reinforced concreteelements of annular cross section]. Construction industry. 1940. № 2. Pp. 72–74. (rus)
10. Dmitriev S.A., Batashev V.M. Prochnost i treshhinostojkost zhelezobetonnyx elementov kolcevogo secheniya [Strength and crack resistance of reinforced concreteelements of annular cross section]. *Treshchinostojkost i deformativnost obychnyh i predvaritelno napryazhennyh zhelezobetonnyh konstrukcij*. 1965. (rus)
11. Dmitriev S.A., Batashev V.M. Issledovanie zhelezobetonnyx elementov s mnogoryadnym raspolozheniem armatury [Investigation of reinforced concrete elements with multilanearrangement of reinforcement]. *Beton i zhelezobeton.* 1966. No. 5. Pp. 55–57. (rus)
12. Valduga R.R. Issledovanie raboty trubchatyx obrazcov iz centrifugirovannogo vysokoprochnogo betona pri centralnom ili vnecentrennom szhatii [Investigation of behavior of high-strength concrete pipe samples by direct and eccentrical compression]. *Issledovaniya po zhelezobetonnym konstrukciyam.* 1969. (rus)
13. Valduga R.R., Kudzys A.P. Issledovanie predelnoj szhimaemosti betona izgibaemyx prednapryazhennyx elementov kolcevogo secheniya [Investigation of maximum concrete compressibility of flexural prestressing elements of annular cross section]. *Zhelezobetonnye konstrukcii.* 1967. (rus)
14. Valduga R.R., Kudzys A.P. O prochnosti centrifugirovannogo zhelezobetona pri szhatii [On centrifugated reinforced concrete compression strength]. *Issledovaniya po zhelezobetonnym konstrukciyam.* 1966. No. 1. (rus)
15. Kudzys A.P. K voprosu opredeleniya prochnosti centrifugirovannogo betona [On the question of determination of centrifugated concrete strength]. *Stroitelstvo i arhitektura.* 1964. Vol. IV. No. 1. Pp. 28–31. (rus)
16. Kudzys A.P., Valduga R.R. O prochnosti centrifugirovannogo betona na szhatie pri izgibe [On

Литература

1. Schule F. Versuche und Berechnung von Eisenbetonmasten mit ringformigen Quersehnitt // Beton und Eisen. 1908. № 4. Pp. 41–44.
2. Abeles P. Schleuderbetonmaste fur Bahn-Fahrleitungen // Beton und Eisen. 1935. № 15. Pp. 27–30.
3. Abeles P. Mast und Mastfusse aus Schleuder beton // Zeitschrift O.J.A.V. 1961. No. 25/26. Pp. 13-17.
4. Михайлов В.В. Теория и практика центрифугированного бетона. // М. Гостройиздат. 1939.
5. Korner C. Universell verwedbare Spannbetonmaste. Entwicklung eines neuen sortimentes. // Schriftenreihen der Bauforschung, Reihe stahlbeton. 1967. №6. Pp. 108-112.
6. ГОСТ 22687.0-85, ГОСТ 22687.3-85. Стойки железобетонные центрифугированные для опор высоковольтных линий электропередач.
7. Ахвердов И.Н. Высокопрочный бетон. М.: Госстройиздат. 1961. 162 с.
8. Ахвердов И.Н. Железобетонные напорные центрифугированные трубы. М.: Госстройиздат, 1967.
9. Дмитриев С.А. Расчет железобетонных элементов кольцевого сечения. // Строительная промышленность, 1940. № 2. С. 72-74.
10. Дмитриев С.А., Баташев, В.М. Прочность и трещиностойкость железобетонных элементов кольцевого сечения // Трещиностойкость и деформативность обычных и предварительно напряженных железобетонных конструкций. 1965.
11. Дмитриев С.А., Баташев В.М. Исследование железобетонных элементов с многорядным расположением арматуры // Бетон и железобетон. 1966. №5. С. 55-57.
12. Валдуга, Р.Р. Исследование работы трубчатых образцов из центрифугированного высокопрочного бетона при центральном или внецентренном сжатии // Исследования по железобетонным конструкциям. 1969.
13. Валдуга, Р.Р., Кудзис, А.П. Исследование предельной сжимаемости бетона изгибаемых преднапряженных элементов кольцевого сечения // Железобетонные конструкции. 1967.
14. Валдуга, Р.Р., Кудзис, А.П. О прочности центрифугированного железобетона при сжатии // Исследования по железобетонным конструкциям. 1966. №. 1.
15. Кудзис А.П. К вопросу определения прочности чентрифугированного бетона // Строительство и архитектура. 1964. Вып. 4. № 1. С. 28-31.
16. Кудзис А.П., Валдуга Р.Р. О прочности центрифугированного бетона на сжатие при изгибе // Строительная механика и конструкции. Вильнюс. 1964. № 7. С. 58-60.
17. Кудзис А.П., Новиков Ю.Н. Изготовление преднапряженных железобетонных опор ЛЭП-35 // Бетон и железобетон. 1960. № 11. С. 22-25.
18. Kudzys A., Kliukas R. Reliability index design in reinforced concrete structures of annular cross sections // The 10th International Conference “Modern Building Materials, Structures and Techniques”: Selected papers. Vol. 2. Lithuania. Vilnius: Technika, 2010. Pp. 948–956.
19. Kliukas R., Kudzys A., Vadlūga R. The quasi-ductile resistance of compressed spun concrete beam-columns // Civil Engineering and Management. 2010. Vol. 16. № 2. Pp. 189–196.
20. Kudzys A., Kliukas R. Probability-based design of spun

concrete beam-columns // Journal of Civil Engineering and Management. 2013. Vol. 16. Pp. 451–461.

21. Kudzys A., Kliukas R. The limit state design of spun concrete columns using load combinations recommended by EN 1990 and ASCE/SEI 7–05 // Journal of Civil Engineering and Management. 2015. Vol. 15. Pp. 289–297.
22. Kudzys A., Kliukas R. The resistance of compressed spun concrete members reinforced by high-strength steel bars // Materials and Structures. 2008. Vol. 41. No. 2. Pp. 419-430.
23. Kliukas R. Design of spun concrete columns reinforced by high strength steelbars. Vilnius: Vilnius Gediminas Technical University, 2017.
24. Каланадзе В.С. Определение прочности центрефугированного бетона // Бетон и железобетон. 1962. No. 8. С. 54-59.
25. Михельсон Е.Э. Опоры из центрифугированного железобетона электрических воздушных линий // Тбилиси. Издательство Цодна. 1938.
26. Либерман М.Я., Сорокер В.М. Данные полузаводских опытов по центрифугированному бетону // Строительная промышленность. 1935. №2. С. 38-39.
27. Сороккер В.И. Контроль прочности бетона в центробежных трубах // Строительство и архитектура. 1958. №3. С. 37-38.
28. Сороккер В.И. Козюк М.Ф. Исследование деформативных и прочностных свойств центрифугированного бетона // Энергетическое строительство. 1968. № 9. С. 67–69.
29. Сувал Р., Щуцкий В.Л. Влияние структурной неоднородности центрифугированного бетона на его прочность в изделии. // Ростов-на-Дону. Сборник СЕВКАВНИПИАГРОПРОМ. 1995. С. 60–65.
30. Сувал Р., Щуцкий В.Л., Ткаченко, Г.А. Эксперементальные исследования неоднородности центрифугированного бетона по толщине стенки кольцевого сечения. // Ростов-на-Дону. Сборник СЕВКАВНИПИАГРОПРОМ. 1995. С. 65–70.
31. Щуцкий В.Л., Толипова Т.Д. Исследование несущей способности цилиндрических опор ЛЭП/ИД // Научное обозрение. 2014. № 10. С. 654–658.
32. Fouad H., Detwiler R. High-strength materials for spun concrete poles // Pci Journal. 2012. Pp. 27–32.
33. Volgyi I., Farkas G., Nehme S.G. Concrete strength tendency in the wall of cylindrical spun-cast concrete elements // Periodica Polytechnica: Civil Engineering. 2010. Vol. 54. № 1. Pp. 23–30.
34. Irawan C.E.A. A review of prestressed concrete pile with circular hollow section (Spun pile) // Jurnal Teknologi. 2015. Vol. 72. № 5. Pp. 115–123.

centrifugated concrete compression strength under bending]. *Stroitelnaya mekhanika i konstrukcii.* 1964. No. 7. Pp. 58–60. (rus)

17. Kudzys A.P., Novikov Y.N. Izgotovalenie prednapryazhennyx zhelezobetonnyx opor LEP-35 [Production of prestressed reinforced concrete power transmission line supports – 35]. *Beton i zhelezobeton. 1960.* No. 11. Pp. 22–25. (rus)
18. Kudzys A., Kliukas R. Reliability index design in reinforced concrete structures of annular cross sections. *The 10th International Conference "Modern Building Materials, Structures and Techniques": Selected papers.* Vol. 2. Lithuania. Vilnius: Technika, 2010. Pp. 948–956.
19. Kliukas R., Kudzys A., Vadlūga R. The quasi-ductile resistance of compressed spun concrete beam-columns. *Civil Engineering and Management*. 2010. Vol. 16. No. 2. Pp. 189–196.
20. Kudzys A., Kliukas R. Probability-based design of spun concrete beam-columns. *Journal of Civil Engineering and Management.* 2013. Vol. 16. Pp. 451–461.
21. Kudzys A., Kliukas R. The limit state design of spun concrete columns using load combinations recommended by EN 1990 and ASCE/SEI 7–05. *Journal of Civil Engineering and Management.* 2015. Vol. 15. Pp. 289–297.
22. Kudzys A., Kliukas R. The resistance of compressed spun concrete members reinforced by high-strength steel bars. *Materials and Structures.* 2008. Vol. 41. No. 2. Pp. 419-430.
23. Kliukas R. *Design of spun concrete columns reinforced by high strength steelbars.* Vilnius: Vilnius Gediminas Technical University, 2017.
24. Kalanadze V.S. Opredelenie prochnosti centrifugirovannogo betona [Determination of centrifugated concrete strength]. *Beton i zhelezobeton.* 1962. No. 8. Pp. 54–59. (rus)
25. Mikhelson E.E. *Opory iz centrifugirovannogo zhelezobetona elektricheskix vozdushnyx linij* [Electrical aerial circuit support from centrifugated reinforced concrete]. Tbilisi: Publishing house Tsodna, 1938. (rus)
26. Liberman M.Y., Soroker V.M. Dannye poluzavodskix opytov po centrifugirovannomu betonu [Data of semi-scale tests on centrifugated concrete]. *Stroitelnaya promyshlennost.* 1935. No. 2. Pp. 38–39. (rus)
27. Soroker V.I. Kontrol prochnosti betona v centrobezhnyx trubax [Concrete strength control in centrifugal pipes] *Construction and Architecture.* 1998. No. 3. Pp. 37–38. (rus)
28. Soroker V.I., Kozyuk M.F. Issledovanie deformativnyx i prochnostnyx svojstv centrifugirovannogo betona [Investigation of stress-related and strength properties of centrifugated concrete]. *Energy Construction.* 1998. No. 9, Pp. 67–69. (rus)
29. Suval R., Shchutsky V.L. Vliyanie strukturnoj neodnorodnosti centrifugirovannogo betona na ego prochnost v izdelii [Influence of centrifugated concrete structure heterogeneity on its strength in a product]. Rostov-on-Don. *Sbornik SEVKAVNIPIAGROPROM.* 1995. Pp. 60–65. (rus)
30. Suval R., Shchutsky V.L., Tkachenko G.A. Eksperementalnye issledovaniya neodnorodnosti centrifugirovannogo betona po tolshhine stenki kolcevogo secheniya [Experimental investigation of centrifugated concrete heterogeneity along annular cross section wall height]. *Rostov-on-Don: Sbornik SEVKAVNIPIAGROPROM.* 1995. Pp. 65–70. (rus)
31. Shchutsky V.L., Tolipova T.D. Issledovanie nesushhej sposobnosti cilindricheskix opor LEP/ID [Investigation of bearing capacity of cylindrical power transmission line supports]. *Scientific overview.* 2014. No. 10. Pp. 654–658. (rus)

Dedukh D.A., Schsuzkiy V.L., Kuzmenko A.A. Spun concrete properties of power transmission line supports. *Magazine of Civil Engineering.* 2017. No. 7. Pp. 37–51. doi: 10.18720/MCE.75.4.

32. Fouad H., Detwiler R. High-strength materials for spun concrete poles. *Pci Journal*. 2012. Pp. 27–32. (rus)
33. Volgyi I., Farkas G., Nehme S.G. Concrete strength tendency in the wall of cylindrical spun-cast concrete elements. *Periodica Polytechnica: Civil Engineering.* 2010 Vol. 54. No. 1. Pp. 23–30. (rus)
34. Irawan C.E.A. A review of prestressed concrete pile with circular hollow section (Spun pile). *Jurnal Teknologi*. 2015. Vol. 72. No.. 5. Pp. 115–123. (rus)

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Дедух Д.А., Щуцкий В.Л., Кузьменко А.А. Свойства центрифугированного бетона опор линий электропередач // Инженерно-строительный журнал. 2017. № 7(75). С. 37–51.

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Reliability of scour protection design near the platform "Prirazlomnaja"

Надежность конструкции защиты от размывов дна вблизи платформы «Приразломная»

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Key words: design storm; waves; wave spectrum; significant height of wave; peak period of spectrum; erosion; scour protection; rock fill grading; safety criteria

Ключевые слова: расчетный шторм; волны; спектр волнения; значительная высота волн; период пика спектра; размыв; защита от размыва; гранулометрический состав каменной наброски; критерии безопасности

Abstract. Results of experimental researches of scour protection for marine ice-resistant platform (MIRP) "Prirazlomnaja" at impact of irregular waves together with a current are presented and analyzed in the article. Experimental researches were executed taking into account maintenance of the basic similarity criterions. In particular, difference of density of sea water in natural conditions and density of fresh water in modelling conditions was taking into account at physical modelling. The various directions of irregular waves relatively MIRP and various combinations of simultaneous impact of waves and currents were considered at carrying out of experimental researches. On the basis of the natural data and the results of experimental researches safety criteria for scour protection of MIRP "Prirazlomnaja" containing 4 diagnostic parameters and their critical levels (warning and ultimate) were proposed.

Аннотация. В статье приводятся и анализируются результаты экспериментальных исследований защиты от размывов дна для морской ледостойкой платформы (МЛСП) «Приразломная» при воздействии расчетного нерегулярного волнения совместно с течением. Экспериментальные исследования проводились с учетом соблюдения основных критериев подобия. В частности, при физическом моделировании учитывалось отличие плотности морской воды в натурных условиях и пресной воды в модельных условиях. При проведении экспериментальных исследований задавалось различное направление нерегулярного волнения относительно МЛСП и рассматривались различные сочетания одновременного воздействия волн и течений. На основе натурных данных и результатов экспериментальных исследований предложены критерии безопасности для защиты от размывов дна вблизи МЛСП «Приразломная», содержащие 4 диагностических показателя и их критические уровни (предупредительный и предельный).

Introduction

MIRP "Prirazlomnaja" is located in the Pechora Sea. It was moved on the place in 2011 and it was placed in operation in 2013. The first lot of oil from MIRP "Prirazlomnaja" has been shipped in 2014. MIRP "Prirazlomnaja" is the first in the world platform for oil recovery behind polar circle. Scour protection of the bottom for MIRP "Prirazlomnaja", which was realised as two-layer rockfill, had width of the protection layer on the top of 25 m. Experimental researches and designing of scour protection for MIRP "Prirazlomnaja" were made at the usage of regular waves [1].

The considerable number of publications is devoted to researches of scour protection nearby marine hydraulic structures, for example, [2–17]. In the mentioned publications possible variants of scour protection [8, 9], questions of physical and mathematical modelling of rockfill scour protection stability [2–5, 11, 12, 16, 17], features of the hydrodynamic impacts leading to initiation of scour [2–6, 16] are discussed; results of experimental researches are resulted and summarized in [2–4, 7, 8, 13–16]. At the

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same time it is necessary to notice that experimental researches are necessarily executed by designing of scour protection nearby the bases of marine platforms of gravitational type. On the basis of these experimental researches the final design of scour protection is fulfilled.

Carrying out of experimental researches for the realized design of scour protection for MIRP "Prirazlomnaja" with width of protective layer on top of 25 m had the purpose to receive an additional substantiation of reliability of scour protection design at hydrodynamic impact from waves (irregular) and currents. Also these experimental researches had to detect necessity of strengthening of scour protection for MIRP "Prirazlomnaja".

Development of safety criteria for scour protection of MIRP "Prirazlomnaja" was defined by necessity of reliable operation and timeliness of acceptance of technical decisions for a non-admission of extreme and abnormal events.

Initial data

Initial data for carrying out of experimental researches of scour protection are presented below. In location of MIRP "Prirazlomnaja" water depth at the mean sea level is 19.4 m, the maximum elevation of sea level at storm surge and tide phenomena repeatability of 1 times in 100 years is 2.2 m. Wave characteristic at a storm repeatability of 1 times in 100 years: the considerable wave height (h_S) - h_S =5.9 m; the peak period of a wave spectrum (T_p) - T_p =15.7 s; wave spectrum is TMA spectrum with spectrum parameter γ= 2.5. Extreme current velocity repeatability of 1 times in 100 years is 1.2 m/s.

Average salinity of water in the location of MIRP "Prirazlomnaja" is 30 ‰, i.e. average density of sea water ρ_w=1030 kg/m^3.

The top soil layer of a sea-bottom (3-5m in depth) in the location of MIRP "Prirazlomnaja" consists basically of sand and sandy loam. Particles with the sizes 0.05–0.25 mm are prevail in grading of the top soil layer.

The design of scour protection for MIRP "Prirazlomnaja" is presented on fig. 1.2. The top protective layer is executed from a stone in the size 0.3-0.4 m. The bottom filter layer is executed from gravel in the size 20–70 mm. Dimensions in plain view of the MIRP "Prirazlomnaja" base are 126 x 126 m.

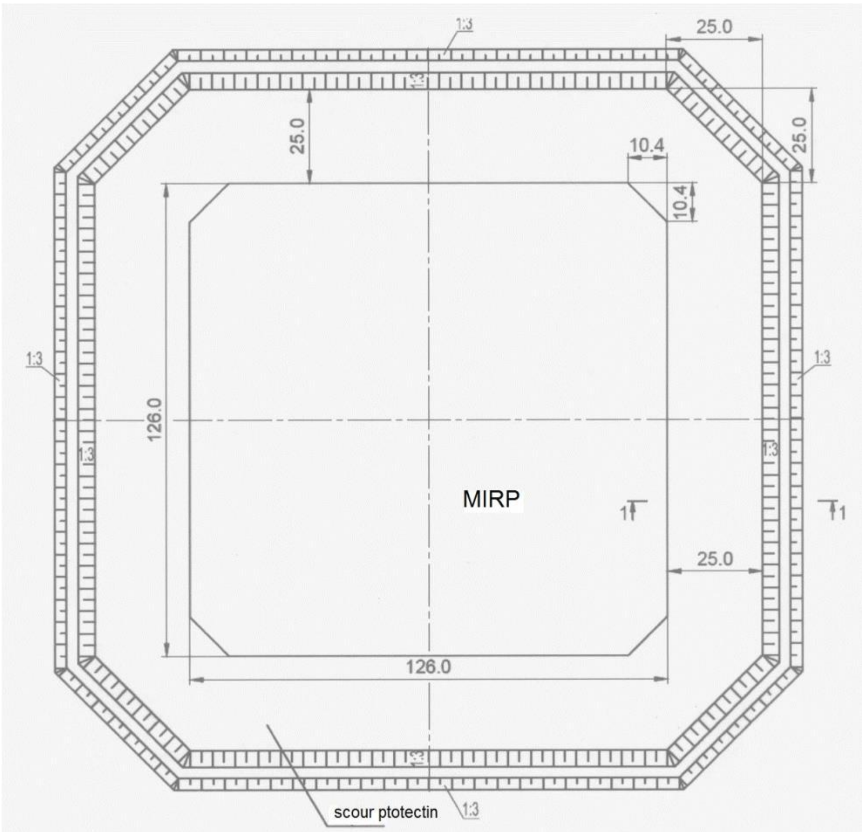

Figure 1. Plan view of scour protection for MIRP "Prirazlomnaja" (the natural sizes)

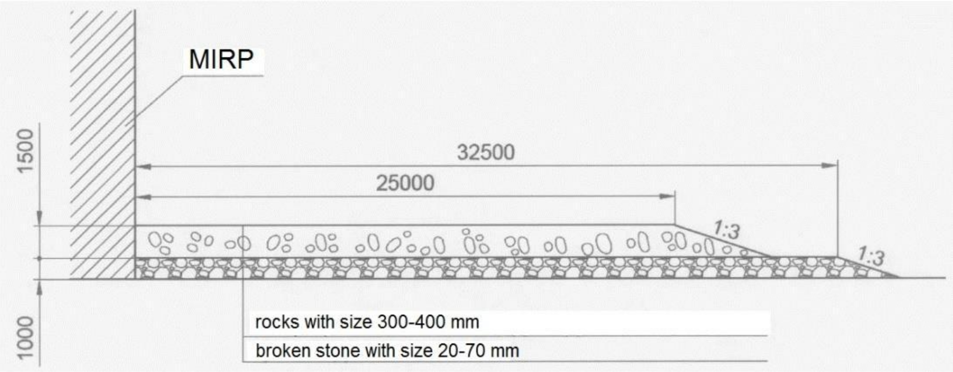

Figure 2. Cross-section of scour protection for MIRP "Prirazlomnaja" (the natural sizes)

Experimental researches

At carrying out of experimental researches the model of base of gravity type (BGT) of MIRP "Prirazlomnaja" was reproduced as geometrically similar to natural BGT. The general view of wave pool, in which researches were conducted, is shown on Figure 3. Below of foreground Figure 3 wave absorber is visible, on the far end of wave pool wavemaker of piston type is located and the hole for water delivery and creation of currents is visible, model BGT and scour protection is located in a zone of wave pool with a scouring material (a thickness of scouring material layer is 30 cm).

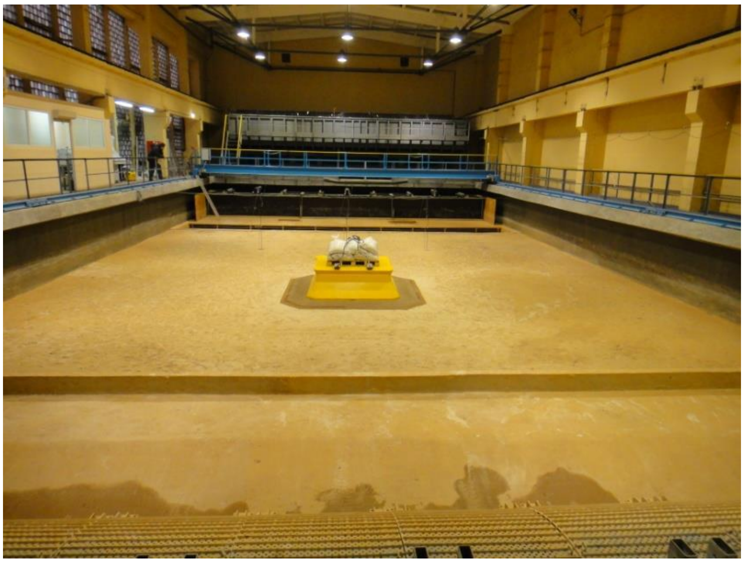

Figure 3. General view of wave basin with BGT of MIRP "Prirazlomnaja"

The basic modern requirements for experimental researches of wave impact on hydraulic facilities (HF) and their separate elements are resulted in [16]. In particular, the model scale according to [16] should satisfy a condition that the considerable height of waves on model was more than 0.05 m (as a rule, linear scale of model is greater 1:60). Herewith it is especially noticed that for the mostfull modelling of wave natural conditions it is required to reproduce irregular waves with specified spectrum. Also the special requirement is imposed to elimination of the reflected waves in modelling conditions – it is supposed reflexion no more 5 % of waves.

In the present researchers the linear scale of model has been chosen 1:60 (λ = 60). Firstly, the choice of this model scale was caused by requirements [16]. Secondly, stream constriction at impact of waves and currents in modelling conditions can lead to essential defect of hydrodynamic modeling results (the width of a modelling basin has the fixed size and for enough big sizes of model it can lead to flow defect). The width of a modelling wave basin of JSC “The B.E.Vedeneev VNIIG” was 15 m, the characteristic linear planned size of model **(**B**)** at scale 1:60 was B = 2.1 m (or 3 m if waves are directed at an angle 45^0 to a plane of one side of the platform). Thus, already at model scale 1:60 in the conditions of VNIIG wave basin the obstruction factor (the ratio of effective cross-section and full section of a modelling basin) was 0.86 (or 0.8).

At modelling on the basis of geometrical similarity, similarity of Froude numbers **(**Fr**)**

$$Fr = V/(gd)^{0.5}$$

and time scale as $\lambda^{0.5}$, scales of physical characteristics can be defined according to table 1 (V – velocity, g – acceleration of free falling, d – depth)

Table 1. Scales of model characteristics

Name	Scale	Scale for *l*=60
length	λ	60
square	λ^2	2600
volume	λ^3	216000
time	$\lambda^{0.5}$	7.75
velocity	$\lambda^{0.5}$	7.75
mass*)	λ^3	216000

*) concerning a choice of modelling stone weight of a protective layer see the following explanation.

The difference in density of water for modelling and natural conditions affects on the forces which impact at stones of a protective layer. Therefore for correct modelling it is necessary to estimate quantity of this effect and to choose correct scale of modelling stone weight of a protective layer (in modelling conditions use, as a rule, fresh water). In [16] for the account of the above described effect it is offered to have similarity on stability parameter (N_s)

$$N_S = h_S / (\Delta D_{50}), \qquad \Delta = (\rho_r - \rho_w) / \rho_w$$

Here D_{50} – average diameter of stone, ρ_r – stone density, ρ_w – water density.

Hereby, for correct modelling of hydrodynamic impact at the stones of protective layer with account of equality of stability parameter in natural and modelling conditions it is necessary to change size of stones of protective layer (D_{50}) as follows (the index “m” concerns to model, an index “p” – to nature)

$$\frac{D_{50,m}}{D_{50,p}} = \frac{1}{\lambda}\frac{\Delta_p}{\Delta_m}$$

Assuming that stone density in nature and in model conditions are equal $\rho_{r,p}=\rho_{r,m}$=2650 kg/m^3, we receive that the scale of protective layer stone size should be equal $\lambda_{r,\ diam}$= 63.

Let us notice that taking into account the chosen scale of model λ =60 it is impossible to make correct modelling of the material composing a bottom in location MIRP "Prirazlomnaja" as the size of modelling bottom material should be less 0.01 mm. Therefore sizes of scour of a bottom near scour protection for model conditions it is necessary to consider as estimating, however locations of zones with possible maximum scouring in depth of unprotected bottom can be detected. The sea-bottom in VNIIG wave basin was modelled by sand with size D_{50}=0.16 mm.

Reynolds's number $Re=V\cdot d/\nu$ (also Reynolds's numbers $Re=V\cdot h/\nu$, $Re=V\cdot B/\nu$, ν – kinematic coefficient of viscosity) for modelling conditions exceeded 0.5·10^5 that corresponds to automodelling area. Reynolds's number $Re_*=V_*\cdot D_{50}/\nu$ ($V_* = \sqrt{\tau / \rho}$ – dynamic velocity, τ – shear stress on bottom, defined according [16]) for modelling conditions exceeded 3.0·10^2 that corresponds to area of independence of critical value of Shields parameter from Re_*.

Experimental researches of scour protection for MIRP "Prirazlomnaja" were executed as sequence of tests for each of two wave directions to plane side of platform (45 ^{o} and 90 ^{o}) at design scenario (waves for a storm with repeatability 1 times in 100 years and a current with repeatability 1 times in 100 years) according to the data presented in Table 2.

Carrying out of researches in the presence of a co-ordinated current, in the absence of a current and at the account of wave transformation on opposite current allows to envelope all range of possible hydrodynamic impacts on scour protection at the design storm. Necessity of execution of the test No. 3 explains that for currents and waves with opposite directions the height of waves for conditions of MIRP "Prirazlomnaja" increases in 1.1 times (see [18]). The sequence of experiments has been chosen taking into account increase of wave impact (in the presence of a co-ordinated current the wave height was minimum, and for wave transformation on opposite current the wave height was maximum).

Table 2. Parameters of the executed tests for wave directions with angle 90º and 45º to a plane of side of MIRP "Prirazlomnaja" base

№№	Wave nature/model (*l*=60)		Current nature/model	Duration nature/model		Note
	h_s, m	T_p, s	V_{av}, m/s	test duration	total time of impact	
1a	5.9/0.098	15.7/2.03	1.2/0.155	3h/23.25min	3h/23.25min	waves+current with repeatability 1 times in 100 years co-ordinated in a direction
1b	5.9/0.098	15.7/2.03	1.2/0.155	3h/23.25min	6h/46.5min	waves+current with repeatability 1 times in 100 years co-ordinated in a direction
1c	6.5/0.108	15.7/2.03	1.2/0.155	6h/46.5min	12h/93min	waves+current with repeatability 1 times in 100 years co-ordinated in a direction
2	5.9/0.098	15.7/2.03	0	6h/46.5min	18h/139.5min	Waves (without current) with repeatability 1 times in 100 years
3	6.5/0.108	15.7/2.03	-	6h/46.5 min	24h/186.0min	Correspond to wave parameter for opposite current

Separate experiments were executed during time which correspond to duration of peak of design storm 3–6 hours of natural time (it was also useful for estimation of dynamics of bottom and scour protection deformations).

The protective layer stones of MIRP "Prirazlomnaja" were modelled by means of gravel, which grain-size distribution is presented on Figure 4. We will notice that grain-size distribution has been received by means of gravel sifting through a set gauging sieves.

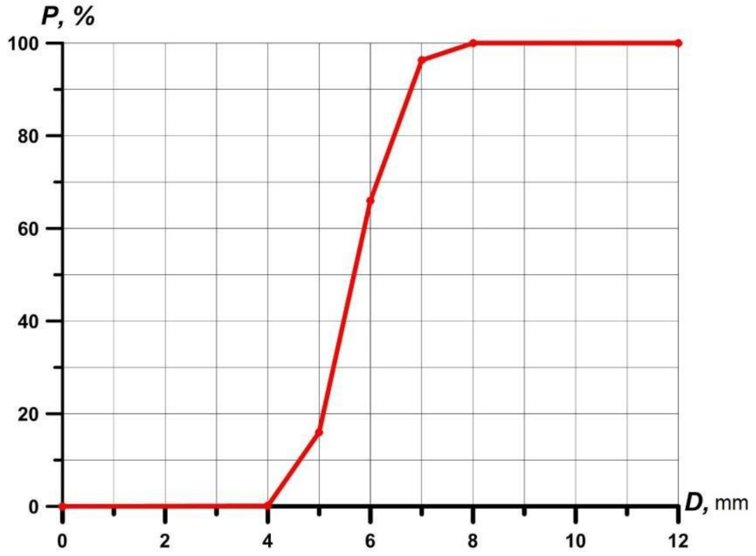

Figure 4. Model grain-size distribution of protective layer stones

Average diameter of a modelling material was D_{50} = 5.65 mm (Fig. 4) that will be co-ordinated good enough with required on criteria of similarity value 350/63 = 5.56 mm. Also it is necessary to notice that in a range *P* from 10 % to 90 % there was a material with size from 4.7 mm to 6.8 mm (that in recalculation for the natural sizes coincides well with a required range 300–400 mm).

The model filter layer was reproduced by means of sand with size 0.4–1.1 mm that will be co-ordinated good enough with required on criteria of similarity nature value 20–70 mm. The model grain-size distribution of the filter layer material is shown on Figure 5.

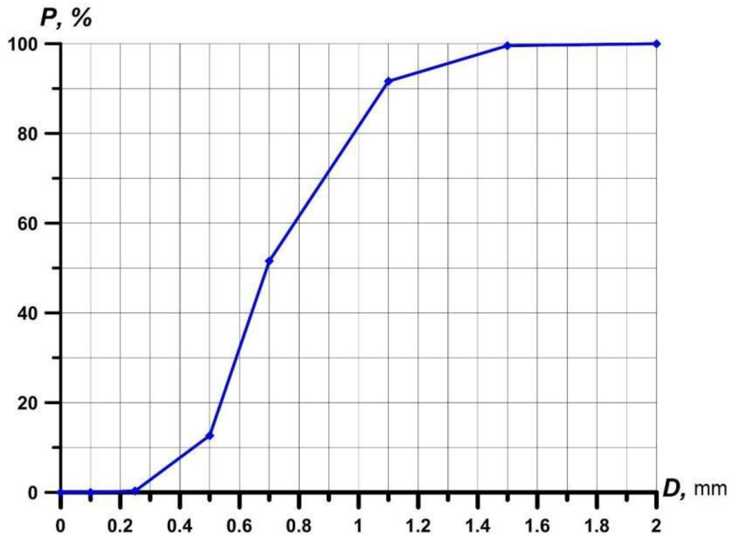

Figure 5. Model grain-size distribution of filter layer material

At calibration of current characteristics it was taken into consideration that the theoretical profile of velocity should change under the logarithmic law [19]:

$$U(z) = V_{cp} \frac{8.5 + 2.5ln(z/k)}{6.5 + 2.5ln(d/k)}$$

Here *k* – roughness parameter ($k = 2.5D_{50}$ [4]), *z* – vertical coordinate.

Results of comparison of the measured and theoretical distributions of velocity on depth for current modelling with repeatability 1 times in 100 years are shown on Figure 6, Velocity measurements were made by means of the velocity gauge MiniWater20Micro.

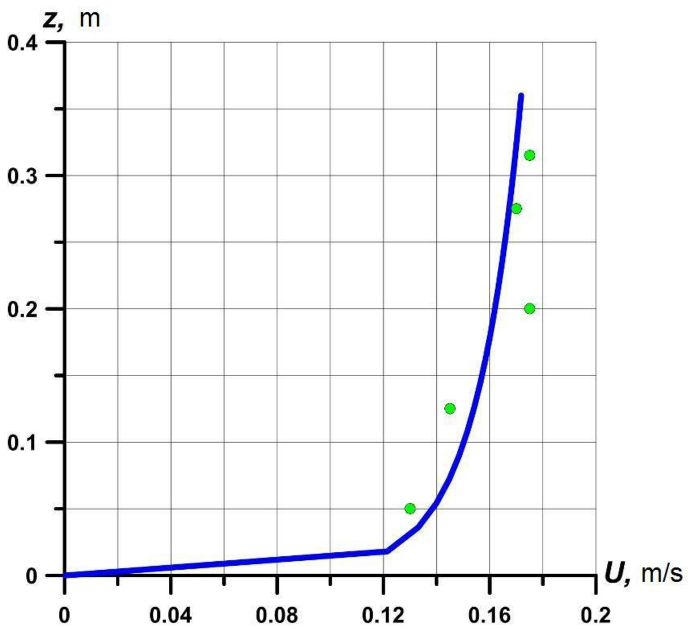

Figure 6. Distribution of current velocity on depth (repeatability – 1 times in 100 years) for model conditions (—— – theoretical profile of velocity, ° – measured value of velocity)

Vertical distribution of velocity for chosen location of model MIRP "Prirazlomnaja" will be co-ordinated well enough with theoretical profile for the steady turbulent flows (Fig.6).

At calibration of wave conditions the possible location of model MIRP "Prirazlomnaja" was defined so to receive demanded wave characteristics in this place. As example, comparison of the required and measured wave spectra for tests 1,2 (table 2) is shown on Figure 7.

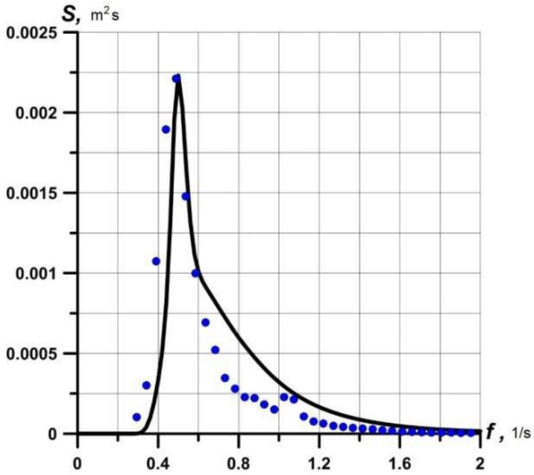

Figure 7. Wave spectrum for conditions of tests 1,2 with repeatability 1 times in 100 years (——— – required distribution, ● – measured value)

Results and Discussion

The first series of experiments was executed for wave direction with angle 90° to a plane side of BGT. Certain moments and results of the executed experimental researches are shown on Figures 8–12. Measurements of scour protection and bottom levels around model BGT were made by means of Trimble M3 total station (the error of measurements did not exceed 1 mm). Data on dynamics of scour protection and bottom levels are presented on Figures 13–15.

Figure 8.View on BGT model and scour protection before the beginning of experimental researches for wave direction with angle 90° to a plane side of BGT

Figure 9. Interaction of waves with BGT during execution of researches for wave direction with angle 90° to a plane side of BGT "Prirazlomnaja" (view from windward side)

Figure 10. View on BGT model and scour protection (from leeward side) after after execution of tests 1a, 1b, 1c, 2, 3 for wave direction with angle 90º to a plane side of BGT "Prirazlomnaja"

Figure 11. View on BGT model and scour protection (from windward side) after after execution of tests 1a, 1b, 1c, 2, 3 for wave direction with angle 90º to a plane side of BGT "Prirazlomnaja"

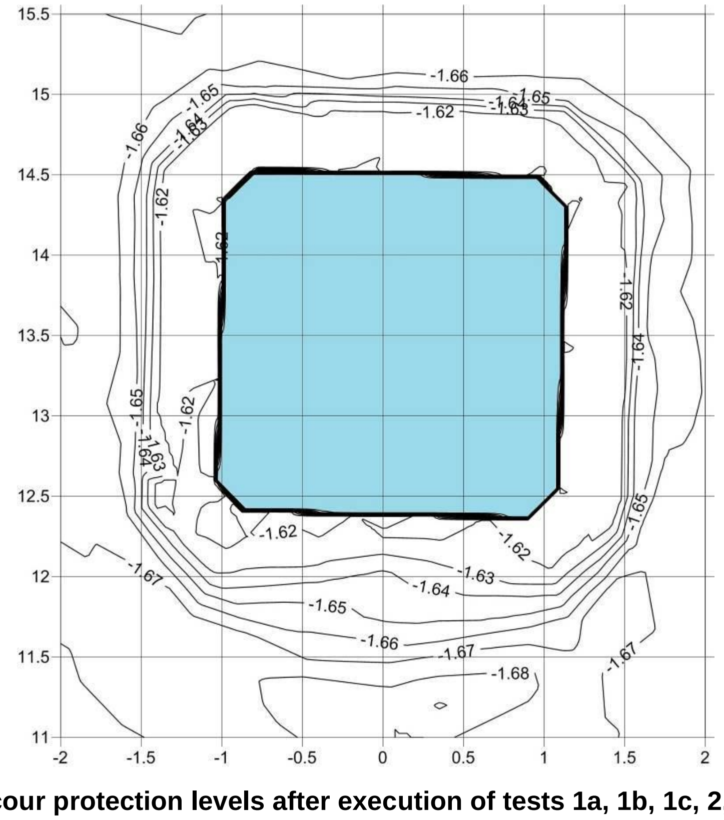

Figure 12. Plan of scour protection levels after execution of tests 1a, 1b, 1c, 2, 3 for wave direction with angle 90º to a plane side of BGT "Prirazlomnaja" (level – 1.62 m corresponds to the top level of scour protection before researches)

The geometry of the central section of windward scour protection face after execution of corresponding tests (Table 2) is shown on Figure 13.

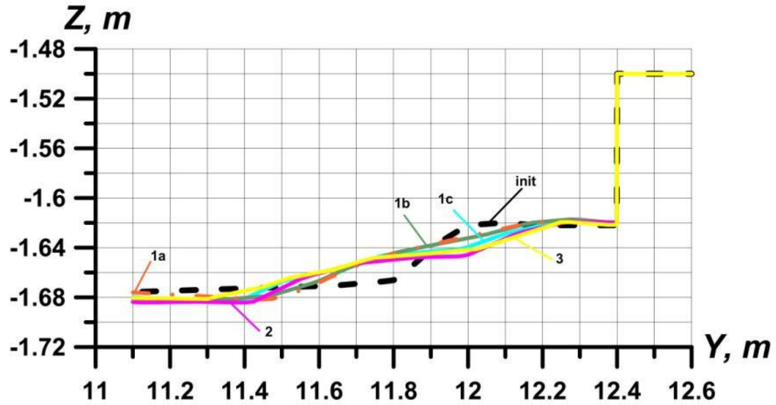

Figure 13. Geometry of the central section of windward scour protection face after execution of tests 1a, 1b, 1c, 2, 3 for wave direction with angle 90º to a plane side of BGT "Prirazlomnaja" (— — — – initial geometry).

Let us notice that the leeward face geometry, and also lateral faces of scour protection after execution of 5 tests has not changed practically. On Figures 14, 15, for example, the geometries of the central sections of left side face and leeward face of scour protection are shown after execution of tests 1a, 1b, 1c, 2, 3.

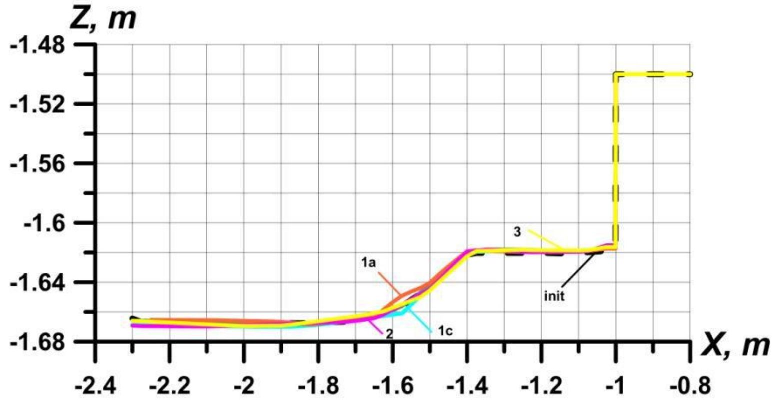

Figure 14. Geometry of the central section of left side scour protection face after execution of tests 1a, 1b, 1c, 2, 3 for wave direction with angle 90º to a plane side of BGT "Prirazlomnaja" (— — — – initial geometry)

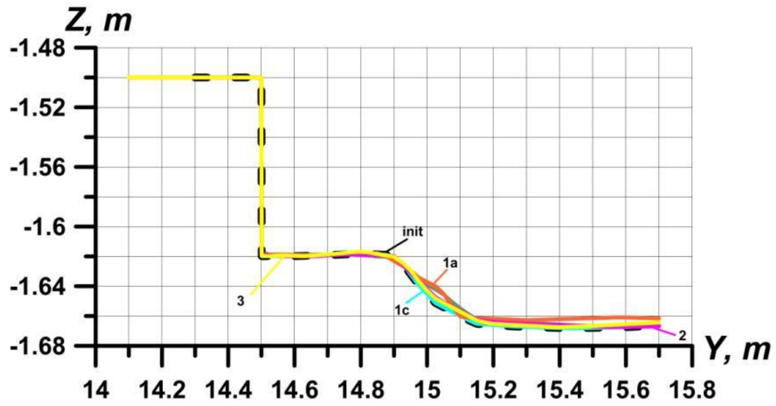

Figure 15. Geometry of the central section of leeward scour protection face after execution of tests 1a, 1b, 1c, 2, 3 for wave direction with angle 90º to a plane side of BGT "Prirazlomnaja" (— — — - initial geometry)

Small deformations of a protective layer were observed near to angular points of interface of windward face with the cutaway corners of BGT. The maximum vertical deformation (scour) of protective layer in these local zones near to angular points after execution of all 5 tests did not exceed 1 sm and was not critical.

Let us notice that after 24 hours of impact of a storm with repeatability of 1 times in 100 years scour protection with width of a protective layer 25 m (on top) has kept the function of protection of the foundation from scour for a case of wave direction with angle 90º to a plane side of BGT. As follows from Figures 13–15, deformations of scour protection induced of waves for a design storm in any combination with a current are stabilized after ~18–24 hours. Herewith adjacent part of protective layer on distance 10–12 m (the natural sizes) from BGT surface was not deformed except vicinity of angular points of BGT where vertical size of scour can reach 0.6 m (in natural conditions).

The second series of experiments was executed for wave direction with angle 45º to a plane side of BGT. Certain moments and results of the executed experimental researches are shown on Figures 16, 17.

Figure 16. Interaction of waves with BGT during execution of researches for wave direction with angle 45º to a plane side of BGT "Prirazlomnaja" (view from windward side)

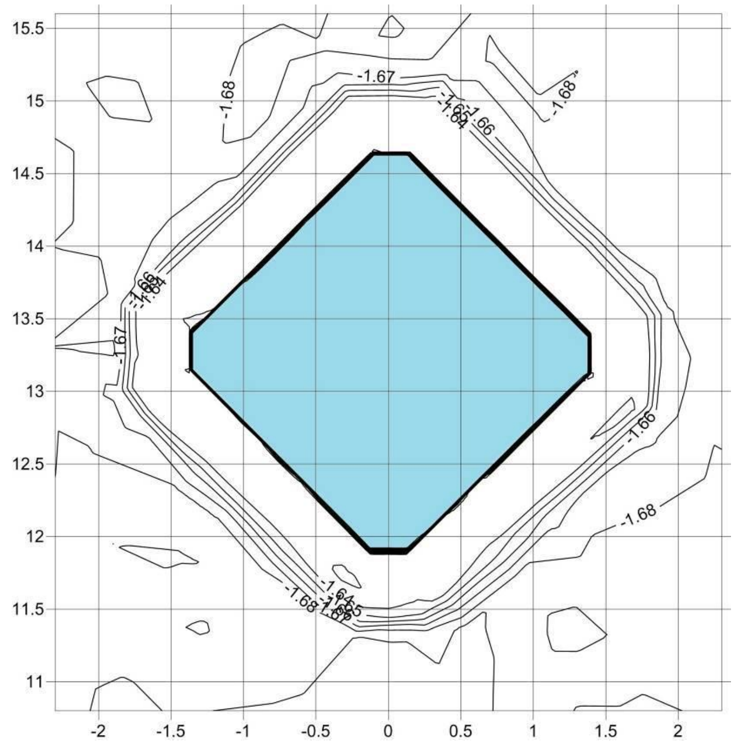

Figure 17. Plan of scour protection levels after execution of tests 1a, 1b, 1c, 2, 3 for wave direction with angle 45º to a plane side of BGT "Prirazlomnaja" (level – 1.64 m corresponds to the top level of scour protection before researches)

Let us notice that after 24 hours of impact of a storm with repeatability of 1 times in 100 years scour protection with width of a protective layer 25 m (on top) has kept the function of protection of the foundation from scour for a case of wave direction with angle 45º to a plane side of BGT. Scour protection surface practically was not deformed except vicinity of angular points of BGT where vertical size of scour can reach 0.6 m (in natural conditions).

As it was noted in [2], at present time there are no empirical relations allowing reliably to predict size of bottom scours for marine hydraulic structures of the big sizes and any form. Even for cylindrical piles with big diameters for values of Keulegan-Carpenter numbers (*KC*) KC <1 prognosis of a bottom scours is not possible [2]. In experimental researches of scour protection of the bottom for MIRP "Prirazlomnaja" the Keulegan-Carpenter number was equal $KC \approx 0{,}2$.

The obtained experimental results about scours of unprotected bottom near to rockfill for MIRP "Prirazlomnaja" have been co-ordinated in whole with the data [1]. Scours of unprotected bottom in the presented experiments were about 1.2–1.8 m, in [1] scours of unprotected bottom (scour protection had width on top of 20 m) were 1.5–2.5 m. The stabilization time of a protective layer deformations under impacts of design waves and currents in the present experimental researches was some more than in [1]. Unfortunately, in [1] it has not been given the concrete data about protective layer deformations, therefore it is not possible to spend comparison of this data at impacts of irregular (in the present experiments) and regular waves (in [1]).

Experimental results on protective layer deformations under design impacts of waves and currents show that the realized scour protection provides reliable protection of a bottom. These results should be considered in development of safety criteria of scour protection for MIRP "Prirazlomnaja".

Safety criteria

In the present domestic and foreign normative literature [2, 16, 20–24] do not contain specific requirements to setting of safety criteria for scour protection near marine stationary platforms. In [16] (chapter 10) it explains by means of the big variety of natural conditions and a variety of platform base designs. The general recommendations for setting of safety criteria assume the presence of the natural data on a situation of scour protection and presence of experimental data on behaviour of scour protection under design impacts. As an example the natural data for one of scour protection cross profile located near to the center of North face of MIRP "Prirazlomnaja" are shown on Figure 18. Comparing the data on the monitoring, which was spent in 2012-2016, and results of experimental researches of scour protection it is possible to notice that character of scour protection deformations at wave impacts in natural and laboratory conditions are coordinated in whole. It is observed flattening of a scour protection slope, the basic deformations (scours) of a protective layer are observed in the central part of the scour protection faces, significant scours in a zone adjoining to BGT on distance of 10–12 m is not observed. Such agreement of the natural and laboratory data, which were received independently, indicates that monitoring and experimental researches were executed at good level.

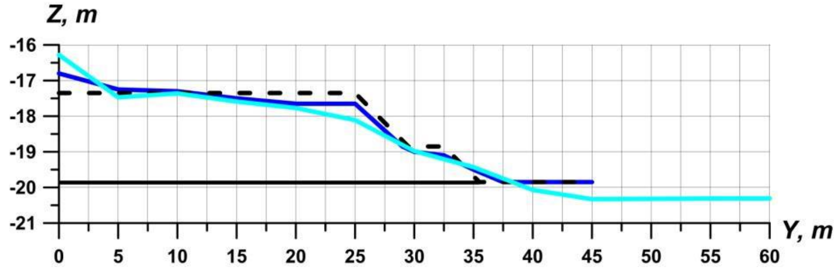

Figure 18. Natural data on the changes of geometry of scour protection section located near to the center of North face of MIRP "Prirazlomnaja" - - - - – a design profile, ——— – a profile after building of scour protection in 2011, ——— – a profile in conditions of 2016).

According to results of experimental researches different parts of scour protection were deformed differently (the central part of scour protection faces can be deformed much more strongly, than that which were located more close to edge, see Figure12). Therefore it is reasonably to divide each of lateral scour protection faces on 4 segments, and also to consider in addition segments of scour protection, adjoining to the cutaway corners of BGT. In total, thus, it will be possible to consider 20 segments of scour protection. Safety criteria will be applied to each segment separately and the total condition of scour protection will be estimated on a segment with the worst condition.

As the diagnostic indicators, characterizing a condition of scour protection of MIRP "Prirazlomnaja", it is proposed to use:

- the area of scour (level lowering) of a protective layer, exceeding the specified critical value from design level, in a zone adjoining to the base of a platform on distance of 12 m (D1);

- depth (in meters) local scour (level lowering) of a protective layer from design level (D2);

- a steepness of scour protection slopes (D3);

- depth of local scours of unprotected bottom near to scour protection on distance 10м from outside of scour protection (D4).

For diagnostic indicator D1 the criterion of condition K1 (warning level) is offered from a requirement that on distance of 12 m from the BGT of MIRP "Prirazlomnaja" the total area where scour exceeds average stone size (D_{50}= 0.35 m), makes more than 50 % from the area of a considered zone for each segment. The criterion of condition K2 (ultimate level) for diagnostic indicator D1 is offered from a requirement that on distance of 12 m from the BGT of MIRP "Prirazlomnaja" the total area where scour exceeds half of protective layer thickness (0.75 m), makes more than 50 % from the area of a considered zone for each segment. We will notice that changes of diagnostic indicator D1 can be connected with protective layer scours as a result of wave impacts and also with subsidence of a protective layer as a result of washing away of filter layer particles.

For diagnostic indicator D2 the criterion of condition K1 (warning level) is offered from a requirement that on distance of 12 m from the BGT of MIRP "Prirazlomnaja" depth of local scours of a protective layer make no more than two average diameters of a stone (0.7 m) for each considered segment. The criterion of condition K2 (ultimate level) for diagnostic indicator D2 is offered from a requirement that on distance of 12 m from the BGT of MIRP "Prirazlomnaja" depth of local scours of a protective layer make no more than three average stone diameters (1.05 m) for each considered segment. We will notice that introduction of diagnostic indicator D2 is connected with possible ice impacts and also with influences on scour protection from falling subjects which can affect on its ability to protect of a sea-bottom near platform from scours.

For diagnostic indicator D3 the criterion of condition K1 (warning level) is offered from a requirement that the steepness of a slope does not exceed the design level (1 to 3) for each considered segment. The criterion of condition K2 (ultimate level) for diagnostic indicator D3 is offered from a requirement that the steepness of a slope does not exceed a natural steepness of a stone slope under water (1 to 1.2) for each considered segment. We will notice that introduction of diagnostic indicator D3 is connected with possible ice influences and also with the possible underwater landslips because of washing away of filter layer particles and particles of a bottom soil.

For diagnostic indicator D4 the criterion of condition K1 (warning level) is offered from a requirement that depth of local scour of unprotected bottom on distance of 10 m from outside of scour protection does not exceed 2.0 m for each considered segment. The criterion of condition K2 (ultimate level) for diagnostic indicator D4 is offered from a requirement that depth of local scour of unprotected bottom on distance of 10 m from outside of scour protection does not exceed 6.0 m for each considered segment. We will notice that introduction of diagnostic indicator D4 is connected with possible scours of unprotected bottom which can lead to transformation of scour protection slopes. Choice of level K1 equal of 2.0 m was made because of scour depth of unprotected bottom did not exceed 2.0 m in experimental researches and in natural observations. Choice of level K2 equal of 6.0 m was made because of such scour depth of unprotected bottom near scour protection for design steepness of slope begins to affect on area located on distance of 12 m from the BGT of MIRP "Prirazlomnaja".

The total condition of scour protection for MIRP "Prirazlomnaja" is offered to estimate according [25]. If diagnostic indicators D1-D4 do not exceed criterion K1 for all 20 segments then total condition of scour protection is estimated as normal. If at least one of diagnostic indicators D1-D4 exceed criterion K1 for any segment and do not exceed criterion K2 total condition of scour protection is estimated as potentially dangerous. If at least one of diagnostic indicators D1-D4 exceed criterion K2 for any segment total condition of scour protection is estimated as abnormal.

At present time according results of natural observations in 2016 the total condition of scour protection for MIRP "Prirazlomnaja" is normal.

Conclusions

1. Experimental researches of scour protection for MIRP "Prirazlomnaja" were executed at design hydrodynamic impact from waves (irregular) and currents. It was shown that at impact of design storm with various combinations to a design current scour protection with width of a protective layer on top 25 m and stone sizes of a protective layer 0.3-0.4 m provides reliable protection of a bottom against scour near BGT at various directions of waves to MIRP "Prirazlomnaja".

2. Safety criteria for scour protection of MIRP "Prirazlomnaja" based on results of experimental researches and natural data on a state of scour protection were proposed.

3. By the results of natural observations in 2016 the technical condition of scour protection for MIRP "Prirazlomnaja" according to the accepted criteria of safety is estimated as normal.

References

1. Devis M.Kh., Mishchenko S.M. Eksperimentalnoye issledovaniye mestnykh razmyvov dna u osnovaniya morskikh gidrotekhnicheskikh sooruzheniy [Experimental study of local erosion of the bottom at the base of marine hydraulic structures]. *Izvestiya VNIIG.* 2000. Vol. 236. Pp. 140–151.
2. Sumer B.M., Fredsoe J. *The Mechanics of Scour in the Marine Environment.* Singapore. Word Scientific. 2002. 536 p.
3. Whitehouse R. *Scour at Marine Structure.* Thomas Telford. London. 1998. 216 p.
4. Hoffmans G., Verheij H. *Scour manual.* Balkema. Netherland. 1997. 224 p.
5. Annandale G. *Scour Technology. Mechanics and Engineering Practice.* Mc Grow-Hill. 2006. 368 p.
6. *EM 1110-2-1100.* The coastal engineering manual. US Army Corps. of Engineers. 2002.
7. May R., Ackers J., Kirby A. *Manual on scour at bridges and other hydraulic structure.* CIRIA. London. 2002. 225 p.
8. Khalfin I.Sh. *Vozdeystviye voln na morskiye neftegazo-promyslovyye sooruzheniya.* M.: Nedra. 1990. 311 c.
9. Belyaev N.D., Lebedev V.V., Nudner I.S., Semenov K.K., Shchemelinin D.I. Selection of protective measures against scouring at the foundations of offshore gravity platforms. *Magazine of Civil Engineering.* 2015. No. 3(55). Pp. 79–88. (rus)
10. Whitehouse R., Harris J., Sutherland J., Rees J. The nature of scour development and scour protection of offshore windfarm foundations // Marine Pollution Bulletin. 2011. Vol. 62. No. 1. Pp. 73–88.
11. Rudolph D., Raaijmakers T.C., Sonneville B. Challenges and recent advances in offshore scour modeling. *12th International Conference The JACK-UP PLATFORM Design, Construction & Operations.* London, UK, 2009.
12. Sumer B.M. Mathematical modeling of scour: A review. *J.of Hydraulis research.* 2007. Vol. 45. No. 6. Pp. 723–735.
13. Sumer B.M., Nielsen A.W. Sinking failure of scour protection at offshore wind turbine foundation. *Proceedings of the ICE-Energy. ICE.* 2013. Vol. 166(4). Pp. 170–188.
14. Vos L.D, De Rouck J., Troch P., Frigaard P. Empirical design of scour protections around monopile foundations: Part 1: Static approach. *Coastal Engineering.* 2011. Vol. 58. No. 6. Pp. 540–553.
15. Vos L.D, De Rouck J., Troch P., Frigaard P. Empirical design of scour protections around monopile foundations: Part 2: Dynamic approach. *Coastal Engineering.* 2012. Vol. 60. Pp. 286–298.
16. *CIRIA, CUR, CETMEF.* The Rock Manual/ The use of rock in hydraulic engineering (2-nd edition). C683, CIRIA, London, 2007. 1267 p.
17. Semenov K.K., Lebedev V.V., Nudner I.S., Geidarov N.A., Ivanov K.S., Zagharov Yu.N., Belyaev N.D., Mishina A.V., Schemelinin L.G. Impacts of waves and currents on the soil near gravity-tipe offshore platform foundation. *Proceedings*

Литература

1. Дэвис М.Х., Мищенко С.М. Экспериментальное исследование местных размывов дна у основания морских гидротехнических сооружений// Известия ВНИИГ. 2000. Т. 236. С. 140–151.
2. Sumer B.M., Fredsoe J. The Mechanics of Scour in the Marine Environment. Singapore. Word Scientific. 2002. 536 p.
3. Whitehouse R. Scour at Marine Structure. Thomas Telford. London. 1998. 216 p.
4. Hoffmans G., Verheij H. Scour manual. Balkema. Netherland. 1997. 224 p.
5. Annandale G. Scour Technology. Mechanics and Engineering Practice. Mc Grow-Hill. 2006. 368 p.
6. EM 1110-2-1100. The coastal engineering manual. US Army Corps. of Engineers. 2002.
7. May R., Ackers J., Kirby A. Manual on scour at bridges and other hydraulic structure. 2002. CIRIA. London. 225 p.
8. Халфин И.Ш. Воздействие волн на морские нефтегазо-промысловые сооружения. М.: Недра. 1990. 311 с.
9. Беляев Н.Д., Лебедев В.В., Нуднер И.С., Семенов К.К., Щемелинин Д.И. Выбор мероприятий по защите от размыва оснований гравитационных платформ для освоения шельфа // Инженерно строительный журнал. 2015. № 3. С. 79–88.
10. Whitehouse R., Harris J., Sutherland J., Rees J. The nature of scour development and scour protection of offshore windfarm foundations // Marine Pollution Bulletin. 2011. Vol. 62. № 1. Pp. 73–88.
11. Rudolph D., Raaijmakers T.C., Sonneville B. Challenges and recent advances in offshore scour modeling // 12th International Conference The JACK-UP PLATFORM Design, Construction & Operations. London, UK, 2009.
12. Sumer B.M. Mathematical modeling of scour: A review // J.of Hydraulis research. 2007. Vol. 45. № 6. Pp. 723–735.
13. Sumer B.M., Nielsen A.W. Sinking failure of scour protection at offshore wind turbine foundation // Proceedings of the ICE-Energy. ICE. 2013. Vol. 166(4). Pp. 170–188.
14. Vos L.D, De Rouck J., Troch P., Frigaard P. Empirical design of scour protections around monopile foundations: Part 1: Static approach // Coastal Engineering, 2011. Vol. 58. № 6. Pp. 540–553.
15. Vos L.D, De Rouck J., Troch P., Frigaard P. Empirical design of scour protections around monopile foundations: Part 2: Dynamic approach // Coastal Engineering. 2012. Vol. 60. Pp. 286–298.
16. CIRIA, CUR, CETMEF. The Rock Manual/ The use of rock in hydraulic engineering (2-nd edition). C683, CIRIA, London, 2007. 1267 p.
17. Semenov K.K., Lebedev V.V., Nudner I.S., Geidarov N.A., Ivanov K.S., Zagharov Yu.N., Belyaev N.D., Mishina A.V., Schemelinin L.G. Impacts of waves and currents on the soil near gravity-tipe offshore platform foundation // Proceedings of the 25th International Offshore and Polar

of the 25th International Offshore and Polar Engineering Conference. USA. 2015. Vol. 2. Pp. 807–814.

18. Gurgenidze A.T., Trapeznikov YU.A., Rumyantseva S.A. et al. *Teoreticheskiye osnovy i metody rascheta vetrovogo volneniya* [Theoretical bases and methods for calculating wind waves]. Leningrad: Gidrometeoizdat, 1988. 263 p.
19. Loytsyanskiy L.G. *Mekhanika zhidkosti i gaza. 3-ye Izdaniye* [Mechanics of liquid and gas]. Moscow: Nauka, 1970. 904 p.
20. *ISO 19906:2010.* Petroleum and natural gas industries — Arctic offshore structures. Brussels. 2010. 465 p.
21. *Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms – Working Stress Design.* American Petroleum Institute. 2000.
22. *CAN/CSA-S471-92.* General Requirements, Design Criteria, the Enviroment, and Loads/ Forming Part of the Code for the Design, Construction, and Installation of fixed Offshore Structures.
23. Russian State Standard GOST R 54483-2011. Platformy morskiye dlya neftegazodobychi. Obshchiye trebovaniya [Marine platforms for oil and gas production. General requirements]. Moscow, 2012. 26 p.
24. Russian Set of Rules SP 23.13330.2011. Osnovaniya gidrotekhnicheskikh sooruzheniy [Foundations of hydraulic structures]. Moscow, 2011. 109 p.
25. *RD 153-34.2-21.342-00. Metodika opredelenia kriteriev bezopasnosti gidrotehnicheskih sooruzheniy.* [Guidance document RD 153-34.2-21.342-00. Method for determination of safety criteria of hydraulic structures] Moscow. 2000. 16 p. (rus).

Engineering Conference. USA. 2015. Vol. 2. Pp. 807–814.

18. Гургенидзе А.Т., Трапезников Ю.А., Румянцева С.А. и др. Теоретические основы и методы расчета ветрового волнения. Под ред. Давидана И.Н.. Л.: Гидрометеоиздат, 1988. 263 с.
19. Лойцянский Л.Г. Механика жидкости и газа. 3-е Издание. М.: Наука, 1970. 904 с.
20. ISO 19906:2010. Petroleum and natural gas industries — Arctic offshore structures. Brussels. 2010. 465 p.
21. Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms – Working Stress Design. American Petroleum Institute. 2000.
22. CAN/CSA-S471-92. General Requirements, Design Criteria, the Enviroment, and Loads/ Forming Part of the Code for the Design, Construction, and Installation of fixed Offshore Structures.
23. ГОСТ Р 54483-2011. Платформы морские для нефтегазодобычи. Общие требования. М., 2012. 26 с.
24. СП 23.13330.2011. Основания гидротехнических сооружений. Актуализированная редакция СНиП 2.02.02-85. М., 2011. 109 с.
25. РД 153-34.2-21.342-00. Методика определения критериев безопасности гидротехнических сооружений. М., 2000. 16 с.

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The effect of particulate reinforcement on strength and deformation characteristics of fine-grained concrete

Влияние дисперсного армирования на прочностные и деформативые характеристики мелкозернистого бетона

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Key words: fiber concrete, disperse reinforcement; fiber; strength and deformation properties of concrete; disperse-reinforced fine-grained concrete; the density at normal humidity conditions; the limit of tensile strength in bending; tensile strength under compress

Ключевые слова: фибробетон; дисперсное армирование; волокна; прочностные и деформационные свойства бетона; дисперсно-армированный мелкозернистый бетон; плотность в нормальных влажностных условиях; предел прочности на растяжение при изгибе; предел прочности при сжатии

Abstract. One of the options to improve the reliability and increase the service life of concrete structures can be a reinforcement of the total volume of fine-grained concrete with different fiber types. The article presents the results of complex researches on studying of influence of parameters of the dispersed reinforcement (fiber, length and type of fiber, dosage by volume) and the material of the fibers of strength of disperse-reinforced fine-grained concrete on a stretching at a bend, the estimation of efficiency and accountability in the calculation of building structures. The article considers the possibility of using industrial materials. It is proven that rougher texture leads to a larger force of adhesion between the particles of filler and the cement matrix. In addition, the large surface area of angular aggregate facilitates the development of greater traction. The article considers the issues of application of steel fibers to disperse reinforcement of fine-grained concrete. As a binder we used fine ground cement and the binder with low water demand. Their physic-mechanical characteristics of binders were studied. From the results of experiments it was found that grinding cement with plasticizing additive "Polyplast SP-1" in the amount of 0.6 % by weight of cement is more intense. This shows that in addition to the plasticizing action, it has an intensifying effect at grinding; this is due to the wedging action of most additives. It is also seen that the kinetics of grinding TMC and VNV on attrition crushing granite similar and previously studied industrial raw materials. As reinforcing material steel wave fiber were used. For increasing the strength and deformation characteristics were developed by the compositions of fine-grained fiber-reinforced concrete on technogenic raw materials (screening instruments) and composite binders with the use of nanodispersed powders (NDP). The use of composite binder and high-density packing of grains of filler significantly increases the strength characteristics. Optimal selection of filler allowed to receive on technogenic Sands of Kursk magnetic anomaly of the fiber-reinforced concrete with strength limit under compression – 160.2 MPa, at a bend of 31.2 MPa.

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Аннотация. Одним из вариантов повышения надежности и увеличения сроков эксплуатации железобетонных конструкций может быть армирование всего объема мелкозернистого бетона с помощью различных видов фибры. В статье представлены результаты комплексных исследований по изучению влияния параметров дисперсного армирования (длина волокон и вид фибры, дозировка по объему) и материала волокон на прочность дисперсно-армированного мелкозернистого бетона на растяжение при изгибе, дана оценка эффективности такого метода и возможность учета при расчете строительных конструкций. В статье рассмотрена возможность применения техногенного сырья. Доказано, что более грубая текстура приводит к большей силе сцепления между частицами заполнителя и цементной матрицей. Кроме того, большая площадь поверхности угловатого заполнителя дает возможность развития большей силы сцепления. В статье рассмотрены вопросы применения стальной фибры для дисперсного армирования мелкозернистых бетонов. В качестве вяжущего использовался тонкомолотый цемент и вяжущее низкой водопотребности. Были изучены их физико-механические характеристики вяжущих. Из результатов экспериментов установлено, что помол цемента с пластифицирующей добавкой «Полипласт СП-1» в количестве 0,6% от массы цемента проходит интенсивнее. Это свидетельствует, что помимо пластифицирующего действия, она обладает и интенсифицирующим действием при помоле, это объясняется расклинивающим действием самой добавки. Также видно, что кинетика размалываемости ТМЦ и ВНВ на отсеве дробления гранита аналогична, как и на ранее изученном техногенном сырье. В качестве армирующего материала использовалась стальная волновая фибра. Для увеличения прочностных и деформативных характеристик были разработаны составы мелкозернистого фибробетона на техногенном сырье (отсев КВП) и композиционных вяжущих с применением нанодисперсного порошка (НДП). Установлено, что применение композиционных вяжущий и высокоплотной упаковки зерен заполнителя значительно повышают прочностные показатели. Оптимальный подбор заполнителя позволил получить на техногенных песках КМА фибробетон с пределом прочности при сжатии – 160,2 МПа, при изгибе 31,2 МПа. Ключевые слова: фибробетон, дисперсное армирование, волокна, прочностные и деформационные свойства бетона, дисперсно-армированный мелкозернистый бетон, плотность в нормальных влажностных условиях, предел прочности на растяжение при изгибе, предел прочности при сжатии.

Introduction

In recent years, many developed countries increases production of high quality concretes for various purposes, i.e. concrete, performance characteristics which meet or exceed the highest quality criteria, regulated by standards of different countries [1].

The main building material that provides high load capacity and long life span is concrete. The development of modern concrete Sciences is aimed, on the one hand, at improving physical and mechanical characteristics of concrete, and on the other hand, the reduction of costs in production and operation of concrete and reinforced concrete structures. For the implementation of these tasks requires the use of innovative technologies in concrete production.

The use of traditional methods of reinforcement of concrete steel flat or three-dimensional frames leads to heterogeneity of structure, the formation of voids in the concrete, deterioration of the deformation characteristics of structures [2–4], one of the effective ways of the directed formation cementogenesis systems (mortars and concretes) is volumetric dispersed reinforcement. The use of finely dispersed fibre reinforcement, regardless of material (steel, glass, basalt, etc.), aims to promote the formation of more ordered and homogeneous structures, characterized by high resistance to development of cracks [3, 5].

The wide application of fiber-reinforced concrete is found in many areas of construction and successfully used in countries such as South Africa, Germany, Japan, USA, etc. [6–13].

Currently in concrete science is widely used by different types of particulate fillers: crushed waste of metallurgical and energy industries, quartz sand, limestone, and carbonate, the Dolomites, there is a considerable resource base in many regions of the country.

Currently, more and more active in the construction industry implemented multi-component fine-grained concrete, the use of which previously was constrained by several factors: used as filler sand only led to a significant increase in the specific surface of the filler and its emptiness; for obtaining concrete mixtures required elevated (15–25 %) water consumption and cement compared with concrete large aggregate, which ultimately led to increase in shrinkage of concrete, etc [14, 15].

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Fine structure of cement composites has several advantages, among which: the possibility of creating high-quality fine uniform structure without inclusions of grains of large aggregate having a different structure in relation to the cement-sand matrix; high thixotropy and the ability to transform the concrete mix; the possibility of forming structures and products by casting, extrusion, pressing, stamping, spraying, etc. [1]. In addition, the size of the aggregate in concrete allows you to utilize the effect of particulate reinforcement in the manufacture of fiber-reinforced concrete.

Finely fibrous fillers in cement compositions to have a positive influence on the processes of structure formation, the strength of the filled concrete and other physical-mechanical and operational properties of materials. Use of dispersed reinforcement in the hardened cement mortars and concretes allows to:

- increase the tensile strength of shear and tensile bending, impact and fatigue strength;
- reduce shrinkage strain; to increase the fracture toughness by changing the nature of cracking at all levels of the structure;
- increase elasticity, resistance to impact and abrasion;
- increase the frost resistance, water resistance, heat resistance and fire resistance [16–21].

In cement composites, cracks are present from the sub micro - to the macro-scale level. The process of destruction of the concrete structure under the action of force factors at the micro level emerges as a local act of promoting primary cracks to the point of bifurcation, which is a defect structure in the form of grain filler or pores, while at the mouth of the crack resets the critical energy density. The process of destruction of the sample consists of discrete acts of destruction at the micro scale level.

Thus, the destruction of the concrete under action of static loads has a discrete character, and the feasibility of applying particulate reinforcement is dictated by the fractal hierarchy of process fracturing is offered.

A positive effect of the disperse reinforcement of concrete is beginning to tell after reaching the volumetric concentration of fibers contributing to the initial spatial coherence of fire tracker [22, 23]. It is possible to allocate two stages of the mechanism of influence on the structure of concrete and its physical-mechanical characteristics:

- at the stage of fiber structure formation during plastic shrinkage contributes to the redistribution of stresses from the shrinkage of the busiest areas on the entire volume of concrete;
- when loading in the process of operation of structures fiber of the fiber slow the growth of cracks, evenly distribute concentrations and reduce pressure in areas of macro defects, the surface area of the different components of concrete and the points of application.
- The theory of change of creep strain and shrinkage to date, the following [24]:
- reduction of shrinkage deformations is achieved through distributed interaction between fiber fibers with cement stone;
- in composite material with a perfect uniform arrangement of fibers, creep of the concrete matrix is limited to a tangential surface tension between the matrix and fiber reinforcement due to the forces of adhesion of materials.

At the present time a large number of experimental studies for specific production conditions and types of concrete (wire mesh, heavy-duty transport construction, etc.) are held. There is no scientific theoretical basis explaining the relationship of the properties of fine-grained concrete, the dosage and type of fiber of the fiber. To predict the possible positive effects it is necessary to study the interaction of fibers dispersed reinforcement with the mortar matrix, the relationship of the characteristics of various types of concrete for the purpose of modifying / adjusting the strength and performance characteristics to the structural analysis. To expand the scope of fiber in the manufacture of reinforced concrete structures it is necessary to systemize the production and the scientific and experimental experience that allows you to justify its use and define the deformation properties of the structures of fine-grained concrete. To expand the areas of calculation and application of dispersion-reinforced structures necessary theoretical generalization and systematization of the dependencies between process parameters (properties of the starting materials, composition and manufacturing process products), formation of structure and deformation-strength characteristics of fine-grained concrete.

The properties of the material of the fibers depend on the scope of fine-grained concrete and its characteristics. Fiber fibers can be nylon, acrylic, glass, steel, polyester, basalt, polypropylene, cotton and other materials (table 1). At present three main types of dispersion microanatomy are used abroad: fiber (fibre) in the form of short pieces of thin steel wire, glass and polypropylene fibers. In the Russian

Federation more widely used is the fiberglass on the basis of basalt [25]. The difference of material properties fiber reinforcement necessitates a differentiated approach to their use as reinforcement.

Table 1.Technical characteristics of the different fibers of a fiber

Figure	Basalt fiber	Polypropylene fiber	Fiber glass	Steel (metal) fiber
Material	Basalt fiber	Polypropylene	Fiber glass	Carbon steel Wire
Fiber Diameter	13-17 µm	10-25 µm	13-15 µm	0.5-1.2 µm
Fiber length	3.2-15.7 µm	6-18 µm	4.5-18 µm	30-50 µm
Melting point. °C	1 450	160	860	1 550
Resistance to alkali and corrosion	High	Low	Low, alkali – middle	Middle

Works of some researchers [16–21, 26, 27] established that to obtain high-strength compositions must satisfy the following conditions:

- a significant part of the strength of the fibers must be maintained in the process;
- you need a high fiber grip with the mortar matrix and the most dense of their contact without any entrapped air in the contact zone;
- optimal most uniform distribution of fibers throughout the volume of the matrix at simultaneous exclusion of direct contact with each other;
- materials of the fibers should have chemical inertness relative to the cement matrix;
- higher elastic modulus compared to the matrix.

Methods

However, the calculation of bearing capacity of structure, it is necessary to know the dependence of bending strength on the parameters of reinforcement. Comprehensive research work on the creation of multi-factor mathematical models of fiber-reinforced concrete is conducted in the following areas:

- the study of the macrostructure of the disperse-reinforced concrete: the location, the uniformity of distribution, effect of fiber on a surrounding structure of concrete;
- the interaction between fibers and concrete, manifested in the strength characteristics of the resulting composition;
- integration of dispersed reinforcement in the design strength and deformation characteristics of structures and prediction of internal stresses arising under the action of operational loads.

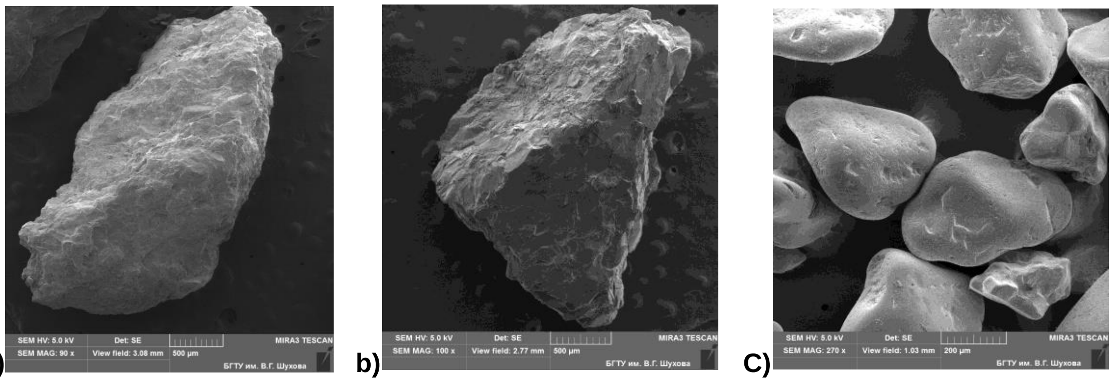
By results of the given experiments conclusions are drawn on that the substantial increase in the length of fiber fibers filler a larger particle size results in a deterioration of the concrete structure and decrease of the strength indicators. The required fiber length of the fiber for maximum strength in bending shall be determined by the maximum aggregate size of concrete, for example, for fine-grained concrete is 2–6 mm, for coarse – 12–20 mm.

Results and Discussion

Currently, experiments carried out at various constant parameters to cover all possible combinations of baselines. In the next stage of research it will be determined by other deformation and strength parameters of disperse-reinforced concretes that are required to calculate the model design. After the arrangement of empirical data they will be calculated based on the strength properties of dispersion-reinforced concrete from the entire range of initial parameters for use in mathematical models to calculate the strength characteristics of this type of concrete.

With the aim of recycling of technogenic raw materials are in the dumps of mining – waste wet magnetic separation of ferruginous quartzite and eliminations of crushing is proposed their use as filler for composite binders and aggregate of fine-grained fiber-reinforced concrete.

The influence of the shape and texture of aggregate on concrete strength is not sufficiently researched, but possibly a rougher texture results in greater force of adhesion between the particles and the cement matrix. In addition, the large surface area of angular aggregate means that you can develop greater traction. Conducted research using scanning electron microscopy (SEM) showed that the screenings from the crushing of granite and quartzitic sandstone (COI) have a rough surface and angular shape, in contrast to the natural sand with a smooth surface and rounded form of grains (Fig. 1).



a) b) C)

Figure 1. The grain: a – dropping out of quartzitic sandstone; b – dropout granite; c – natural sand

Main physico-mechanical properties of fillers are presented in Table 2.

Table 2. Physico-mechanical properties of filler

Indicator	COI Screening crushing	Screening granite	Tavolzhanskiy sand
Module size	3.50	2.89	1.38
Bulk density. kg/m^3	1490	1536	1448
True density. kg/m^3	2710	2640	2630
Voidness %	47.8	51.2	44.9
Water requirement. %	5.5	7.8	7

For obtaining of fine-grained fiber concrete with high performance characteristics, reduction of the clinker component and optimization of the processes of structure formation, appropriate use of highly active composite binders (KV) such as fine crushed cement (TMC) and cementing agent of low water requirement (VNV).

From the results of experiments found that grinding cement with plasticizing additive "Polyplast SP-1" in the amount of 0.6% by weight of cement is more intense. This shows that in addition to the plasticizing action, it has an intensifying effect at grinding; this is due to the wedging action of most additives. It is also seen that the kinetics of development TMC and VNV on attrition crushing granite similar and previously studied industrial raw materials.

The study of the properties of composite binders showed that VNV-100, the activity of more than 70 % higher compared to the baseline cement, decreases water-cement ratio and the normal density in comparison with cement (table 3).

Table 3. Physico-mechanical characteristics of the composite binder

View binder	The normal density of the test, %	Setting time, hours min.		V/C	The activity of the binder (MPa)	
		beginning	ending		bending	compression
CEM I 42.5 N	26.2	2-40	4-50	0.4	7.2	48.9
TMC-50 (on granite)	26.8	2-40	4-40	0.41	5.8	41.7
TMC-50 (on KVP)	27.1	2-30	4-40	0.43	6.5	46.3
VNV-50 (on granite)	23.2	2-10	4-30	0.33	5.2	47.1
VNV-50 (on KVP)	24.3	2-10	4-10	0.35	8.8	52.2
TMC-100	25.3	2-20	4-10	0.44	10.2	71.3
VNV-100	22.8	2-10	3-30	0.28	12.4	85.2

Thus, when introducing super plasticizer "Polyplast SP-1" in the amount of 0.6 % it is possible to obtain binding activity of 85.2 MPa.

The structure of cement stone on VNV-100 denser compared to ordinary Portland cement (Fig. 2).

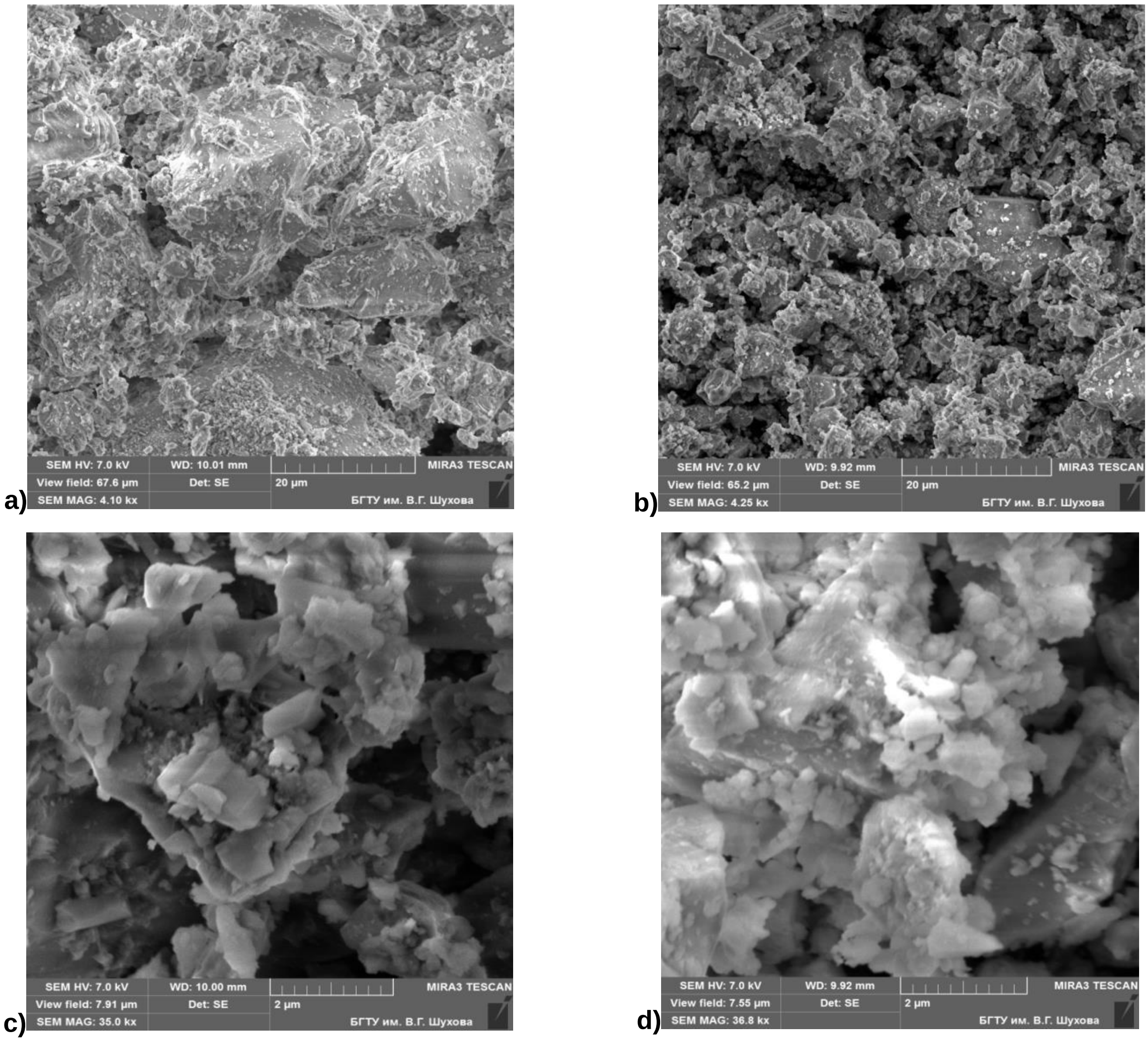


Fig. 2. Microstructure depending on the properties of the binders:
a, c – morphology of neoplasms of the cement stone CEM I 42,5 N;
b, d – morphology of neoplasms of the cement stone VNV-100;
increase a, b -×4000, c, d- ×35000

This is determined by the availability of the thinnest water films between the grains of binder and the formation in a confined volume of low-basic calcium hydrosilicates and other tumors.

To improve the operational characteristics were developed by the compositions of fine-grained fiber-reinforced concrete on technogenic raw materials (screening instruments) and composite binders with the use of nanodispersed powders (NDP). Nanodispersed powder was obtained by Professor V.V. Potapov from natural hydrothermal vents of Kamchatka Krai.

Powder, which is injected as a nano-additive in the cement samples had a specific surface area equal to 156000 m^2/kg (detection was carried out by low-temperature nitrogen adsorption on the ASAP parametre-2010 N Micromeritics), the average particle diameter of 7.3 nm, a density of 35 kg/m^3 (Fig. 3).

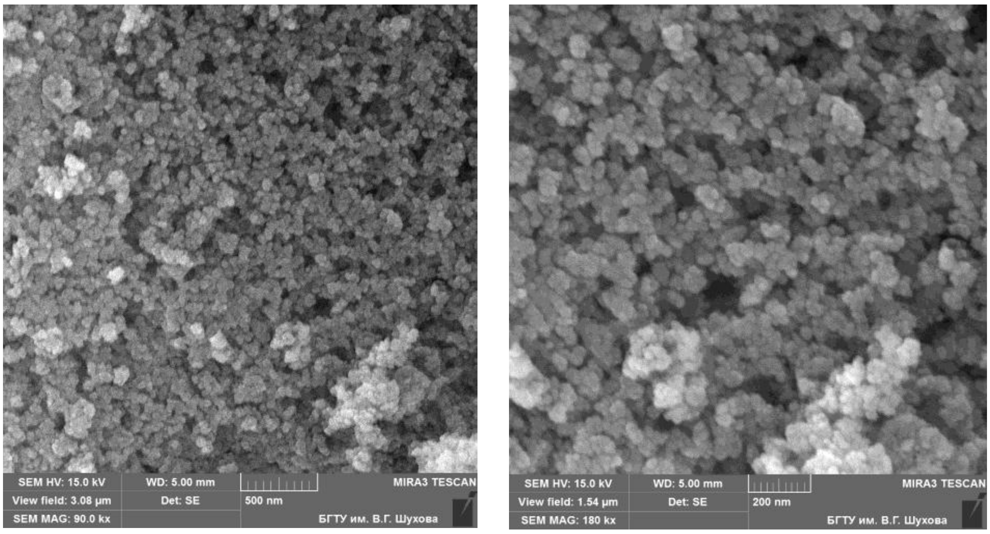


Figure 3. General view of the nanodispersed powder

Dried up sand, elimination of crushing of a quartzitic sandstone and composite astringent material have been mixed before obtaining homogeneous structure then the steel fiber was entered batch wise to avoid formation of their uniform distribution on all volume.

Nanodispersed powder was introduced into the aqueous phase before the mixing of the mixture in the amount of 0.2 % by weight of cement. Uniform distribution of powder particles in the liquid volume was achieved with the help of ultrasonic treatment till their full dissolution, the spent time for it up to 20 minutes. Then water with the dissolved nanodisperse modifier before receiving homogeneous mass was added. After formation and consolidation samples within 24 hours were at a temperature not lower than 15 °C. Then forms have been taken off and concrete samples there were in the curing camera with a temperature of 20 ° C and humidity more than 90 %. Test of samples for determination of durability on compression (cubes 100×100×100 mm) and on stretching at a bend (prisms 100×100×400 mm) were carried out by universal test machine by a standard technique.

Calculation of high density packing of mix fillers on the basis of which the percentage ratio of her components has been established has been made. Earlier [19] the efficiency of application composite astringent materials for fiber concrete has been proved since increase in strength characteristics gains to 60 % at insignificant extension in prime cost of mix due to additional power consumption by their receiving.

Table 4. Physico-mechanical characteristics of fine-grained concrete depending on the type of binding agent

View binder	Material consumption per 1 m^3 of mixture				The NDP. kg/m^3	Steel fiber. kg/m^3	The limit of compressive strength. (MPa)	Tensile Strength in bending. (MPa)
	Knit – ing. kg/m^3	Screening COI. kg/m^3	Sand. kg/m^3	Water. l/m^3				
CEM I 42.5 N	810	1100	340	204	-	-	56.3	14.3
CEM I 42.5 N+ VPU	810	1100	340	208	-	-	67.7	15.1
CEM I 42.5 N	810	1070	340	195	-	75	86.8	17.4
CEM I 42.5 N+VPU	810	1070	310	197	-	75	97.7	18.2
CEM I 42.5 N+VPU	810	1070	310	197	0.2	75	119.4	22.7

Table 5. Physico-mechanical characteristics of fine concrete depending on the type of binding agent

View binder	Material consumption per 1 m^3 of mixture				The NDP. kg/m^3	Steel fiber. kg/m^3	The limit of compressive strength. (MPa)	Tensile Strength in bending. (MPa)
	Kniting. kg/m^3	Screening COI. kg/m^3	Sand. kg/m^3	Water. l/m^3				
TMC-100	810	1100	340	209	-	-	65.7	15.5
TMC-100+SPM	810	1100	340	213	-	-	77.3	16.7
TMC-100	810	1070	310	199	-	75	92.6	19.1
TMC-100+SPM	810	1070	310	201	-	75	102.4	20.5
TMC-100+SPM	810	1070	310	201	0.2	75	127.1	26.5

Table 6. Physico-mechanical characteristics of fine concrete depending on the type of binding agent

View binder	Material consumption per 1 m^3 of mixture				The NDP. kg/m^3	Steel fiber. kg/m^3	The limit of compressive strength. (MPa)	Tensile Strength in bending. (MPa)
	Knit – ing. kg/m^3	Screening COI. kg/m^3	Sand. kg/m^3	Water. l/m^3				
VNV-100	810	1100	340	180	-	-	97.5	17.7
VNV-100+SPM	810	1100	340	185	-	-	100.6	18.6
VNV-100	810	1070	310	172	-	75	118.1	22.2
VNV-100+SPM	810	1070	310	174	-	75	125.8	24.1
VNV-100+SPM	810	1070	310	174	0.2	75	160.2	31.2

Conclusions

The study of physical and mechanical properties showed that the fiber-reinforced concrete on VNV-100 in all cases, exceed the characteristics of samples of similar composition made at other binders. Hence it can be concluded that the use of composite binders with the addition of superplasticizer can significantly increase the strength characteristics of concrete.

The proposed approaches improve the effectiveness of fine-grained concrete used for the production in the construction industry, namely, to optimize the structure at the nano-, micro - and macro-levels through the use of composite binders, nano-dispersed modifier, creation of high-density packing of grains of filler of technogenic raw materials and the dispersed reinforcing fibers, increasing the strength characteristics of the composite 3 times.

References

1. Bazhenov Yu.M., Demyanova V.S., Kalashnikov V.I. *Modifitsirovannyye vysokokachestvennyye betony* [Modified high-quality concrete]. Moscow: Izdatelstvo Assotsiatsii stroitelnykh vuzov, 2006. 368 p.
2. Grübe P., Lemmer C., Rühl M. Vom Gussbeton zum Selbstverdichtenden. *Beton.* Pp. 243–249.
3. Kleingelhöfer P. Noue Betouverflissiger auf Basis Polycarboxylat. *Proc. 13., Ybasil. Weimar.* 1997. Bd. 1. Pp. 491–495.
4. Richard P. Reactive powder concrete: a new ultra-high-strength cementitious. *Material 4th International Symposium on Utilization of High Strength/High*

Литература

1. Баженов Ю.М., Демьянова В.С., Калашников В.И. Модифицированные высококачественные бетоны. М.: Издательство Ассоциации строительных вузов, 2006. 368 с.
2. Grübe P., Lemmer C., Rühl M. Vom Gussbeton zum Selbstverdichtenden // Beton. Pp. 243–249.
3. Kleingelhöfer P. Noue Betouverflissiger auf Basis Polycarboxylat // Proc. 13., Ybasil. Weimar. 1997. Bd. 1. Pp. 491–495.
4. Richard P. Reactive powder concrete: a new ultra-high-strength cementitious // Material 4th International Symposium on Utilization of High Strength/High

Klyuev S.V., Klyuev A.V., Abakarov A.D., Shorstova E.S., Gafarova N.G. The effect of particulate reinforcement on strength and deformation characteristics of fine-grained concrete. *Magazine of Civil Engineering*. 2017. No. 7. Pp. 66–75. doi: 10.18720/MCE.75.6.

Performance Concrete. – Pressde l'Ecole Naturale des Ponts et Chaussées. Paris. France. 1996 Pp. 1343–1349.

5. Richard P, Cheyrezy M. Composition of Reactive Powder Concrete. Scientific Division Bouygues. *Cement and Concrete Research.* 1995. Vol. 25. No. 7. Pp. 1501–1511.
6. Brandt A.M. *Cement-Based Composites: Materials, Mechanical Properties and Performance.* Spon Press; 2 edition. 2009. 544 p.
7. Fernandes G.R. La influenca de alqunas caractericticas de las aridas finos (arenas) en la propiedades des hormogon de cemento Portland. *Cemento e Hormigon*. 1976. Vol. 47. No. 506. Pp. 415–428.
8. Herr O. Les materiaux autocompactants essorables de structure (MACES). Etude de faisabilite d'une nouvelle gamme de materiaux hydrauliques pour les assises de chausses. *Bulletin des laboratoires des Ponts et Chaussees.* 2001. No. 232. Pp. 99–103.
9. Lemm J.M. *HB Fiber-Reinforced Concrete: Principles, Properties, Developments and Applications (Building Materials Science).* William Andrew; illustrated edition Feb 1990. 194 p.
10. Maidl B. *Steel Fibre Reinforced Concrete.* Wiley-VCH, July 1995. 292 p.
11. Piasta J. Rheological properties of concretes with fine aggregate. *Cement and Concrete Researh.* 1985. Vol. 15. No. 2. Pp. 253–260.
12. Steopoe A. Sur la structure des suspensions aqueueses des ciments purs ou mélanges et sur les propriestes techniques de ces suspensions durcies. *Revue des Materiaux de conctructions.* 1981. No. 508. Pp. 1–9.
13. Takemura K. Some Properties of Concrete Using Crushed Stone Pust as Fine Aggregate. *The Cement Association of Japan. 13-th General Meeting Technical Session.* Tokyo, 1976. Pp. 95–97.
14. Lesovik R.V., Klyuev S.V., Klyuev A.V., Netrebenko A.V., Durachenko A.V. High-strength fiber-reinforced concrete containing technogenic raw materials and composite binders with use of nanodispersed powder. *Research Journal of Applied Sciences.* 2014. No. 9. Pp. 1153–1157.
15. Klyuyev S.V., Lesovik R.V., Klyuyev A.V., Netrebenko A.V., Kalashnikov N.V. Fiber Concrete on Composite Knitting and Industrialsand KMA for Bent Designs. *World Applied Sciences Journal.* 2014. No. 30(8). Pp. 964–969.
16. Lesovik R.V., Klyuev S.V., Klyuev A.V., Kazlitin S.A., Netrebenko A.V., Durachenko A.V. Fiber-Reinforced Concretes Made of Technogenic Raw Materials and Composite Binders for Industrial Building Floors. *International Journal of Applied Engineering Research (IJAER).* 2014. Vol. 9. No. 22. Pp. 15811–15824.
17. Rabinovich F.N. *Dispersno armirovannye betony* [Disperse-reinforced concrete]. M.: Strojizdat, 1989. 176 p.
18. Rabinovich F.N. *Kompozity na osnove dispersno armirovannykh betonov. Voprosy teorii i proyektirovaniya, tekhnologiya, konstruktsii* [Composites based on dispersion-reinforced concrete. Theory and design, technology, construction]: Monografiya. M.: Izdatelstvo ACB, 2004. 560 p.
19. Lesovik R.V., Klyuyev S.V., Klyuyev A.V., Netrebenko A.V., Yerofeyev V.T., Durachenko A.V. Fine-Grain concrete reinforced by polypropylene fiber. *Research Journal of Applied Sciences.* 2015. No. 10(10). Pp. 624–628.
20. Volkov I.V. Fibrobeton – sostoyaniye i perspektivy primeneniya v stroitelnykh konstruktsiyakh [The fiber-reinforced Concrete – the state and prospects of application in building construction]. *Stroitelnyye materialy, oborudovaniye i tekhnologii XXI veka.* 2004. No. 5. Pp. 5–7.
21. Lesovik R.V., Klyuyev S.V., Klyuyev A.V., Tolbatov A.A., Durachenko A.V. The Development of textile fine-grained

Performance Concrete. Pressde l'Ecole Naturale des Ponts et Chaussées, Paris. France. 1996 Pp. 1343–1349.

5. Richard P, Cheyrezy M. Composition of reactive powder concrete. scientific division bouygues // Cement and Concrete Research. 1995. Vol. 25. № 7. Pp. 1501–1511.
6. Brandt A.M. Cement-Based Composites: Materials, Mechanical Properties and Performance. Spon Press; 2 edition. 2009. 544 p.
7. Fernandes G.R. La influenca de alqunas caractericticas de las aridas finos (arenas) en la propiedades des hormogon de cemento Portland // Cemento e Hormigon. 1976. Vol.47. № 506. Pp. 415–428.
8. Herr O. Les materiaux autocompactants essorables de structure (MACES). Etude de faisabilite d'une nouvelle gamme de materiaux hydrauliques pour les assises de chaussees // Bulletin des laboratoires des Ponts et Chaussees. 2001. № 232. Pp. 99–103.
9. Lemm J.M. HB Fiber-Reinforced Concrete: Principles, Properties, Developments and Applications (Building Materials Science). William Andrew; illustrated edition Feb 1990. 194 p.
10. Maidl B. Steel Fibre Reinforced Concrete. Wiley-VCH, July 1995. 292 p.
11. Piasta J. Rheological properties of concretes with fine aggregate // Cement and Concrete Researh. 1985. Vol. 15. № 2. Pp. 253–260.
12. Steopoe A. Sur la structure des suspensions aqueueses des ciments purs ou mélanges et sur les propriestes techniques de ces suspensions durcies // Revue des Materiaux de conctructions. 1981. № 508. Pp. 1–9.
13. Takemura K. Some properties of concrete using crushed stone pust as fine aggregate // The Cement Association of Japan. 13-th General Meeting Technical Session. Tokyo. 1976. Pp. 95–97.
14. Lesovik R.V., Klyuev S.V., Klyuev A.V., Netrebenko A.V., Durachenko A.V. High-strength fiber-reinforced concrete containing technogenic raw materials and composite binders with use of nanodispersed powder // Research Journal of Applied Sciences. 2014. № 9. Pp. 1153–1157.
15. Klyuyev S.V., Lesovik R.V., Klyuyev A.V., Netrebenko A.V., Kalashnikov N.V. Fiber concrete on composite knitting and industrialsand kma for bent designs // World Applied Sciences Journal. 2014. № 30(8). Pp. 964–969.
16. Lesovik R.V., Klyuev S.V., Klyuev A.V., Kazlitin S.A., Netrebenko A.V., Durachenko A.V. Fiber-reinforced concretes made of technogenic raw materials and composite binders for industrial building floors // International Journal of Applied Engineering Research (IJAER). 2014. Vol. 9. № 22. Pp. 15811–15824.
17. Рабинович Ф.Н. Дисперсно армированные бетоны. М.: Стройиздат, 1989. 176 с.
18. Рабинович Ф.Н. Композиты на основе дисперсно армированных бетонов. Вопросы теории и проектирования, технология, конструкции: Монография. М.: Издательство АСВ, 2004. 560 с.
19. Lesovik R.V., Klyuyev S.V., Klyuyev A.V., Netrebenko A.V., Yerofeyev V.T., Durachenko A.V. Fine-grain concrete reinforced by polypropylene fiber // Research Journal of Applied Sciences. 2015. № 10(10). Pp. 624–628.
20. Волков И.В. Фибробетон – состояние и перспективы применения в строительных конструкциях // Строительные материалы, оборудование и технологии XXI века. 2004. № 5. С. 5–7.
21. Lesovik R.V., Klyuyev S.V., Klyuyev A.V., Tolbatov A.A., Durachenko A.V. The Development of textile fine-grained fiber concrete using technogenic raw materials // Research Journal of Applied Sciences. 2015. № 10(10). Pp. 701–706.
22. Рабинович Ф.Н. Об уровнях дисперсного армирования

fiber concrete using technogenic raw materials. *Research Journal of Applied Sciences.* 2015. № 10(10). Pp. 701–706.

22. Rabinovich F.N. Ob urovnyakh dispersnogo armirovaniya betonov [On the levels of dispersed reinforcement of concrete]. *Izvestiya Vuzov. Stroitelstvo.* 1981. No. 11. Pp. 30–36.
23. Chernyshov Ye.M., Korotkikh D.N. Povysheniye treshchinostoykosti tsementnogo betona pri mnogourovnevom dispersnom armirovanii yego struktury [Improving the crack resistance of cement concrete in multi-level dispersed reinforcement of its structure] // Sovremennyye problemy stroitelnogo materialovedeniya. Shestyye akademicheskiye chteniya RAASN. Belgorod, 2001. Pp. 587–598.
24. Figovskiy O.L., Beylin D.A., Ponomarev A.N. Uspekhi primeneniya nanotekhnologiy v stroitelnykh materialakh [The success of application of nanotechnology in building materials]. *Nanotekhnologii v stroitelstve.* 2012. No. 3. Pp. 6–21.
25. Ponomarev A.N. Vysokokachestvennyye betony. Analiz vozmozhnostey i praktika ispolzovaniya metodov nanotekhnologii [High-quality concretes. Analysis of possibilities and the practical use of nanotechnological methods]. *Magazine of Civil Engineering.* 2009. No. 6. Pp. 25–33.
26. Morozov V.I., Pukharenko Yu.V., Yushin A.V. The numerical investigations of double-span concrete beams strengthened with fiber reinforced plastics across the oblique section. *Materials Physics and Mechanics.* 2017. Vol. 31. No. 1-2. Pp. 40–43.
27. Bazhenov Yu.M., Lukuttsova N.P., Matveyeva Ye.G. Issledovaniye nanomodifitsirovannogo melkozernistogo betona [Study of nano-modified fine-grained concrete]. *Vestnik MGSU.* 2010. Vol. 2. No. 4. Pp. 415–421.

бетонов // Известия Вузов. Строительство. 1981. № 11. С. 30–36.

23. Чернышов Е.М., Коротких Д.Н. Повышение трещиностойкости цементного бетона при многоуровневом дисперсном армировании его структуры // Современные проблемы строительного материаловедения. Шестые академические чтения РААСН. Белгород, 2001. С. 587–598.
24. Фиговский О.Л., Бейлин Д.А., Пономарев А.Н. Успехи применения нанотехнологий в строительных материалах // Нанотехнологии в строительстве. 2012. № 3. С. 6–21.
25. Пономарев А.Н. Высококачественные бетоны. Анализ возможностей и практика использования методов нанотехнологии // Инженерно-строительный журнал. 2009. № 6. С. 25–33.
26. Morozov V.I., Pukharenko Yu.V., Yushin A.V. The numerical investigations of double-span concrete beams strengthened with fiber reinforced plastics across the oblique section // Materials Physics and Mechanics.2017. Т. 31. № 1-2. Pp. 40–43.
27. Баженов Ю.М., Лукутцова Н.П., Матвеева Е.Г. Исследование наномодифицированного мелкозернистого бетона // Вестник МГСУ. 2010. Т. 2. № 4. С. 415–421.

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Klyuev S.V., Klyuev A.V., Abakarov A.D., Shorstova E.S., Gafarova N.G. The effect of particulate reinforcement on strength and deformation characteristics of fine-grained concrete. *Magazine of Civil Engineering*. 2017. No. 7. Pp. 66–75. doi: 10.18720/MCE.75.6.

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Workability of high rockfill dam with a polymer face

Работоспособность высокой каменно-набросной плотины с полимерным экраном

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Key words: geomembrane; rockfill dam; combined dam; stress-strain-state; numerical modeling; strength; soil-cement; Bovilla dam; perimeter joint

Ключевые слова: геомембрана; каменно-набросная плотина; комбинированная плотина; напряжённо-деформированное состояние; численное моделирование; прочность; грунтоцемент; плотина Бовилла; периметральный шов

Abstract. The results of numerical study of stress-strain-state (SSS) of a rockfill dam with a face whose main watertight element is a polymer geomembrane. Analyses were conducted with consideration of non-linearity of contact interaction of structure elements and non-linearity of soil behavior. Bar finite elements were used for modelling of the thin geomembrane. The study was conducted on the example of Bovilla dam structural design which was built in 1996 in Albania. Initially Bovilla dam was planned to have a reinforced-concrete face, but later there was realized the design with multy-layered face consisting of PVC geomembrane, the underlying layer of soil-cement and a protection layer of reinforced concrete slabs. The face is conjugated with the concrete structure being an integral part of the dam. The results of numerical analyses showed that the weakest part of the dam design is the place of conjugation of the face with the concrete structure. The joint between the face and the concrete structure opens and the face shifts with relative to concrete surface. The polymer geomembrane may withstand these displacements without damages because the face structural design is provided with a compensating device in the form of a geomembrane loop. However, as calculations showed, the face design provokes formation of tensile stresses in protection reinforced concrete slabs and in the soil-cement underlying layer. In case the dam face was made of reinforced concrete, cracking in the face could be less probable. In our opinion, more feasible solution of the conjugation zone could be the alternative when the polymer geomembrane is placed over the face and is covered by soil protection layer. Taking into account high strength and safety of polymer geomembranes they may be recommended to be used as a backup seepage-control element of high concrete faced rockfill dams.

Аннотация. Рассматриваются результаты численного исследования напряжённо-деформированного состояния (НДС) каменно-набросной плотины с экраном, основным водонепроницаемым элементом которого является полимерная геомембрана. Расчёты проводились с учётом нелинейности контактного взаимодействия элементов конструкции и нелинейности поведения грунтов. Моделирование тонкой геомембраны использовались стержневые конечные элементы. Исследование проведено на примере конструкции плотины Bovilla, построенной в 1996 году в Албании. Первоначально плотину Bovilla планировалось выполнить с железобетонным экраном, но в последствии была реализована конструкция с многослойным экраном, состоящим из PVC-геомембраны, подстилающего слоя из грунтоцемента и защитного слоя из железобетонных плит. Экран сопрягается с бетонным сооружением, являющимся составной частью плотины. Результаты численных расчётов показали, что наиболее уязвимым узлом конструкции плотины является узел сопряжения экрана с бетонным сооружением. Шов между экраном и бетонным сооружением раскрывается, а экран смещается относительно поверхности бетона. Полимерная геомембрана может выдержать данные перемещения без повреждения, т.к. конструкция экрана предусматривает устройство компенсатора в виде петли геомембраны. Однако, как показали расчёты, конструкция экрана с компенсатором такова, что провоцирует образование растягивающих напряжений в защитных железобетонных плитах и подстилающем слое из грунтоцемента. В случае, если бы экран плотины был выполнен железобетонным, трещинообразование в экране была бы менее

вероятным. По нашему мнению, более удачным решением узла сопряжения был бы вариант, когда полимерная геомембрана уложена поверх экрана и закрыта защитным слоем грунта. Учитывая высокую прочность и надёжность полимерных геомембран, можно рекомендовать использовать их к дублирующий противофильтрационный элемент высоких каменно-набросных плотин с железобетонными экранами.

Introduction

At present the sphere of using synthetic (polymer) materials in hydraulic engineering is expanding. One of the promising trends of this process is use of polymer geomembranes as seepage-control elements in dams and dikes [1–5]. Already 60 years ago the polymer films started to be used for combating seepage in dams and dikes, but mainly in temporary structures [2, 6–8]. By present the possibilities of using polymer items have considerably increased; they became safer. Due to large thickness the modern geomembranes have good puncture resistance. During laboratory tests [9, 10] and field tests [11] it was established that polymer geomembranes may be used for a very long time. The guaranteed life of open geomembranes is 50-100 years [10]. Operation conditions of closed geomembranes in the dams working at temperatures about 0°C are the most favorable for polymer materials, therefore, many of them (for example, high-density polyethylene) may be effective for several hundred years [9].

Experience in hydraulic engineering shows, that up-to-date geomembranes may serve as seepage-control elements (SCE) of high dams. Namely, covered by a membrane the upstream face of concrete dams and reinforced concrete faces of rockfill dams may be repaired [2, 12, 13]. A number of dams have been constructed where SCE was arranged of geomembrane [2, 14].

In this regard, the type of an embankment dam appears to be promising, in which the main SCE is a polymer geomembrane, laid on smooth rigid bedding. This bedding may be arranged both of concrete and of a cheaper material - soil cement, i.e. mixture of soil with cement

Such a construction was realized in Bovilla dam in Albania in 1996 [1, 2, 15, 16]. This is the highest dam in the world with a seepage-control element made of a polymer geomembrane. The height of the earthfill dam is more than 71.6 m. Bovilla dam has a combined structure – a concrete water-retaining structure with a height of approximately 25 m is integrated in the combined dam (Fig. 1). It is deepened into a rock foundation. Taking into account the embedment of the concrete structure into the foundation the maximum construction height of Bovilla dam is almost 81.6 m. The rockfill is arranged with a lower slope of 1.6. The slope of its upstream face is variable - in the upper part it is 1.55, in the lower part it is 1.6. [1, 2, 15, 16].

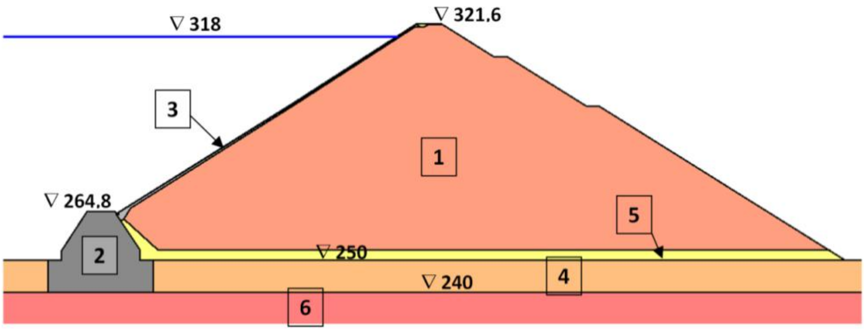

Figure 1. Cross section of Bovilla dam
1 – rockfill shell, 2 – concrete structure, 3 – multi-layer face, 4 – foundation soil layer, 5 – filter layer, 6 – rock foundation

Initially Bovilla dam was designed with reinforced concrete face, but then it was decided to change its structural design. The dam has a face of 3 mm thick PVC geomembrane. The face is placed on the layer of soil-cement bedding about half meter thick. From the upstream side the polymer face is covered with 20-30 cm thick precast concrete slabs. From both sides the geomembrane is protected by a geotextile layers. Thus, the seepage-control element has a complicated multi-layer structure where the geomembrane is the main but not the only one seepage-control measure.

The most complicated part in Bovilla dam design is conjugation of the rockfill part seepage-control face with concrete structure. The geomembrane is rigidly attached to concrete, but is placed with formation of a loop (compensator) permitting the face extension and displacement [2, 15]. A sand pad is

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provided between the soil-cement concrete and the reinforced concrete face to provide geomembrane free displacements.

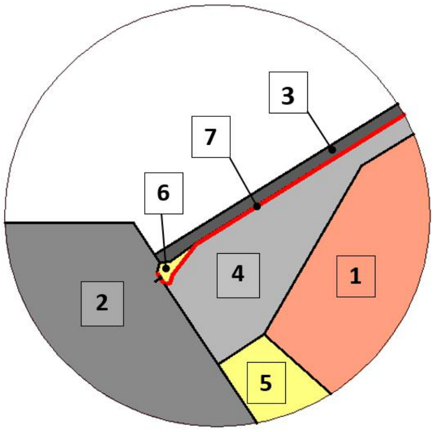

Figure 2. Design of contact zone between the face and the concrete structure.
1 – rip rap, 2 – concrete structure, 3 – reinforced concrete slabs, 4 – soil-cement bedding, 5 – filter layer, 6 – sand pad, 7 – geomembrane.

We studied stress-strain-state (SSS) of Bovilla dam design to assess safety of its SCE, including the place of its conjugation with the concrete structure. Besides, the task was assigned for assessment of effectiveness of the design solution for use of a polymer face in comparison with other structural alternatives.

For this purpose SSS of three SCE alternatives of rockfill dam were studied:

Alternative 1 – continuous by length reinforced concrete face (RCF) laid on soil-cement bedding. This is the initial non-realized alternative of Bovilla dam SCE;

Alternative 2 – realized SCE design where the polymer face was laid on soil-cement bedding and on the top covered with reinforced concrete slabs ;

Alternative 3 – SCE differing from that realized at Bovilla dam (alternative 2) by the fact that the polymer face was laid not on soil-cement bedding but on soil (sand-gravel) bedding.

Methods

SSS studies were conducted by finite element method with the aid of software worked out by Dr. Ph. (Tech) M.P. Sainov [17]. There were considered conditional flat cross section of the dam.

The finite-element model of the structure for Alternative 1 consisted of 730 finite elements, for Alternatives 2 and 3 – 830 finite elements. In calculations the non-linear character of interaction between non-soil structural elements and soils was taken into account. For this purpose the contact finite elements were introduced. In the model of Alternative 1 there were 83 such elements and in Alternatives 2, 3 – 128. The geomembrane in Alternatives 2, 3 was modelled by 27 bar finite elements.

In order to provide, at great difference between rigidity values of structural elements composing the dam, the sufficient accuracy of the obtained results of analyses all the finite elements had cubic approximation of displacements. The total number of degrees of freedom in the Alternative 1 model was 6854, in Alternatives 2, 3 – 7372.

During analyses the staged dam construction was taken into account: 32 design steps each simulating either construction of part of the structure or the reservoir filling.

For modeling non-linear behavior of soil medium the model of Professor L.N. Rasskazov was used [18]. Deformation properties of rockfill were assumed by analogs. The rockfill averaged modulus of deformation amounted to about 60 MPa.

Non-soil materials were assumed to be linearly deformed. For reinforced concrete the modulus of deformation was taken equal to 29000 MPa, Poisson’s ratio – 0.18. For soil-cement the modulus of deformation was taken equal to 5000 MPa, Poisson’s ratio 0.22. For geomembrane material the modulus of deformation was taken equal to 1000 MPa, which approximately corresponds to that of HD polyethylene (high density) [1]. In contacts there was used a non-linear model permitted their opening and slip.

Results and Discussion

By the results of analyses at the adopted properties of rockfill the maximum settlement of an embankment dam under its own weight amounts to 33 cm. The face under hydrostatic pressure displaces for 16.2 cm in direction normal to the slope surface (Fig.3).

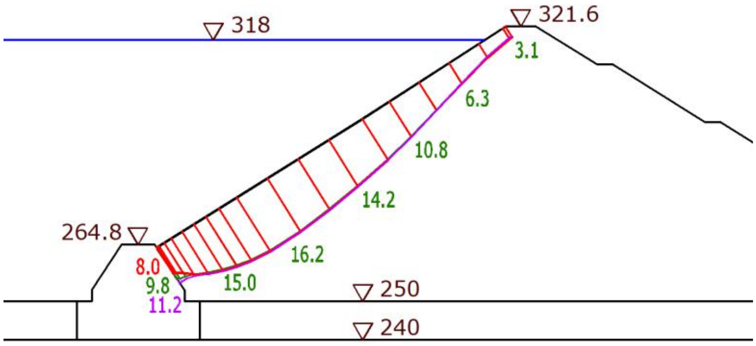

Figure 3. Displacements of the dam upstream face in perpendicular direction (deflections, cm). The green color indicates displacements for Alternative 2, red – for Alternative 1, violet – for Alternative 3.

In the zone of the face conjugation with the concrete structure the displacements of the upstream face are not equal to 0, because soil of the embankment dam slips relative to the concrete surface. The face displacement relative to the concrete structure comprised 8÷12 cm depending on the Alternative (Fig. 3). Besides, there observed opening of the joint between reinforced concrete slabs covering the upstream face and the concrete structure. In Alternative 1 it amounted to 1.8 cm, in Alternative 2 it was 2 cm.

The zone of conjugation between the earthfill and concrete structures is the weakest and the least safe zone in the dam design. In this zone the soil mass (and contact “concrete-earth”) is subject to complicated deformations and has unfavorable SSS. This also affects the SSS of reinforced concrete face (Alternative 1), reinforced concrete layer (Alternatives 2, 3), covering the dam upstream face.

In Alternative 1 the lower part of RCF adjoining the concrete structure is subject to deflection toward the upstream side, while the rest part deflects toward the downstream side. As a result tensile stresses in direction along the face (longitudinal stresses) arise in the RFC lower part. On the upper face of RCF they reach 2.8 MPa (Fig.4a). These stresses exceed design tensile strength of concrete. The rest part of RCF is compressed in longitudinal direction by stresses with values up to 5.3 MPa.

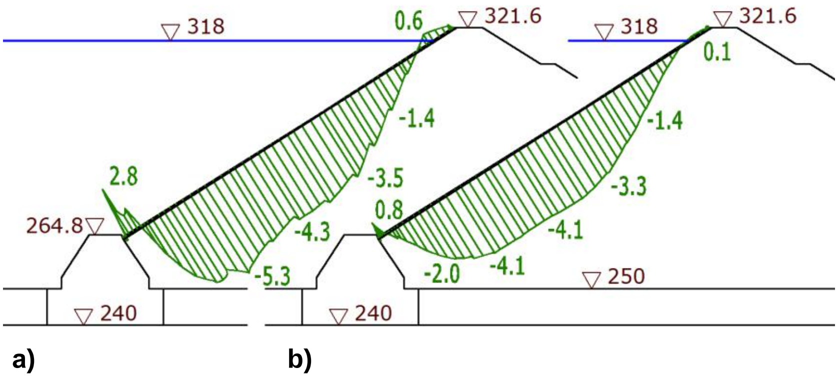

Figure 4. Longitudinal stresses in reinforced concrete face (MPa) in Alternative 1 a – on the upstream face, b – on the downstream face

Soil-cement under-face zone also has unfavorable SSS in the zone of contact with the concrete structure. Tensile stresses in it in longitudinal direction reach 1.7 MPa (Fig. 5b).

Thus, we may expect formation in RCF and in the under-face zone of transversal cracks which threaten water tightness of the dam SCE. This explains the reason why at designing Bovilla dam it was decided to refuse from RCF and choose the polymer face.

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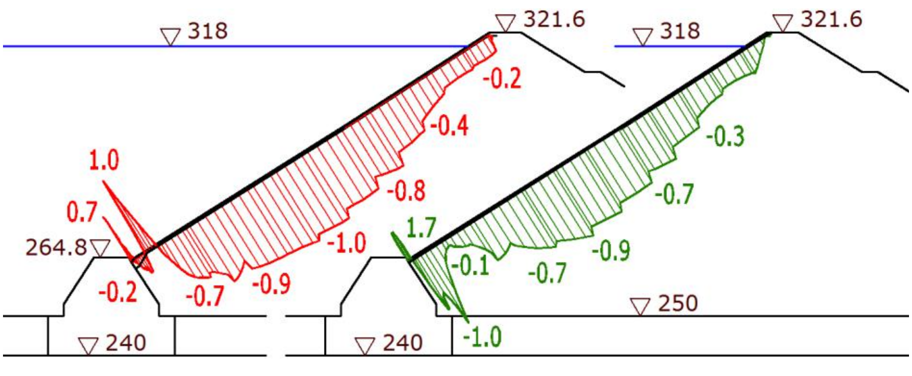


a) b)

Figure 5. Longitudinal stresses in the soil-cement bedding downstream face (MPa)
a – Alternative 2, b – Alternative 1.

However, at transfer to multi-layer structural design of SCE (Alterantive 2) the strength condition of the embankment dam constructions in the zone of contact with the concrete structure did not improve. This is connected with complicated arrangement of the interface zone. In the already constructed dam to provide geomembrane free deformations a cavity was provided between the concrete slabs and their soil-cement bedding. The soil-cement zone was expanded to avoid weakening of soil-cement bedding due to arrangement of the cavity.

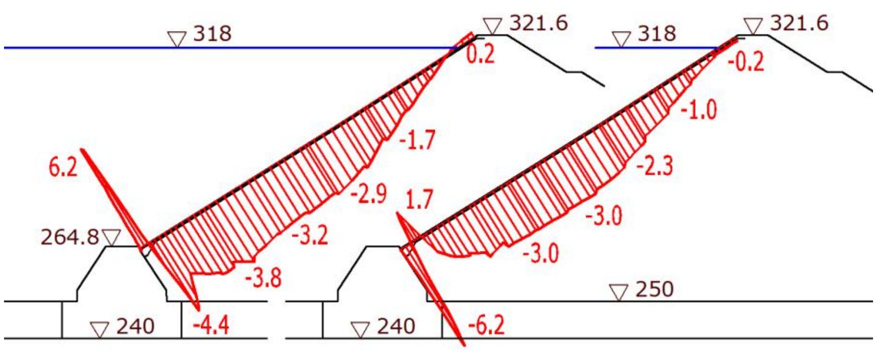
Such design of the interface affects its operation. Analyses permitted revealing the following pattern of this interface operation. Under the action of hydrostatic pressure the face tends to «slip» and turn relatively the concrete structure. But at turn the lower end of the soil-cement zone rests on the concrete structure, which cause in it concentration of compressive stresses of about 3 MPa. Due to presence of the thrust practically on the entire length, the contact of the multi-layer face and the concrete structure opens. The largest opening (2 cm) is observed on the top of the dam face.

In addition, the presence of a cavity between the concrete coating and soil-cement bedding affects the interface SSS (Fig. 2). Concrete cover with one part is pressed into the cavity, while the other is supported by low-compressible bedding. Due to this, the concrete cover undergoes strong bending deformations.

Due to bending, the top face of the coating of reinforced concrete slabs experiences tensile longitudinal stresses, which reach 6.2 MPa (Fig. 6b). The zone of tension extends to 5.8 m. A zone of tensile stresses is also formed on the downstream face at a certain distance (Fig. 6b). Thus, the formation of cracks in the reinforced concrete coating is inevitable.

The unfavorable SSS also has an under-face zone of soil-cement (Fig. 5a), and cracking is also expected in it.

Thus, water impermeability of SCE is entirely determined by the integrity of the polymer geomembrane.



a) b)

Fig.6. Longitudinal stresses in the reinforced concrete cover (MPa) in Alternative 2
a – on the upstream face, b – on the downstream face.

By the results of analyses due to the openings and shear movements on the contact of the face with the concrete structure, the geomembrane undergoes deformations of the elongation. The geomembrane's loop thus straightens. Due to straightening the deformations of elongation do not cause significant tensile stresses in the geomembrane. According to the results of analyses with deformation modulus of 1000 MPa, the maximum of tensile stresses is only 0.5 MPa. This is much less than the tensile strength of polyethylene (30 MPa). If we assume that the geomembrane is made of PVC, then the safety margin of the geomembrane will be even higher.

In the rest of the geomembrane parts no significant stresses occur.

Thus, the design of the interface of a multi-layer screen with a concrete structure ensures integrity and reliability of geomembranes at displacements on the contact between two structures, but at the same time provokes crack formation in the elements protecting the geomembrane: in the cover of reinforced concrete slabs and in soil-cement bedding. Damage to these rigid structures is dangerous for the geomembrane, because sharp edges of damaged elements may cause a puncture.

Therefore, another calculation was carried out - for Alternative 3, in which the hard soil-cement bedding was replaced with gravel-sand. The analyses showed that in this case the state of the interface is somewhat improved, but not radically. Opening of the joint decreases to 1.7 cm, but the face displacement relative to the concrete structure increases to 11.2 cm (Fig. 3). Also, as in Alternative 2, a zone of tensile longitudinal stresses is formed on the upstream face of the reinforced concrete cover. However, they do not exceed 4.7 MPa (Fig. 7a).

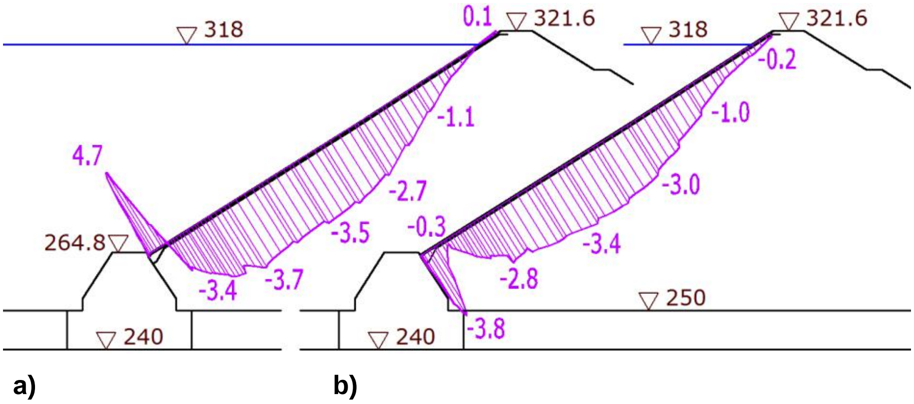


Fig.7. Longitudinal stresses in the reinforced concrete cover (MPa) in Alternative 3
a – on the upstream face, b – on the downstream face.

Thus, in Alternative 3 damages of reinforced concrete cover and subsequent puncture of geo-membrane may occur. In order to avoid them it is better to refuse from using concrete cover in the lower part of the dam and substitute it by a layer of impervious soil. Similar design of interface was used during repairs of New Exchequer dam [19]. New Exchequer dam is of similar design as Bovilla dam; it also refers to combined dams, but is was made not with a polymer face but with a reinforced concrete face. In this dam the conjugation place of the reinforced concrete face and the concrete structure is covered on the top by a geo-membrane, which in its turn is covered by a soil layer.

This example shows that at construction of Bovilla dam it was not necessary to refuse from the reinforced concrete face in favor of the polymer face. It was sufficient to lay a geo-membrane in the lower part of the face to provide water tightness. It should be stressed that substitution of the reinforced concrete face by a polymer face did not decrease the cost of Bovilla dam, because its structural design envisages maintaining the reinforced concrete cover of the upstream slope for protection of the polymer face.

The advantage of using the polymer face is a high level of safety of the dam seepage control facility. The example of Boviila dam permits recommending for construction of ultra-high dams the use of combined (double) seepage-control element consisting of reinforced concrete (or soil-cement) face covered by a geo-membrane. However, the geo-membrane should not be open, because otherwise its service life decreases. It is reasonable to cover the combined seepage-control element with a soil layer. Earlier we recommended the dam of such structural design [20].

Conclusions

Use of polymer geomembrane as a seepage-control face of a high rockfill dam significantly improves the reliability of the dam seepage-control facility. This is because polymer materials have high strength and elasticity, which allows them to experience significant deformations inherent in embankment dams without damage. Rockfill dams with a seepage-control face of polymer geomembrane are more efficient type of dams than dams with reinforced concrete face. Rational solution is the construction of rockfill dams with a combined (double) seepage-control element, including a reinforced concrete (or soil-cement) face covered with a geomembrane/

The most vulnerable node of a dam design with a polymer face is its interface with a rock foundation or a concrete structure. In this zone, one can expect the appearance of tensile stresses in the geomembrane. Used in the Bovilla dam conjugation (which provides arrangement of the geomembrane loop) ensures the integrity of the geomembrane, even with significant movements at the interface. However, there is a risk of damage to the protective layers of the geomembrane, which may entail damage to the geomembrane. Apparently, it is more rational to provide a protective coating in the compensator assembly of the geomembrane not of rigid concrete slabs, but of soil or other pliable material, as was done during repairs of New Exchequer dam.

References

1. ICOLD. *Geomembrane sealing systems for dams. Design principles and review of experience.* Bulletin 135, 2010. 464 p.
2. Cazzuffi D., Giroud J.P., Scuero A., Vaschetti G. Geosynthetic barriers systems for dams. *9th International Conference on Geosynthetics.* Brazil, 2010. Pp. 115–163.
3. Radchenko V.P., Semenkov V.M. Geomembrany v plotinakh iz gruntovykh materialov [Geomembranes in dams from soil materials]. *Gidrotekhnicheskoye stroitelstvo.* 1993. No. 10. Pp.46–52. (rus).
4. Solskiy S.V., Orlova N.L. Perspektivy i problemy primeneniya v gruntovykh gidrotekhnicheskikh sooruzheniyakh sovremennykh geosinteticheskikh materialov [Prospects and problems of application in groundwater waterworks modern geosynthetics]. *Izvestiya VNIIG im. B.Ye.Vedeneyeva.* 2010. Vol. 260. Pp. 61–68. (rus)
5. Glagovskiy V.B., Solskiy S.V., Lopatina M.G., Dubrovskaya N.V., Orlova N.L. Geosinteticheskiye materialy v gidrotekhnicheskom stroitelstve [Geosynthetics in hydraulic engineering]. *Gidrotekhnicheskoye stroitelstvo*. 2014. No. 9. Pp. 23–27. (rus)
6. Reltov B.F., Krichevskiy I.Ye. Perspektivy primeneniya rulonnykh plastmass v kachestve ekranov plotin iz mestnykh materialov [Prospects for the use of plastics as a roll screen embankment dams]. *Gidrotekhnicheskoye stroitelstvo*. 1964. No. 1. Pp. 29–32. (rus)
7. Abramson Yu.L, Tolkachev L.A., Fishman Yu.A. Stroitelstvo na gornykh rekakh vysokikh zemlyanykh peremychek v dve ocheredi s perelivom pavodka na promezhutochnoy otmetke [Building on the high mountain rivers embankment cofferdams in two stages with an intermediate overflow flood mark]. *Gidrotekhnicheskoye stroitelstvo*. 1968. No. 1. Pp. 8–12. (rus).
8. Belikov M.P., Panasenko G.A., Antsiferov A.S. Zemlyanaya plotina s plastmassovoy protivofiltratsionnoy zashchitoy v rayone vechnoy merzloty [Earthfill dam with plastic anti-seepage protection in the permafrost area]. *Gidrotekhnicheskoye stroitelstvo*. 1968. No. 10. Pp. 20–24. (rus).
9. Koerner R.M., Hsuan Y.G., Koerner G.R. Lifetime predictions of exposed geotextiles and geomembranes. *Geosynthetics International*. 2017. No. 24(2). Pp. 198–212.
10. Tian K., Benson C.H., Tinjum J.M., Edil T.B. Antioxidant depletion and service life prediction for HDPE geomembranes exposed to low-level radioactive waste leachate. *Journal of Geotechnical and Geoenvironmental Engineering.* 2017. No. 143(6). 04017011.
11. Gallagher E.M., Tonks D.M., Shevelan J., Belton A.R., Blackmore R.E. Investigations of geomembrane integrity within a 25-year old landfill capping. *Geotextiles and Geomembranes*. 2016. No. 4(5). Pp. 770–780.
12. Scuero A.M., Vaschetti G.L. Underwater repair of a 113 m high CFRD with a PVC geomembrane: Turimiquire

Литература

1. ICOLD. Geomembrane sealing systems for dams. Design principles and review of experience. Bulletin 135, 2010. 464 p.
2. Cazzuffi D., Giroud J.P., Scuero A., Vaschetti G. Geosynthetic barriers systems for dams // 9th International Conference on Geosynthetics. Brazil, 2010. Pp. 115–163.
3. Радченко В.П., Семенков В.М. Геомембраны в плотинах из грунтовых материалов // Гидротехническое строительство. 1993. № 10. С. 46–52.
4. Сольский С.В., Орлова Н.Л. Перспективы и проблемы применения в грунтовых гидротехнических сооружениях современных геосинтетических материалов // Известия ВНИИГ им. Б.Е.Веденеева. 2010. Т.260. С. 61–68.
5. Глаговский В.Б., Сольский С.В., Лопатина М.Г., Дубровская Н.В., Орлова Н.Л. Геосинтетические материалы в гидротехническом строительстве // Гидротехническое строительство. 2014. №9. С. 23–27.
6. Рельтов Б.Ф., Кричевский И.Е. Перспективы применения рулонных пластмасс в качестве экранов плотин из местных материалов // Гидротехническое строительство. 1964. № 1. С. 29–32.
7. Абрамсон Ю.Л, Толкачев Л.А., Фишман Ю.А. Строительство на горных реках высоких земляных перемычек в две очереди с переливом паводка на промежуточной отметке // Гидротехническое строительство. 1968. № 1. С. 8–12.
8. Беликов М.П., Панасенко Г.А., Анциферов А.С. Земляная плотина с пластмассовой противофильтрационной защитой в районе вечной мерзлоты // Гидротехническое строительство. 1968. № 10. С. 20–24.
9. Koerner R.M., Hsuan Y.G., Koerner G.R. Lifetime predictions of exposed geotextiles and geomembranes. Geosynthetics International. 2017. No.24(2). Pp. 198–212.
10. Tian K., Benson C.H., Tinjum J.M., Edil T.B. Antioxidant depletion and service life prediction for HDPE geomembranes exposed to low-level radioactive waste leachate // Journal of Geotechnical and Geoenvironmental Engineering. 2017. No.143(6). 04017011.
11. Gallagher E.M., Tonks D.M., Shevelan J., Belton A.R., Blackmore R.E. Investigations of geomembrane integrity within a 25-2017 old landfill capping // Geotextiles and Geomembranes. 2016. No.4(5). Pp. 770–780.
12. Scuero A.M., Vaschetti G.L. Underwater repair of a 113 m high CFRD with a PVC geomembrane: turimiquire managing dams: challenges in a time of change // Proceedings of the 16th Conference of the British Dam Society. 2010. Pp. 474–486.
13. Sembenelli P., Rodriquez E.A. Geomembranes for earth and earth-rock dams: state-of-the-art report // Proc. Geosynthetics Applications, Design and Construction, M.B. de Groot, et al., Eds., A.A. Balkema. 1996. Pp. 877–888.
14. Корчевский В.Ф., Обополь А.Ю. О проектировании и

Managing Dams: Challenges in a Time of Change. *Proceedings of the 16th Conference of the British Dam Society*. 2010. Pp. 474–486.

13. Sembenelli P., Rodriquez E. A. Geomembranes for earth and earth-rock dams: state-of-the-art report // Proc. Geosynthetics Applications, Design and Construction. 1996. Pp. 877–888.
14. Korchevskiy V.F., Obopol A.Yu. O proyektirovanii i stroitelstve Kambaratinskikh gidroelektrostantsiy na r. Naryne v Kirgizskoy Respublike [On the design and construction of the Kambarata hydropower stations on the Naryn river in Kyrgyzstan]. *Gidrotekhnicheskoye stroitelstvo.* 2012. No. 2. Pp. 2–12. (rus).
15. Sembenelli P., Sembenelli G., Scuero A.M. Geosynthetic system for the facing of Bovilla Dam. *Proceedings of the 6th International Conference on Geosynthetics.* Atlanta, Georgia, USA. 1998. International Fabrics Association International, Roseville. Minnesota. Pp. 1099–1106.
16. Sembenelli P., Sembenelli G., Scuero A. Bovilla: a product of dam history. *Geotechnical Fabrics Report*. 1998. Vol. 16. No. 6. Pp. 30–36.
17. Sainov M.P. Vychislitelnaya programma po raschetu napryazhenno-deformirovannogo sostoyaniya gruntovykh plotin: opyt sozdaniya, metodiki i algoritmy [Computer program for the calculation of the stress-strain state of soil dams: the experience of creation, techniques and algorithms]. *International Journal for Computational Civil and Structural Engineering*. 2013. No. 9(4). Pp. 208–225. (rus).
18. Rasskazov L.N., Dzhkha Dzh. Deformiruyemost i prochnost grunta pri raschete vysokikh gruntovykh plotin [Deformability and Strength of Soils in High Soil Dam Calculation]. *Gidrotekhnicheskoye stroitelstvo*. 1987. No. 7. Pp. 31–36. (rus).
19. Brown H.M., Kneitz P.R. Repair of new exchequer dam. *Water Power and Dam Construction*. 1987. No. 39(9). Pp. 25–29.
20. Sainov M.P., Kotov F.V. Razrabotka i obosnovaniye konstruktsii kamennoy plotiny s gruntotsementobetonnym ekranom dlya usloviy Kraynego Severa [Development and Substantiation of the Structure of a Masonry Dam Having a Soil Cement Membrane and Designated for the Climate of the Far North of Russia]. *Vestnik MGSU*. 2013. No. 3. Pp. 187–195.

строительстве Камбаратинских гидроэлектростанций на р. Нарыне в Киргизской Республике // Гидротехническое строительство. 2012. № 2. С. 2–12.

15. Sembenelli P., Sembenelli G., Scuero A.M. Geosynthetic system for the facing of Bovilla Dam // Proceedings of the 6th International Conference on Geosynthetics. Atlanta, Georgia, USA. 1998. International Fabrics Association International, Roseville. Minnesota. Pp.1099–1106.
16. Sembenelli P., Sembenelli G., Scuero A. Bovilla: a product of dam history // Geotechnical Fabrics Report. 1998. Vol.16. № 6. Pp. 30–36.
17. Саинов М.П. Вычислительная программа по расчёту напряжённо-деформированного состояния грунтовых плотин: опыт создания, методики и алгоритмы // International Journal for Computational Civil and Structural Engineering. 2013. № 9(4). С. 208–225.
18. Рассказов Л.Н., Джха Дж. Деформируемость и прочность грунта при расчете высоких грунтовых плотин // Гидротехническое строительство. 1987. № 7. С. 31–36.
19. Brown, H.M., and Kneitz, P.R. Repair of new exchequer dam // Water Power and Dam Construction. 1987. № 39(9). Pp. 25–29.
20. Саинов М.П., Котов Ф.В. Разработка и обоснование конструкции каменной плотины с грунтоцементобетонным экраном для условий Крайнего Севера // Вестник МГСУ. 2013. № 3. С. 187–195.

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Selection criteria of space planning and structural solutions of low-rise buildings

Критерии выбора объемно-планировочных и конструктивных решений малоэтажных зданий

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Key words: optimal selection; energy-efficiency; comfort; safety; effectiveness of capital investments; space-planning and structural solutions; low-rise construction

Ключевые слова: оптимальный выбор; энергоэффективность; комфорт; безопасность; эффективность капитальных вложений; объемно-планировочные и конструктивные решения; малоэтажное строительство

Abstract. The present study is devoted to development of methodology used for optimal selection of space-planning and structural solutions of low-rise buildings. Objective of the study is developing the system of criteria influencing the optimal selection of space-planning and structural solutions of low-rise buildings and structures aimed at enhancing the efficiency of capital investments, energy and resource saving, creating comfortable conditions for the population considering climatic zoning of the construction site. Developments of the project can be applied while implementing investment-construction projects of low-rise housing at different kinds of territories based on the local building materials. The system of criteria influencing the optimal selection of space-planning and structural solutions of low-rise buildings has been developed. Methodological basis has been also elaborated to assess optimal selection of space-planning and structural solutions of low-rise buildings satisfying the requirements of energy-efficiency, comfort and safety, and economical efficiency. Elaborated methodology enables to intensify the processes of low-rise construction development for different types of territories taking into account climatic zoning of the construction site. Development of low-rise construction processes should be based on the system of approaches which are scientifically justified; thus it allows enhancing energy efficiency, comfort, safety and economical effectiveness of low-rise buildings.

Аннотация. Проведено исследование по разработке методики оптимального выбора объемно-планировочных и конструктивных решений малоэтажных зданий. Целью исследования является разработка системы критериев, влияющих на оптимальность выбора объемно-планировочных и конструктивных решений малоэтажных зданий и сооружений, направленных на повышение эффективности капиталовложений, энерго- и ресурсосбережение, создание комфортных условий для населения с учётом климатического районирования местности строительства. Данные разработки могут быть применены при реализации инвестиционно-строительных проектов малоэтажной застройки различного типа территорий на основе местной базы строительных материалов. Разработана система критериев, влияющих на оптимальность выбора объемно-планировочных и конструктивных решений малоэтажных зданий. Разработаны методические основы оценки оптимальности выбора объемно-планировочных и конструктивных решений малоэтажных зданий, отвечающих требованиям энергоэффективности, комфорта и безопасности, экономической эффективности. Разработанная методика позволяет интенсифицировать процессы развития малоэтажного строительства для различного типа территорий с учетом климатического районирования местности строительства. Развитие процессов малоэтажного строительства должно основываться на системе научно-обоснованных подходов, что позволяет повысить энергоэффективность, комфорт, безопасность и экономическую эффективность малоэтажных зданий.

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Introduction

Strategic goal of state housing policy is defined as formation of affordable housing market, meaning economy-class buildings with integration of energy sources, energy-saving technologies, modern materials, and provided with comfortable living conditions for the citizens. In this regards a number of federal programs has been approved; the programs are aimed at integrated land use planning and management, increasing the performance of housing construction development enabling to create comfortable living conditions [1–2].

Special place in this process should be taken by low-rise multiple-flats construction, as this type of construction is considered to be one of the most optimal formats of new territories development where there are no problems with free land plots and these territories are marked with high provision of natural energy producing materials (hydrocarbons, biofuel, etc.).

Increase in growth rates of comprehensive low-rise housing development, expansion of social and transport infrastructure (stimulating social and economic processes) requires development of optimal space-planning and structural solutions of buildings at the design stage depending on its geographical location.

It should be noted that Russia has a great potential in the field of energy efficiency and use of renewable energy sources. However low efficiency of applying energy-saving technologies in construction is a reason of high costs of housing and public utilities during buildings operation. Such a situation testifies on the necessity of rational use of independent heat supply sources in the projects of low-rise housing taking into account climatic zoning.

Integrated approach for optimal selection of space-planning and structural solutions will contribute to increasing the volume of construction of economy-class houses, reduction of its costs, corresponding to the requirements of comfort, safety and energy efficiency of buildings, increasing the amount of citizens being able to improve their living conditions on their own. This will enable to solve the problem of generating capacity deficit on one side and save significant amount of money of owners of houses on the other side [3].

Foreign construction experience is mainly based on environmental protection, i.e. introduction of efficient and profitable measures to reduce greenhouse gas emissions [4]. Housing construction in most countries is considered as a unique system enabling to provide enhancement of new projects. The main directions of system approach are as follows: 1) heating and cooling, 2) hot water, 3) household appliances, 4) lighting, 5) fridges and deep-freezers [5]. More and more countries pass on to renewable energy sources from oil and gas; preliminary calculations show that it allows reduction of energy consumption of building by 60–80% [6]. Within the project CEPHEUS (Cost Efficient Passive Houses as European Standards) five European countries built houses according to the Passivhaus standard from scientific point of view. In order to reduce energy for heating the following measures have been performed: appropriate heat isolation, passive use of solar energy, extra glazing, and active ventilation [7].

Some authors suggest improving the optimal shape of low-rise buildings and defining the optimal building structure that could provide the required capacity of wind power generator [8]. Other authors offer two-stage design optimization approach resulting in high indicators of energy-efficiency. Apart from that, they investigate single-sided ventilation and cross-ventilation models under varied thermal load requirements to compare the preferable design solutions for each scenario [9].

Low-rise construction in Russia is currently one of the most significant reserves of construction complex development that enables to increase the construction growth rate in a down economy. However it needs not only the governmental assistance, but also the strict requirements of legal framework.

Quite large number of research papers are devoted to the issues of standard setting, increasing of requirements to thermal protection of buildings and energy efficiency [10–13]. Authors consider the development history of regulatory requirements to thermal protection, the problems of energy efficiency in construction, requirements of buildings energy efficiency. The mentioned studies are aimed at regulation of irrational consumption of energy sources and maintaining the definite parameters of internal environment inside the building. Conducted research on development of low-rise construction in Russia, they concluded the general projected growth of low-rise construction, including: energy efficiency, economical efficiency and ecological development of low-rise housing

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[14]. The most complete typology of low-rise buildings and problems of low-rise housing development on the territory of Russia [15]. Described the principles of formation of space-planning solutions for low-rise buildings and comfortable living [16–17]. The typology of buildings upon the level of their energy efficiency can be found in [18]. The methodology to evaluate the effectiveness of applying energy-efficient measures in buildings [19]. In monograph [20] conducts a complete consideration of energy consumption and primary energy and resource consumption, however, the issues of maintenance costs has not been solved.

The whole complex of studies in energy-efficient buildings and their structures is based on a substantive knowledge basis [21–24]. However, though many studies have been conducted in the field of energy efficiency and energy saving in construction, still they are characterized by fragmentary mature and are not completely systematized upon the special features of regional conditions. Regulatory documents in energy efficiency mostly present results of studies devoted to separate regions [25].

Special attention should be paid to development of new space-planning solutions, search of optimal structural solutions giving a special accent on local materials use, meeting the requirements of energy efficiency, effectiveness of capital investments, comfort and safety and the use of renewable energy sources for engineering systems during the whole life cycle of a building.

Materials and methods

Results of systematization of the objects of low-rise construction necessary for development of classification framework enable to define the criteria for optimal selection of space-planning and structural solutions of low-rise buildings. These criteria can be defined as follows:

1. Safety.
2. Comfort of living.
3. Energy efficiency.
4. Economic efficiency.

Based on the suggested classification and criteria framework the indicators forming methodological environment for selection of optimal space-planning and structural solutions for low-rise buildings have been defined (Fig.1).

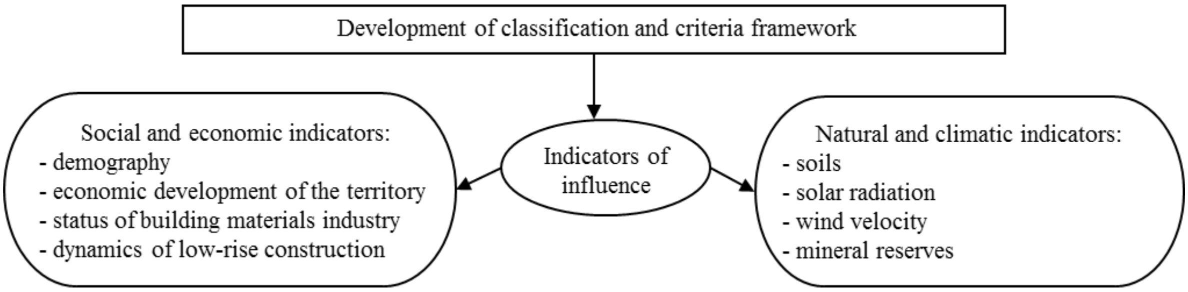

Figure 1. Indicators for defining optimal space-planning and structural solutions of low-rise buildings

Suggested methodology allows moving beyond the traditional schemes of selection of space-planning and structural solutions of low-rise buildings and gives a chance to make an optimal choice using the defined criteria. The resulting matrix is given in Table 1.

The first criterion is defined as safety of living. For that purpose wall materials are typologized by the following features: the degree of fire resistance, structural design of building, thickness of walls, wear degree (for old buildings). Thus we can define the external shell of a building at the initial stage.

The next criterion is defined as comfort of living. The given criterion is classified by the type of territory of living: urban, suburban, rural, remote. Depending on the territory type typological criterion is appropriate to be classified by the ratio of land plot area to the total area of the building. Low-rise buildings of economy class are required to have the definite land plot area. Total area of house land plots depends on the territory of living. The number of floors is typologized up to three floors.

The next criterion is building typology according to energy efficiency. It includes energy efficiency of structural elements of low-rise building and type of energy supply. In order to assess heat protective properties of building envelopes we need to define the climatic conditions of the construction site. Construction-based climatic zones are defined to find optimal design and space-planning solutions of urban low-rise housing construction for each of the zones.

All residential units referring to the group of economy-class housing should correspond to the requirements influencing the energy efficiency of buildings; the energy consumption is checked with respect to accordance to the established standards. Energy efficiency classes are established in accordance with the acting regulatory documents.

Apart from building envelopes being the main reserve of energy saving one also needs reasonable interlinked arrangement of energy sources and energy consumers (electricity, heating, gas), reducing expenses for transportation. Selection and justification of applying centralized, locally-centralized and local systems of engineering equipment depends on the type of living territory. Selection of independent energy sources and non-renewable energy sources depends on the opportunity to provide engineering specifications for connection to urban network.

The final stage is the criterion of capital investments. It is formed from the construction cost, cost for engineering systems of a building, total expenses for operation of building during the whole lifecycle.

Total expenses for construction and building operation will enable to define the effectiveness of capital investments per square meter by comparative method with traditional systems. Resulting from calculation we get construction cost of low-rise building and cost per square meter during the whole life cycle of a building.

Results and Discussion

Foreign experience of many countries shows the need to improve the requirements for ensuring the required energy efficiency in buildings [26]. One of the important tasks for today is to determine the optimal typology of buildings depending on climatic conditions. Recently, the building typology is being developed and enhanced all over the world. In EU countries, in the USA, and in China they use the project on building typology named TABULA. The number of building types differs in different countries, e.g. there are 14 types in the USA, 72 in Spain [27].

Consideration of these recommendations in the practice of modern housing construction can serve the basis for establishing novel advanced tendencies in architecture of comprehensive low-rise housing construction both in space-planning as well as in structural solution of low-rise houses meeting the requirements of energy-efficiency.

This section will be devoted to building of energy efficient low-rise house and its justification applicable to the chosen type of space-planning and structural solutions, contributing to increase of energy-efficiency, safety, comfort of living and economic effectiveness.

The area under study is presented by territory of continental climate of temperate zone, where low-rise houses are in higher demand from the point of social and economic development of newly developed territories.

The suggested methodology considers low-rise house of block-type. Low-rise multi-apartment residential building (block) is given as a two-storey building with the sizes in outer axles 9.6 m x 8.0 m, with a built-in garage (Fig.2). The number of blocks depends on the social necessity and the amount of money allocated from the budget. Space-planning solution provides standard insolation of all the premises for long-term presence of people.

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Table 1. Typology of low-rise buildings while optimal selection of space-planning and structural solutions by the criteria of safety, comfort of living, economical effectiveness of capital investments

Typology of low-rise buildings by energy efficiency criterion

typological feature, boundary / calculated value			IA	IB	IC	ID	IIA	IIB	IIC	IID	IIIA	IIIB	IIIC	IIID	IVA	IVB	IVC	IVD
			Given condition – climatic areas of construction															
Structural elements	Exterior walls	One-layered																
		Two-layered																
		Multi-layered																
	Insulant	Organic base																
		Inorganic base																
		Mixed type																
		Reflecting type																
	Façade finishing	Selection is additional																
	Translucent structures	Selection is additional																
	Foundation	Mat																
		Strip																
		Detached																
	Roof	Sloped																
		Flat																
Energy efficiency class of a building	A++																	
	A+																	
	A																	
	B+																	
	B																	
	C+																	
	C																	
	C-																	
	D																	
	E																	
By energy supply type	Centralized																	
	Locally-centralized																	
	Local																	

Typology of low-rise buildings by safety criterion

typological feature, boundary / calculated value		Given condition – wall material
		Wall material (brick, wood, light concrete, foam concrete)
By fire resistance	I	
	II	
	III	
	IV	
By structural solution	Frame	
	Frameless	
	Volume-block-type	
Service life by groups of solidity	I	
	II	
	III	
	IV	
	V	
Wear %	Up to 30	
	30–60	
	More than 60	

Typology of low-rise buildings by comfort of living criterion

typological feature, boundary / calculated value		Urban	Suburban	Rural	Remote
		Given condition – type of territory for living			
Ration of land S to total S of a building, m^2/m^2	2/1				
	4/1				
	6/1				
Number of floors	1				
	2				
	3				
The level of capital improvement, %	50%				
	75%				
	100%				
Level of lightning, lx	up to 50				
	50–100				
	100-150				
Level of noise in residential buildings, dBA	up to 30				
	30–45				
	no more than 55				

Typology of low-rise buildings by the criterion of economic effectiveness of capital investments

typological feature, boundary / calculated value		Detached	Block-type	Blocks of flats
		Given condition – type of building		
Cost 1 m^2	up to 3 minimum monthly wage			
	from 3 to 5 minimum monthly wage			
	from 5 to 7 minimum monthly wage			
	more than 7 minimum monthly wage			
Operation costs 1 m^2 per month	up to 0.2% of the average wage in region			
	up to 0.2–0.6% of the average wage in region			
	up to 0.6–1% of the average wage in region			
	higher than 1% of the average wage in region			

While designing foundations we considered the most peculiar soils of the selected region, as well as the existing experience of design, construction and maintenance of buildings in similar engineering-geological and hydro-geological conditions. For block-type low-rise houses built in temperate zone of continental climate it is reasonable to apply two variants of foundation:

1. Slightly in-ground foundations (belt-type monolithic).

2. Helical metal or bored piles.

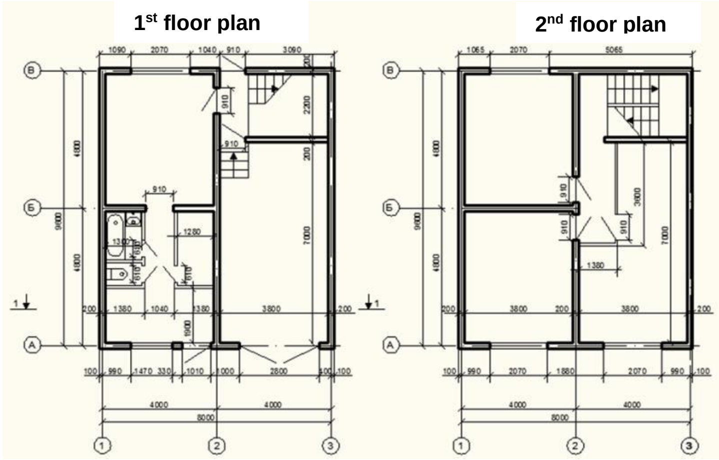

Figure 2. Space-planning solutions of low-rise block-type residential building

Thermal design of exterior building envelope is made in accordance with Russian Construction Codes: "Thermal protection of buildings. Revised edition of Construction Rules and Regulations 23-02-2003" and Construction Codes 131.13330.2012 "Construction Climatology". Outer shell is calculated as a plane wall separating air environment with different temperature and humidity, constrained by parallel surfaces and transverse to heat flow. Nominal value of reduced total thermal resistance of building envelope satisfying the requirements of sanitary-and-hygiene and comfort is calculated by Construction Codes 50.13330.2012 "Thermal protection of buildings. Revised edition of Construction Rules and Regulations 23-02-2003". As a load-bearing layer of the exterior wall the following options have been considered: ceramic brick, gas-concrete block, timber (pine-tree), wall panel from expanded clay concrete [28].

Two types of roofing can be used in the designed building: inclined and flat. Corresponding calculations have been conducted. Building materials under study are reliable and durable, correspond to the climate conditions. Variability of choice is conditioned by minimum cost of material.

Optimal technical solutions with reduced energy consumption have been justified by the method of multiple-choice design of energy-saving equipment for low-rise buildings. Comprehensive use of energy-saving building envelopes and renewable energy sources allows solving completely the problem of energy supply considering the requirements of energy efficiency, economical feasibility and comfort of living during building maintenance. In particular the schemes of energy supply have been justified, such as centralized, local and new developed locally-centralized.

Heat balance of building, heat losses for heating of infiltrating air has been defined by means of calculations. Calculation of heat losses through building envelopes has also been conducted. We defined hot water and heat expenses. Resulting from the calculations it was decided to use the combined heat supply system. The basic system is heat pump; solar collectors provide the necessary covering of expenses for hot water supply. Wind power generators cannot be used at the chosen territory of residential buildings due to restrictions of noise exposure.

Taking into account the condition of maximum energy-saving for low-rise buildings of block-type we accept horizontal heating system with placement of heat pump in the technical room. In the periods of peak loads corresponding to the periods of long-lasting low outside temperatures the conditions of comfort in the rooms do not worsen due to using diesel generator, placed in each garage of the flats. Hot water supply per one block-section requires settlement of 13 solar collectors with total surface of 26 square meters.

Ventilation system is natural. Data recording on energy resource consumption is performed automatically by means of dispatcher system.

While designing low-rise building special attention has been paid not only to financial effectiveness of construction but also to the operation effectiveness. Thus, effectiveness indicator is multi-component, as the present project considers effectiveness during construction and during further building operation. Economy during construction has been reached by means of integration of optimal design solutions enabling to increase thermal protection of building envelopes and by means of equipment (in case of absence of such equipment for heating of low-rise building we would have to build a boiler-house thus leading to additional expenses). Operation effect has been obtained by means of economy on communal public services, heating and hot water supply in particular [29].

While calculating the effectiveness of capital investments of low-rise buildings of block type per accounting period we consider costs of building materials, construction and installation works, costs of replacement or repair, service life, cost per one year of operation. Calculations of further results and expenses within one accounting period (time horizon) accepted as 30 years have been conducted. Calculation step has been accepted as a one year [30].

Results of thermotechnical calculation of building envelope, glazing and doors, roofing and flooring have been taken as a basis. Energy systems: heating system, hot water supply for domestic needs, water disposal system. Comparative results of initial capital expenses and annual expenses for building structures are given in Table 2.

Table 2. Comparison of costs of building materials of low-rise building

Material of building envelope	**Capital expenditures for building structures, rub per square meter**	
	Total cost of the building	Cost per one year of operation considering replacement or repair
1. Brick	15487	310
2. Aerated concrete	19624	392
3. Beam	13929	279
4. Expanded clay concrete	20189	404

Comparative analysis has been held for centralized, locally-centralized and local heating systems of low-rise block-type house. Comparison of cost of heat energy from different sources is given in Table 3.

Table 3. Comparison of cost of heat energy from different sources

Indicators	**Value**
Cost per 1 kW of electric energy, rubles	3.1
Cost per 1 kW of heat energy obtained from heat pump, rubles	0.4
Cost per 1 kW of heat energy obtained from gas-fired boiler, rubles	0.5
Cost per 1 kW of heat energy according to tariff of company TGC-11 (Tomsk, Russia), rubles	3.24

Tariff that is used by Public limited company TGC-11 for selling of heat is 8.1 times higher than cost for heat from heat pump and 6.48 times higher than cost from gas-fired boiler. Thus it can be seen that using heat pumps is more profitable from the point of communal expenses. Using locally-centralized system of heat supply allows reduction of heating expenses due to economy of unit of heat energy.

Utility payments have been calculated depending on the type of energy supply (Table 4).

Table 4. Comparison of utility payments by different type of energy supply per year

Indicators, rubles	Type of energy supply		
	Centralized	Locally-centralized	Local
Annual expenses for heating, rubles	548	58	72.5
Annual expenses for hot water supply, rubles	267.47	Considered in heating	37.9
Operation costs per year, rubles	20	Not required	Not required
Maintenance and seasonal operations per year, rubles	0	0	33
Total, rubles	835.47	58	143.4

Calculation results have shown that cost per one square meter of heating while using centralized heat supply is 14.4 times higher than of locally-centralized heat supply with the use of heat pumps and 5.8 times higher than while using gas-fired boiler.

Resulting from calculations of economical efficiency of block-type low-rise building the least expenses for building structures are provided by beam house. Capital expenditures for components of building structures made from wood considering replacement or repair comprise 13929 rub/m^2 per one block-section. Cost of building per one year of operation makes 279 rub/m^2 per one block-section.

Total annual economic effect is reached by using locally-centralized type of energy supply in comparison with centralized and local type in calculation per one block-section. Cost of utility services (hot water supply + heat supply) by locally-centralized type of energy supply per year makes 58 rub/m^2.

Conclusions

The system of criteria influencing the optimal selection of space-planning and structural solutions of low-rise buildings satisfying the requirements of energy-efficiency, comfort and safety, economical effectiveness has been developed. Resulting from the suggested methodology we decided to accept the option of block-type. By means of increasing thermal protection of building envelopes we get economy of energy sources during building operation.

Optimal technical solutions of energy provision of low-rise buildings of block-type have been justified. During comprehensive development of low-rise buildings of block-type at the newly developed territories which do not have access to heat network, the optimal solution is use of local type of energy supply. Increasing the level of applied technical solutions leads to increased value of capital expenditures for construction, however the effect is reached by means of economy of fuel and energy resources and social protection of the population.

Methodology of effectiveness of capital investments of block-type low-rise buildings enabled to define the operation costs per one square meter in a year. Resulting from calculations of further expenses and results within the limit of accounting year the most economically feasible is the option of residential building of block-type made from wood. Calculations demonstrate that due to heat economy, increase of lump-sum costs will pay off within 5 years.

The objectives solved within the project allowed justifying the technical capability and economical feasibility of building low-rise energy-efficient buildings of economy class. The maximum energy-saving effect can be reached during comprehensive consideration of space-planning and structural solutions and using renewable energy sources during constriction of engineering systems.

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References

1. Resolution of the Government of the Russian Federation No. 1050 from 17.12.2010 On Federal Targeted Program “Dwelling” for the Years 2015 – 2020” (edited 25.08.2015). (rus)
2. *Russian Federation. Laws. On Foundation of Assistance to Housing and Utility Sector Reformation* (updated 29 July, 2017). (rus)
3. Minaev N.N., Filushina K.E., Gusakov A.M., Zharova E.A., Gusakova N.V., Formation of a regional process management model for energy efficiency of low-rise residential construction. *Medit. J. Soc. Sci.* 2015. No. 6(3). Pp. 155–160.
4. Weizsäcker E., Hargroves K.C. and Smit M. H. *Faktor pyat'. Formula ustoychivogo rosta* [Factor Five. The Formula for Sustainable Growth: a Report to the Roman Club] Ast-Press Book, Moscow, 2013. 368 p. (rus)
5. *Energy Information Administration* (EIA). Annual Energy Outlook 2006. EIA, Washington, 2006.
6. *International Energy Agency* (IEA). Energy Technology Perspectives 2006: Scenarios and Strategies to 2050. IEA,

Литература

1. Постановление Правительства РФ № 1050 от 17.12.2010 «О федеральной целевой программе «Жилище» на 2015 - 2020 годы» (ред. от 25.08.2015).
2. Федеральный закон №85-ФЗ «О Фонде содействия реформированию жилищно-коммунального хозяйства» (ред. от 29.07.2017).
3. Minaev N.N., Filushina K.E., Gusakov A.M., Zharova E.A., Gusakova N.V., Formation of a regional process management model for energy efficiency of low-rise residential construction // Medit. J. Soc. Sci. 2015. № 6(3). Pp. 155–160.
4. Эрнст Ульрих фон Вайцзеккер, Карлсон Харгроуз, Майкл Смит. Фактор пять. Формула устойчивого роста: Доклад Римскому клубу. М.: АСТ-ПРЕСС Книга, 2013. 368 с.
5. Energy Information Administration (EIA), Annual Energy Outlook 2006. EIA, Washington, 2006.
6. International Energy Agency (IEA), Energy Technology Perspectives 2006: Scenarios and Strategies to 2050. IEA, Paris, 2006.

Paris, 2006.

7. *International Energy Agency* (IEA). Cool appliances: Policy strategies for energy efficient homes. IEA, Paris, 2003.
8. Zhou H., Lu Y., Liu X., Chang R., Wang B. Harvesting wind energy in low-rise residential buildings: Design and optimization of building forms. *J. Cleaner Prod.* 2017. No. 167. Pp. 306–316.
9. Chen X., Yang H. A multi-stage optimization of passively designed high-rise residential buildings in multiple building operation scenarios. *Appl Energy*. 2017. No. 206. Pp. 541–555.
10. Vasilyev G.P., Gornov V.F., Lichman V.A., Yurchenko I.A. A method of assessing energy consumption of buildings during commissioning. *ARPN J. Eng. Appl. Sci.* 2015. No. 10(15). Pp. 6509–6512.
11. Kornienko S.V. The experimental analysis and calculative assessment of building energy efficiency. *Applied Mechanics and Materials.* 2014. No. 618. Pp. 509–513.
12. Gorshkov A., Perlova E., Karpova S., Rakova X., Bondarenko E., Platonova M. The Architectural Concept Of The Building With Energy Consumption. *Appl. Mechan. Mater.* 2015. No. 725–726. Pp. 1580–1588.
13. Tabunschikov Yu.A., Brodach M.M. System analysis for designing energy-efficient buildings. *Architecture and Modern Information Technologies*. 2015. No. Special. P. 14. (rus)
14. Kazeikin V.S., Baronin S.A., Chernikh A.G., Androsov A.N. *Problemnyye aspekty razvitiya maloetazhnogo zhilishchnogo stroitel'stva Rossii* [Problem Aspects of Development of Lowrise Construction in Russia] Moscow: INFRA-M Publisher, 2011. 278 p. (rus)
15. Asaul A.N., Kazakov I.N., Pasiada N.I., Denisova I.V. *Teoriya i praktika maloetazhnogo zhilishchnogo stroitel'stva v Rossii* [Theory and Practice of Low-rise Residential Housing]. Petersburg: Gumanistika, 2005. 563 p. (rus)
16. Kozachun G.U., Smik O.G., Printsipy formirovaniya ob"yemno-planirovochnykh resheniy zhilykh usadebnykh domov s obsluzhivaniyem [Principles of the formation of space-planning solutions for residential manor houses with maintenance]. *Housing.* 2008. No. 7. Pp. 35–37. (rus)
17. Kalabin A.V. Tipologiya zhilykh zdaniy maloy i sredney etazhnosti: sovremennoye sostoyaniye [Typology of residential buildings of small and medium-level storeys: modern condition]. *Academic News UralNIIProject RAASN.* 2014. No. 1. Pp. 63–69. (rus)
18. Bashmakov I.A., Tipologiya i sertifikatsiya rossiyskikh zdaniy po urovnyu energoeffektivnosti [Typology and certification of Russian buildings in terms of energy efficiency]. *Association of AVOK Engineers Energy Saving*. 2015. No. 8. Pp. 50–55. (rus)
19. Vasiliev G.P., Lichman V.A., Peskov N.V. Metodika instrumental'nogo opredeleniya energopotrebleniya vvodimykh v ekspluatatsiyu zdaniy [Technique of instrumental determination of energy consumption of buildings commissioned]. *Housing.* 2014. No. 12. Pp. 32–36. (rus)
20. Sheina S.G. *Razrabotka optimizatsionnoy modeli vybora energoeffektivnykh resheniy v maloetazhnom stroitel'stve* [Development of Optimized Model for Selection of Energy Efficient Solutions in Low-rise Construction]. Rostov-on-Don: Rostov State University of Civil Engineering, 2013. 118 p. (rus)
21. Matrosov Yu.A., Butovsky I.N., Norford L.K., Opitz M.W. Standards for heating energy use in Russian buildings: a review and a report of recent progress. *Energy and Build.* 1997. No. 25(3). Pp. 207–222.
22. Gagarin V.G., Neklyudov A.Y. Improving the accuracy of the calculation of thermal capacity of heating systems

7. International Energy Agency (IEA), Cool appliances: Policy strategies for energy efficient homes. IEA, Paris, 2003.
8. Zhou H., Lu Y., Liu X., Chang R., Wang B. Harvesting wind energy in low-rise residential buildings: Design and optimization of building forms // J. Cleaner Prod. 2017. № 167. Pp. 306–316.
9. Chen X., Yang H. A multi-stage optimization of passively designed high-rise residential buildings in multiple building operation scenarios // Appl Energy. 2017. № 206. Pp. 541–555.
10. Vasilyev G.P., Gornov V.F., Lichman V.A., Yurchenko I.A. A method of assessing energy consumption of buildings during commissioning // ARPN J. Eng. Appl. Sci. 2015. № 10(15). Pp. 6509–6512.
11. Kornienko S.V. The experimental analysis and calculative assessment of building energy efficiency // Applied Mechanics and Materials. 2014. № 618. Pp. 509–513.
12. Gorshkov A., Perlova E., Karpova S., Rakova X., Bondarenko E., Platonova M. The Architectural Concept Of The Building With Energy Consumption // Appl. Mechan. Mater. 2015. № 725–726. Pp. 1580–1588.
13. Табунщиков Ю.А., Бродач М.М. Системный анализ проектирования энергоэффективных зданий // Архитектура и современные информационные технологии 2015. Спец.выпуск. С. 14.
14. Казейкин В.С., Баронин С.А., Черных А.Г., Андросов А.Н. Проблемные аспекты развития малоэтажного жилищного строительства России: Монография. М: ИНФРА-М, 2011. 278 с.
15. Асаул А.Н., Казаков Ю.Н., Пасяда Н.И., Денисова И.В. Теория и практика малоэтажного жилищного строительства в России. СПб.: «Гуманистика», 2005. 563с.
16. Козачун Г.У., Смык О.Г. Принципы формирования объемно-планировочных решений жилых усадебных домов с обслуживанием // Жилищное строительство. 2008. № 7. С. 35–37.
17. Калабин А.В. Типология жилых зданий малой и средней этажности: современное состояние // Академический вестник УралНИИпроект РААСН. 2014. № 1. С. 63–69.
18. Башмаков И.А. Типология и сертификация российских зданий по уровню энергоэффективности // АВОК Энергосбережение. 2015. № 8.
19. Васильев Г.П., Личман В.А., Песков Н.В. Методика инструментального определения энергопотребления вводимых в эксплуатацию зданий // Жилищное строительство. 2014. № 12. С. 32–36.
20. Шеина С.Г., Миненко Е.Н. Разработка оптимизационной модели выбора энергоэффективных решений в малоэтажном строительстве: монография. Ростов н/Д: Рост. гос. строит. ун-т, 2013. 118 с.
21. Matrosov Yu.A., Butovsky I.N., Norford L.K., Opitz M.W. Standards for heating energy use in Russian buildings: a review and a report of recent progress // Energy and Build. 1997. № 25(3). Pp. 207–222.
22. Gagarin V.G., Neklyudov A.Y. Improving the accuracy of the calculation of thermal capacity of heating systems when designing the buildings with high energy efficiency // International Journal for Housing Science and Its Applications. 2015. № 39(2). Pp. 79–87.
23. Gagarin V.G. Thermal performance as the main factor of energy saving of buildings in Russia // Procedia Engineering. 2016. Pp. 112–119.
24. Beregovoi A.M., Proshin A.P., Beregovoi V.A., Eremkin A.I. Unautoclaved foam concrete and its constructions adapted to regional conditions // Proceedings of the International Conference on the Use of Foamed Concrete in Construction. 2005. Pp. 113–120.

when designing the buildings with high energy efficiency. International Journal for Housing Science and Its Applications. 2015. No. 39(2). Pp. 79–87.

23. Gagarin V.G. Thermal performance as the main factor of energy saving of buildings in Russia. *Procedia Engineering.* 2016. Pp. 112–119.
24. Beregovoi A.M., Proshin A.P., Beregovoi V.A., Eremkin A.I. Unautoclaved foam concrete and its constructions adapted to regional conditions. *Proceedings of the International Conference on the Use of Foamed Concrete in Construction.* 2005. Pp. 113–120.
25. *Federal Law No.261 On Energy Saving and Enhancing Energy Efficiency and on Introduction of Changes into Definite Legislative Actions of the Russian Federation* (rus) (updated 29 July, 2017).
26. Gamtessa S.F. An explanation of residential energy-efficiency retrofit behavior in Canada. *Energy and Buildings.* 2013. No. 57. Pp. 155–164.
27. Eppler M.J., Hoffmann F., Pfister R. *Rigor and relevance in management typologies: assessing the quality of qualitative classifications 1, University of St. Gallen.* 2011. [Online] URL: www.knowledgecommunication.org. (date of application: 23.09.2017)
28. Filushina K.E., Gusakova N.V., Dobrynina O.I., Zharova E.A., Merkuleva Y.A., Runkova A.S., Minaev N.N. Regional programs on energy saving and energy efficiency: the national construction sector. *International Journal of Economics and Financial Issues.* 2016. No. 6(S5). Pp. 85–90.
29. Gusakova N., Minaev N., Filushina K.,. Gusakov A, Approaches to optimum selection of space-planning and structural solutions of low-rise buildings. *AIP Conf. Proc.* 2017. No. 1800. P. 050008.
30. Gusakova N.V., Gusakov A.M. Technical regulation in low-rise construction in the context of heat supply energy efficient projects. *IOP Conf. Series: Materi. Sci. Eng.* 2015. No. 71. P. 012050.

25. Федеральный закон №261 «Об энергосбережении и о повышении энергетической эффективности и о внесении изменений в отдельные законодательные акты Российской Федерации» (с изменениями на 29 июля 2017 года)
26. Gamtessa S.F. An explanation of residential energy-efficiency retrofit behavior in Canada // Energy and Buildings. 2013. № 57. Pp. 155–164.
27. Eppler M.J., Hoffmann F., Pfister R. Rigor and relevance in management typologies: assessing the quality of qualitative classifications 1, University of St. Gallen. 2011. [Электронный ресурс].URL: www.knowledgecommunication.org. (дата обращения: 23.09.2017)
28. Filushina K.E., Gusakova N.V., Dobrynina O.I., Zharova E.A., Merkuleva Y.A., Runkova A.S., Minaev N.N. Regional programs on energy saving and energy efficiency: the national construction sector // International Journal of Economics and Financial Issues. 2016. № 6(S5). Pp. 85–90.
29. Gusakova N., Minaev N., Filushina K., Gusakov A., Approaches to optimum selection of space-planning and structural solutions of low-rise buildings // AIP Conf. Proc. 2017. № 1800. P. 050008.
30. Gusakova N.V., Gusakov A.M. Technical regulation in low-rise construction in the context of heat supply energy efficient projects // IOP Conf. Series: Materi. Sci. Eng. 2015. № 71. P. 012050.

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Performance characteristics of differentially quenched rails

Эксплуатационные показатели дифференцированно термоупрочненных рельсов

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Key words: rails; differential quenching; wear; rate of wear; curves of small radius

Ключевые слова: рельсы; дифференцированное термоупрочнение; износ; интенсивность износа; кривые малого радиуса

Abstract. With the development of railway transport, the train speed, the traffic intensity, and the axle load all permanently increase in magnitude. This increase adversely affects the rail operation. The quantity of rail defects and, especially, the rate of rail-head surface wear both show an increase. Such service conditions require an enhanced mechanical strength of rails, their enhanced resistance to wear, and prolonged service life. In the present study, we analyze the operation performance of modern differentially quenched rails in comparison with domestic volume-quenched rails and Nippon Steel rails (Japan). For evaluating the wear resistance of rails, in the present study we carried out measurements of their side wear at curves of small radius. As a result of the study, it was found that the resistance to wear of differentially quenched DT350 rails 1.5–1.7 times exceeded the resistance of volume-quenched T1 rails. The general-purpose DT350 rails and the advanced DT370IK rails with enhanced wear resistance and enhanced contact endurance exhibited roughly identical wear rates and therefore offered equally long service periods. The Nippon Steel rails have displayed a greater resistance to wear. The side wear of those rails was found to be 1.6 times smaller than that of DT rails, and it met the normative value for surveyed curves of small radius.

Аннотация. С развитием железнодорожного транспорта скорость, интенсивность движения и нагрузка на ось возрастают. Это неблагоприятно сказывается на рельсах. Возрастает количество дефектов и особенно износ поверхности головки рельсов. Такие эксплуатационные условия требуют увеличения прочности и износостойкости рельсов, повышения их срока службы. В настоящей работе проведен анализ эксплуатационной работы новых дифференцированно термоупрочненных рельсов в сопоставлении с рельсами объемного термоупрочнения отечественного производства и японскими рельсами компании "Ниппон Стил". Для оценки износостойкости рельсов проводились измерения бокового износа головки рельсов в кривых малого радиуса. В результате исследования было установлено, что рельсы дифференцированного термоупрочнения категории ДТ350 более износостойки, чем рельсы объемного термоупрочнения категории Т1 в 1,5–1,7 раза. Рельсы общего назначения категории ДТ350 и повышенной износостойкости и контактной выносливости ДТ370ИК имели примерно одинаковую интенсивность износа и срок службы. Большей износостойкостью обладали рельсы компании "Ниппон Стил". Их интенсивность бокового износа в 1,6 раза меньше, чему у рельсов категории ДТ и соответствовала нормативному показателю для исследуемых кривых.

Introduction

The traffic safety, and the transportation continuity and efficiency at railway transport, largely depend on the state of track structure and, in particular, on the state of rails. The capital investments in rails come as the most expensive part of infrastructure fixed assets. These factors define the necessity for strengthening the requirements to rail service properties. The latter properties are in turn defined by the service life, reliability, safety, and maintainability indexes of rails.

As early as several years ago, JSC "RZD" railroads involved no world-class rails capable of meeting the company's increasing requirements to rails in terms of service life as well as in terms of the strategic development objectives and tasks of the industry.

Косенко С.А., Акимов С.С. Эксплуатационные показатели дифференцированно термоупрочненных рельсов // Инженерно-строительный журнал. 2017. № 7(75). С. 94–105.

The latter statement is corroborated by the annual withdrawal of more than 100 thousand defective and faulty rails from tracks. The main cause for this withdrawal was insufficiently high a contact fatigue strength (for more than 50 % of withdrawn rails) [1].

The situation has begun to change dramatically after 2013. The implemented grand-scale reconstruction, and technical re-equipment and modernization of rail production at JSC "EVRAZ ZSMK" and JSC "Mechel" have allowed the companies to master state-of-the-art technologies. A key point in the realization of the implemented technology has become the production of 100-m long differentially quenched rails. The quality and characteristics of the new rails fully meet the European requirements [2] and the Russian national standards GOST R 51685-2013 [3].

A weak point of the previous rail manufacture technology was considerable residual internal stresses occurring in the rails. The occurrence of such stresses is related with imperfect rail quenching, which process defines the mechanical and working characteristics of rails [4]. Considerable hardening strains emerge in rails during the volume quenching of rails in oil implemented in drum hardeners. Subsequent rail strengthening leads to the emergence of considerable residual stresses [1, 5].

A.D. Konyukhov and E.A. Shur [6] have shown that the increase of residual tensile stresses to 150 MPa from 0-MPa conventional level leads in R65 rails to a reduction of the number of cycles to crack formation by a factor of 2.7, and to a reduction of the number of cycles to fracture by crack development by a factor of 4.

The rail differential quenching technology using compressed air is free of this drawback. In the post-quench cooling process, there exists a possibility to regulate both the airflow rate and air pressure in the rail-head and rail-base quenches. In this way, the rail buckling can be minimized (i.e., reduced by a factor of 4.3).

The reliability indexes of rails fabricated by the new technology proved to be 50 % increased. Indeed, whereas the rails manufactured by the previous technologies were capable of ensuring an 80-% gamma resource on handling the freight volume of 500 mln gross tons, the new rails has proven to be capable of ensuring a 85-% gamma resource on handling the freight volume of 750 mln gross tons. Field tests performed by JSC "VNIIZhT" have shown that the failure-free load life of the new rails exceeded 600 mln tons, this value being 2.5 times greater than the failure-free load life of volume-quenched rails [7].

To date, more than 1.5 million tons of DT rails have been already supplied to the JSC "RZD" railroad network [8].

The laying of new DT rails, including their laying at the West-Siberian Infrastructure Directorate (WSID), began in 2014. Since then, the length of laid rails has reached 751.6 km; this value presently amounts to 8 % of the total length of main lines. The annually laid volumes of DT rails are shown in Figure 1.

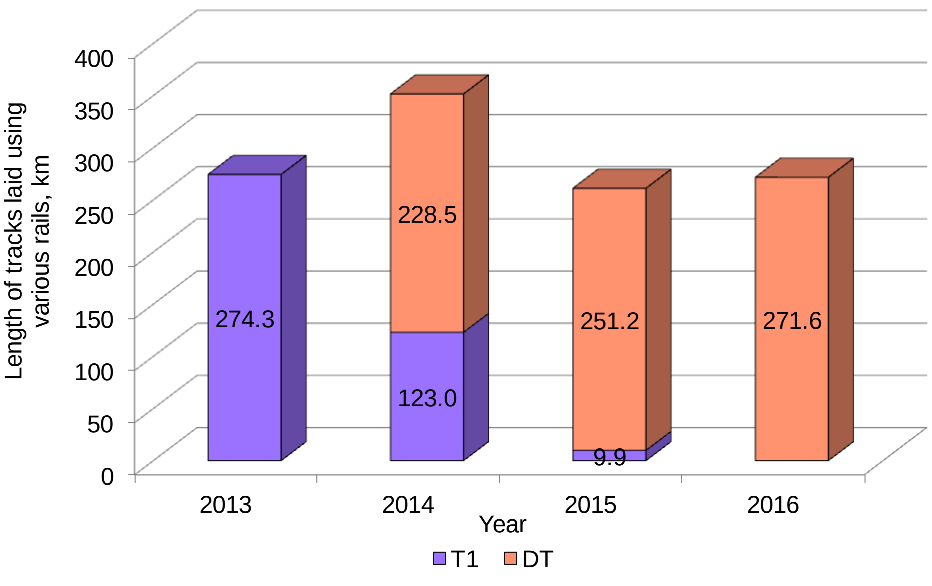

Figure 1. Annually laid volumes of T1 and DT rails at WSID

Of this volume, 646.4 km of track rails were rails replaced during track modernization and continuous rail replacement accompanied with mid-level track maintenance works. During the target replacement of high rails on curved tracks, 105.2 km of track rails were laid.

A diagram illustrating the laying of DT350 rails of total length 741.4 km at the WSID polygon with breakdown by curve radii is shown in Fig. 2. A main portion (84 %) of those rails was laid in straight track sections and curved track sections of radius 650 m and over.

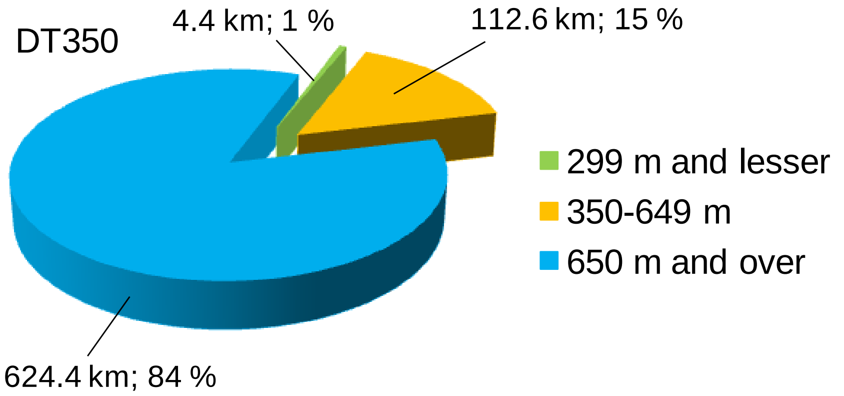

Figure 2. Laid volumes of DT350 rails versus curve radius at WSID

In 2015, the laying of DT370IK rails with enhanced resistance to wear has begun. However, a low fraction of special-purpose rails deserves mention. To date, 9.8 km of track rails was laid, this length amounting to 1.5 % of the total length of the DT rails laid. The laying of DT rails was performed by a target program in curved track sections (Fig. 3).

From Figure 3, it is seen that most of rails with enhanced resistance to wear (more than 60 %) were laid in the range of curve radii from 350 to 649 m.

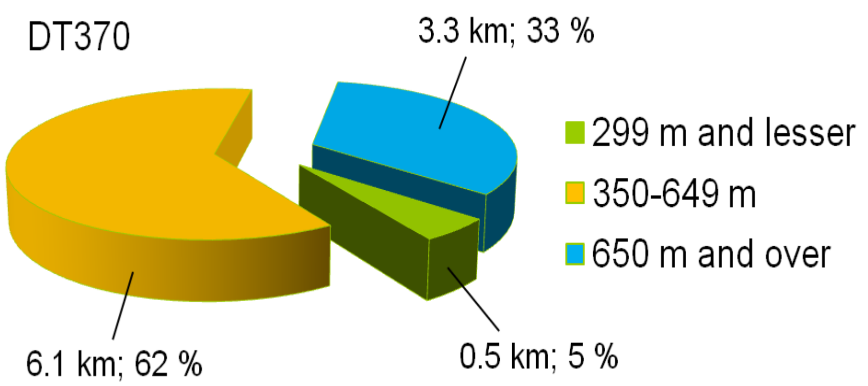

Figure 3. Laid volumes of DT370IK rails versus curve radius at WSID

With the passage from volume-quenched to differentially quenched rails, a possibility to evaluate the difference between the new rails in terms of their quality and functional reliability has emerged. The wear of the wheel–rail system is one of the most high-cost problems for railroads [9–12] and, in particular, for track facilities. This circumstance is primarily related with railway traffic safety and maintenance planning [13].

From the tribilogical standpoint, the rail wear is defined as any damage caused to rails related with the gradual loss or displacement of materials from the wheel-rail interface [14].

Numerous studies have shown that the wheel/rail wear depends on the slip along and, especially, across, the direction of wheel rolling on the rail, on the track curvature, on the load exerted on rail, on the train speed, on dynamic actions, on wheel/rail material properties, and on the environmental conditions and contaminations [5, 15–17].

In this connection, the purpose of the present study was an evaluation and performing a comparison of the operation performance of differentially quenched rails with that of volume-quenched rails and Nippon Steel rails (Japan) in curves of small radius.

The following tasks were posed:

1. Choice of reference track sections laid with rails of different types yet functioning under similar operating conditions (curve radius, track layout);

2. Measurement of rail side wear in round curves and determination of the rate of this wear;

3. Plotting the dependences of side wear values and side wear rates on the hauled gross tonnage with single-factor regression equation.

4. Prediction of the service lives of the various rails with evaluation of the service durability of differentially quenched rails.

Methods

For performing a comparison between T1, DT350, DT370IK, and Nippon Steel rails, reference track sections in curves of radii 350 to 390 m between the Izdrevaya and Sokur station, 1-st main line, were put under observation. For evaluating the rail service lives, rail-head side wear measurements were performed. The side wear comes as the main factor to limit the service life of rails at small-radius curves [18]; the mechanism of this limitation depends on many other factors [19].

The measurements were carried out over the entire length of round curves in 4-m steps along the thrust line. For excluding measurement inaccuracies, the measurement stations were marked with white paint on rail web.

For determining the rail side wear and side wear rate, initially we measured and evaluated the actual mean width of the rail head of new rails laid in curved track sections. This was made in connection with the rail production tolerance (± 0.5 mm).

The difference between the measured rail head width of a new and worn-out rail gives the rate of side wear:

$$\Delta B = B - b_1, \tag{1}$$

where ΔB is the rail side wear rate, B is the rail head width of a new rail at a distance 13 mm from the tread surface, and b_1 is the rail head width of the worn-out rail at a distance 13 mm from this surface.

From measured data, the average rail side wear over the whole length of the round curve was determined:

$$\Delta B_{avg} = \frac{\sum_{i=1}^{n} \Delta B_i}{n}, \tag{2}$$

where ΔB_{avg} is the mean rail side wear value, $\sum_{i=1}^{n} \Delta B_i$ is the sum of measured rail side wear values over the entire round curve, and n is the total number of performed measurements.

Given the tonnage hauled over the period from rail laying to the time of regular side wear measurement, we can determine the side wear rate:

$$\beta = \frac{\Delta B_{avg}}{T}, \tag{3}$$

where β is the side wear rate, T is the tonnage hauled over the period from rail laying in the curve to the side wear measurement.

The rail-head width measurements were performed with a "Puteets" sliding caliper and an SKIG-1 rail-head wear meter at a distance of 13 mm from the treed surface.

Results and Discussion

Some characteristics of the first reference track on which the operation of T1 and DT350 rails was monitored are listed in Table 1.

Table 1. Characteristics of T1- and DT350-rail reference tracks No. 1

Stage, line	**Izdrevaya – Sokur, 1-st main line**
Kilometers	17 km Hundred-Meter Mark 6 – 18 km Hundred-Meter Mark 4
Gross freight traffic, mln tons/ km per year	78.2
Curve radius, m	353
Track profile, promille	10.5
Elevation of outer rail in the curve, mm	90
Velocity of passenger/freight trains, km/hour	0/60

Based on the obtained experimental data, we have plotted diagrams of side wear (Fig. 6) and side-wear rate (Fig. 7) with their respective approximating functions.

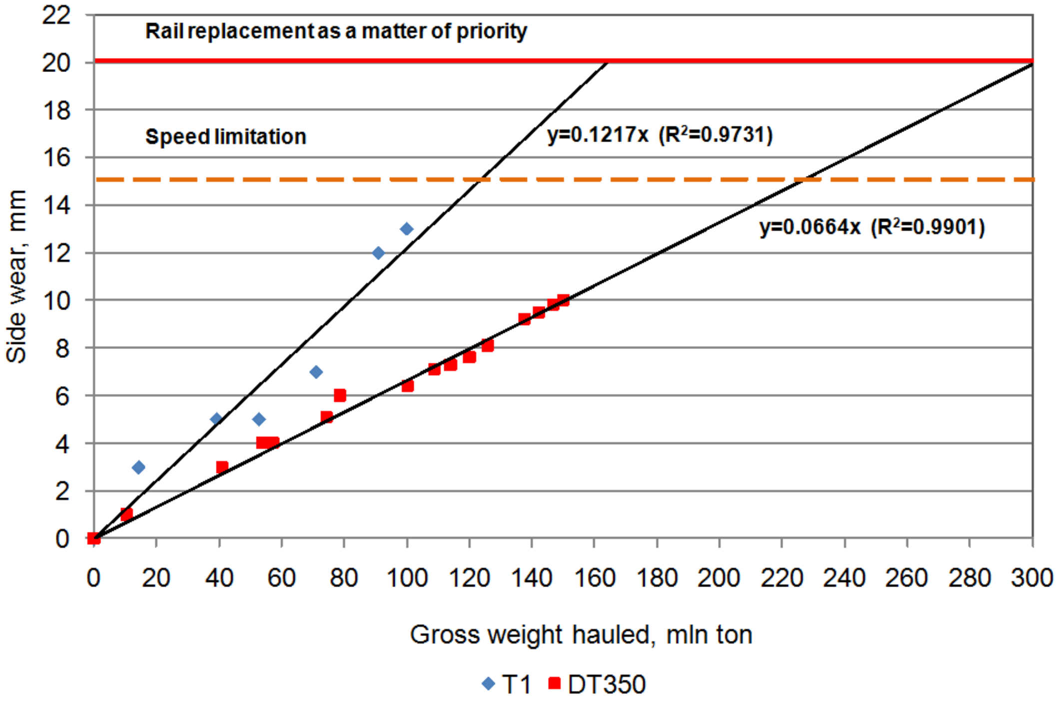

Figure 6. The side wear of T1 and DT350 rails in a curve of 353-m radius (high rails)

For the hauled gross tonnage of 100.1 mln tons, the T1 rails were found to exhibit a 13-mm side wear. After the same hauled tonnage, the wear of DT350 rails has proven to be 6.4 mm, this value being 51 % smaller than that of T1 rails.

A linear approximation to the rail side wear values yields a prediction that the load life of T1 and DT350 rails to the formation of 15-mm wear will amount respectively to 123.3 and 225.9 mln gross tons.

At rail-head side wear values of R65 rails exceeding 15 mm, the train speed is to be restricted to 70 km/hour at curve radii in excess of 350 m and to 50 km/hour at curve radii of 350 m and smaller [20]. Moreover, the re-laying of rail bars with rail edge alteration is to be made on the condition that the maximum side wear does not exceed 15 mm [21].

The rail load life to the formation of 20-mm side wear, over which the rail is to be considered defective and needing to be replaced as a matter of priority [20] is 164.3 mln gross tons for T1 rails and 301.2 mln gross tons for DT350 rails. Thus, the service life of DT350 rails in the surveyed curve is expected to be 1.8 times longer than that of T1 rails.

The mean side wear rate of T1 rails proved to be 0.113 mm/mln gross tons. The same characteristic of DT350 rails proved to be lower, equal to 0.065 mm/mln gross tons. On the average, the side wear rate of DT350 rails on the surveyed curve was 42 % lower than that of T1 rails.

The wear of DT350 rails has proven to be more uniform. The mechanisms of the wear during the breaking-in of wheel with a new rail differ from the mechanisms of wear at the main operation stage of the wheel-rail pair. Right after the laying of the rails, a high rate of their side wear is observed. After the transmission of 10–15 mln gross tons, the wear rate reaches saturation due to the breaking-in of rails with stock wheels with the formation of a conformal contact.

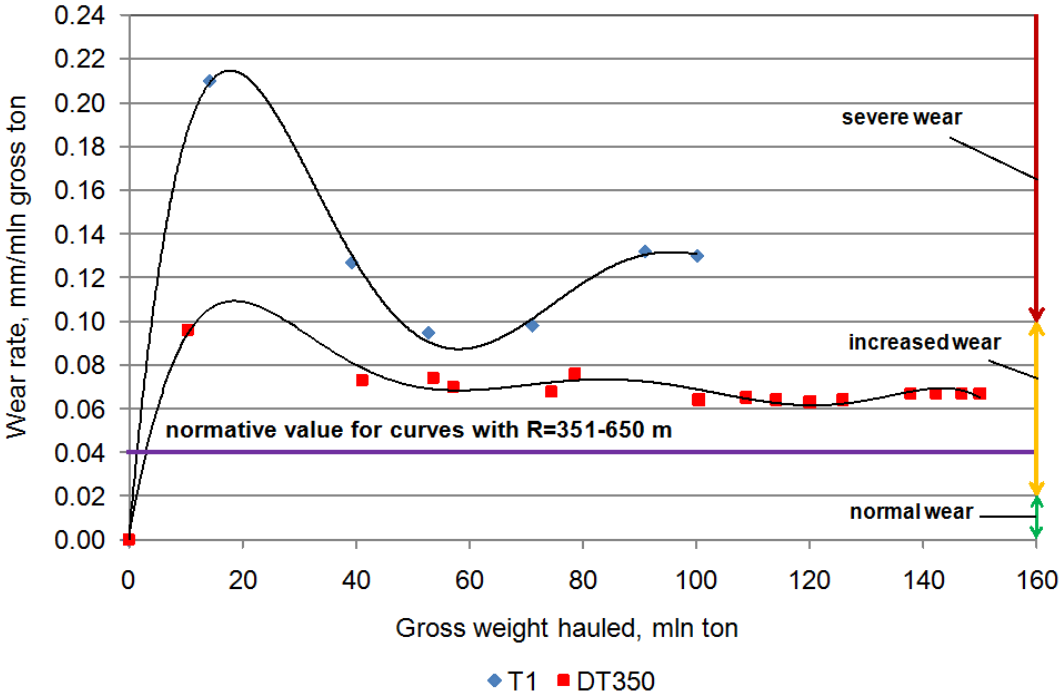

Figure 7. The side wear rate of T1 and DT350 rails in a curve of 353-m radius

The characteristics of the second reference track on which performance characteristics of T1 and DT350 rails were monitored are listed in Table 2.

Table 2. Characteristics of T1- and DT350-rail reference tracks No. 2

Stage, line	Izdrevaya – Sokur, 1-st main line
Kilometers	18 km Hundred-Meter Mark 6 – 19 km Hundred-Meter Mark 5
Gross freight traffic, mln tons/ km per year	78.2
Curve radius, m	390
Track profile, promille	4.7
Elevation of outer rail in the curve, mm	85
Velocity of passenger/freight trains, km/hour	0/60

On the basis of obtained data, similar diagrams of side wear and side-wear rate were plotted; those diagrams are shown in Figs. 8 and 9.

On the second curve of radius 390 m, at identical hauled tonnages the DT350 rails have also shown a side wear smaller than that of T1 rails. For instance, at a hauled gross tonnage of 123.0 mln tons a side wear of 12.0 mm was registered for T1 rails, whereas the DT350 rails have displayed a 10-mm wear, the latter value being 17 % lower than the former wear.

On this curve, the predicted load life to 15-mm rail-head side wear is 140.1 mln gross tons for T1 rails and 195.8 mln gross tons for DT350 rails. The predicted rail load life to the formation of 20-mm critical side wear is 186.7 mln gross tons for T1 rails and 261.1 mln gross tons for DT350 rails. Thus, the service life of DT350 rails in the surveyed curve is 1.4 times longer than that of T1 rails. The latter factor is somewhat smaller than the factor value obtained on the first reference curve.

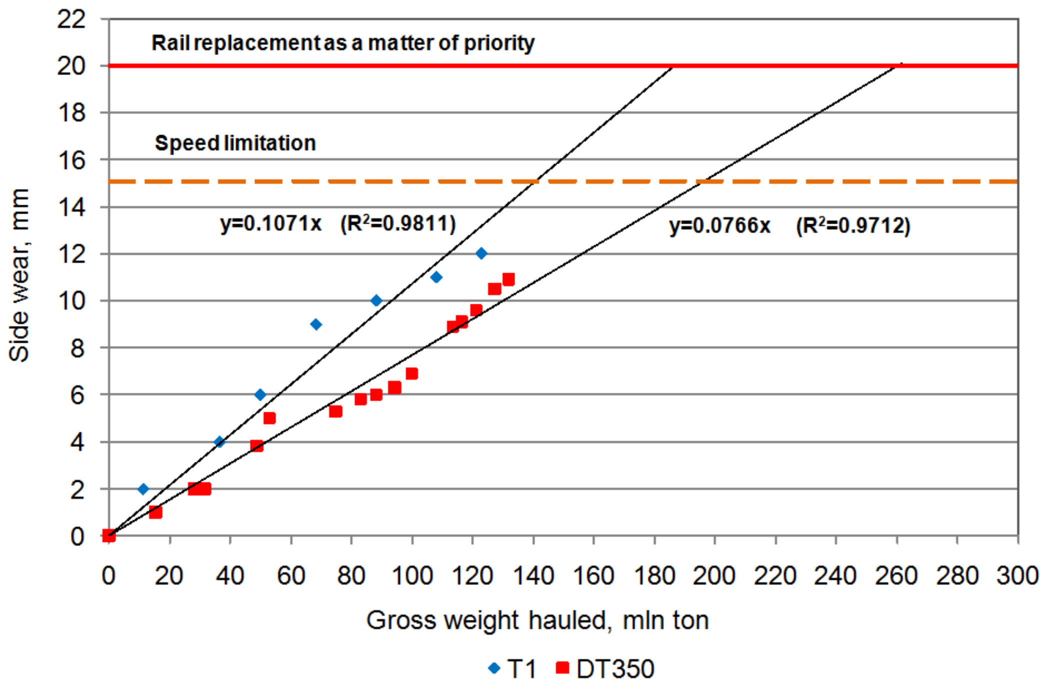

Figure 8. The side wear of T1 and DT350 rails in a curve of radius 390 m (high rails)

The mean side wear rate of T1 rails was found to be equal to 0.106 mm/mln gross tons. For DT350 rails, the same characteristic proved to be 0.070 mm/mln gross tons. The mean rate of wear of DT350 rails on the surveyed curve of radius 390 m and smaller is 34 % lower than that of T1 rails.

On both curves, of radii 353 and 390 m, the volume-quenched T1 rails exhibited a severe (by the classification of [5]) wear of the side surface of high rails. Here, the wear rate exceeds 10 mm/mln gross tons (Figs. 7 and 9). The wear of differentially quenched rails is classified as an increased one, corresponding to the interval of 2 to 10 mm/mln gross tons.

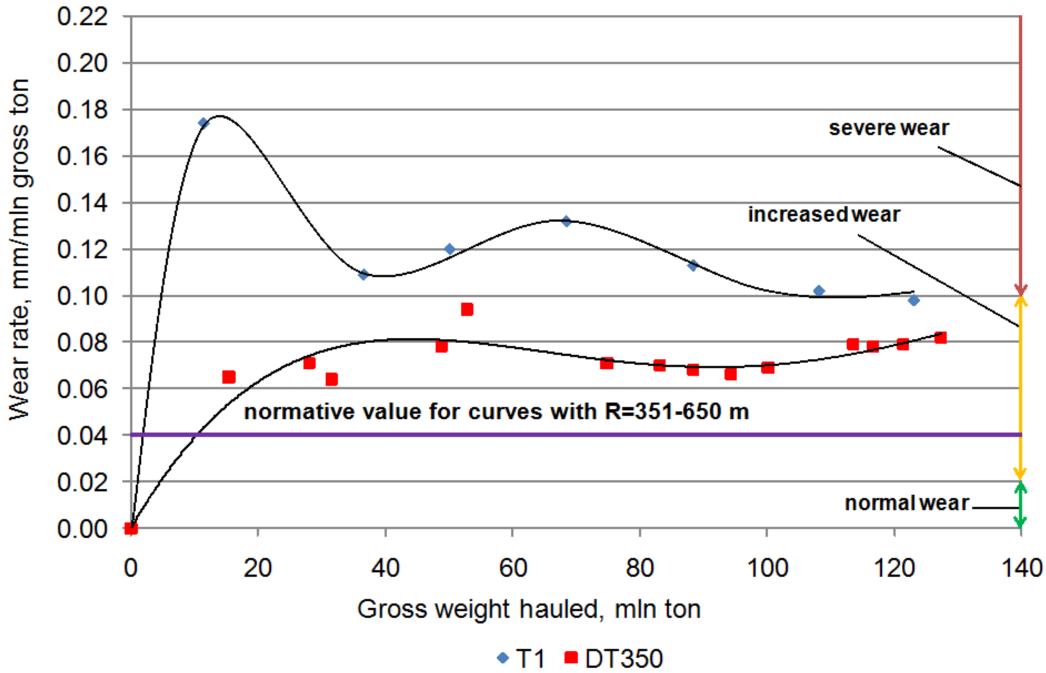

Figure 9. The side wear rate of T1 and DT350 rails in a curve of 390-m radius

For comparison of domestic general-purpose DT350 rails and domestic special-purpose DT370IK rails with Nippon Steel rails, we have monitored the operation of the rails on curved track sections with similar characteristics (Table 3).

Table 3. Characteristics of reference DT350- and DT370IK-rail tracks

Stage, line	**Izdrevaya – Sokur, 1-st main line**	
Kilometers	18 km Hundred-Meter Mark 6 – 19 km Hundred-Meter Mark 5	26 km Hundred-Meter Mark 6 – 27 km Hundred-Meter Mark 4
Gross freight traffic, mln tons/ km per year	78.2	
Rail category	DT350	DT370IK, Nippon Steel
Rail fastening	KB-65	
Curve radius, m	390	392
Track profile, promille	4.7	7.7
Elevation of outer rail in the curve, mm	85	87
Velocity of passenger/freight trains, km/hour	0/60	

Using the measured values of the rail-head widths for the various rails, diagrams of side wear and side-wear rate were plotted (see Figs. 10 and 11).

From the side-wear diagram of Fig. 10, it is seen that, at identical hauled tonnages, the general-purpose DT350 rails and the advanced DT370IK rails with enhanced wear resistance and enhanced contact endurance exhibited roughly identical values of side wear.

The Nippon Steel rails possess a greater resistance to wear. For the hauled tonnage of 105.8 mln gross tons, the side wear of DT370IK rails proved to be 8.30 mm. For the hauled tonnage of 107.6 mln gross tons, the side wear of Nippon Steel rails was found to be 3.97 mm, the latter value being twice smaller than the wear of DT370IK rails.

By analyzing the diagrams, one can compare the service life periods of the Russian and Japanese rails laid in the surveyed curves. The obtained values are summarized in Table 4.

The Japanese rails offer a longest service life. Before the emergence of 20-mm critical side wear, those rails can pass 413.2 mln gross tons, the latter weight being 1.6 times heavier than the weights that can be passed by DT350 and DT370IK rails.

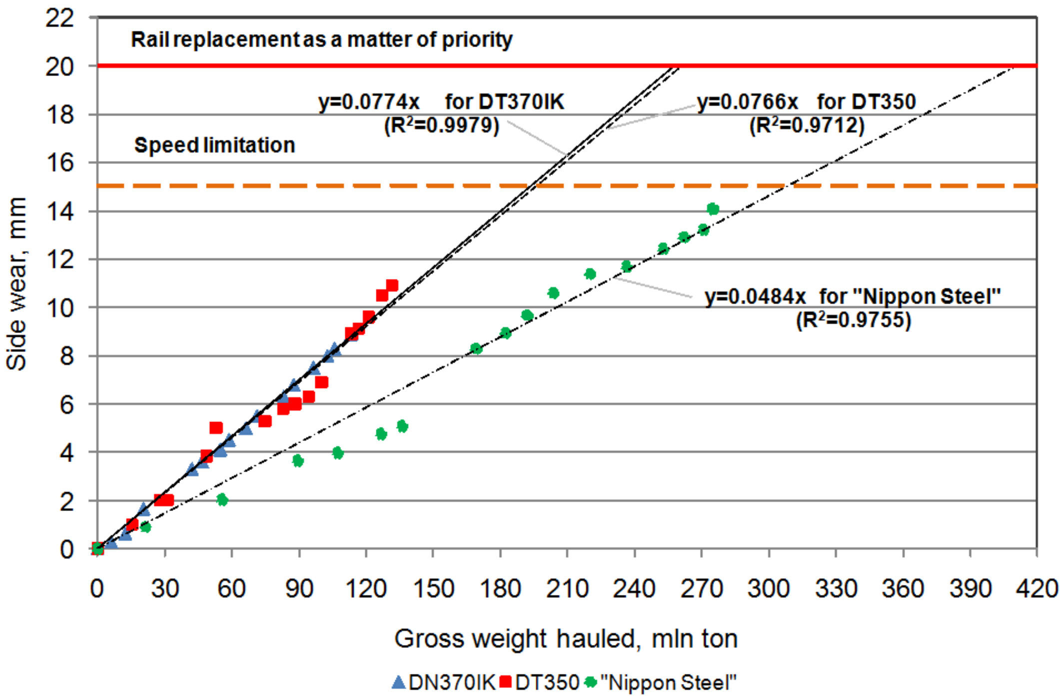

Figure 10. The side wear of DT350, DT370IK, and Nippon Steel rails on curves of 390- and 392-m radii

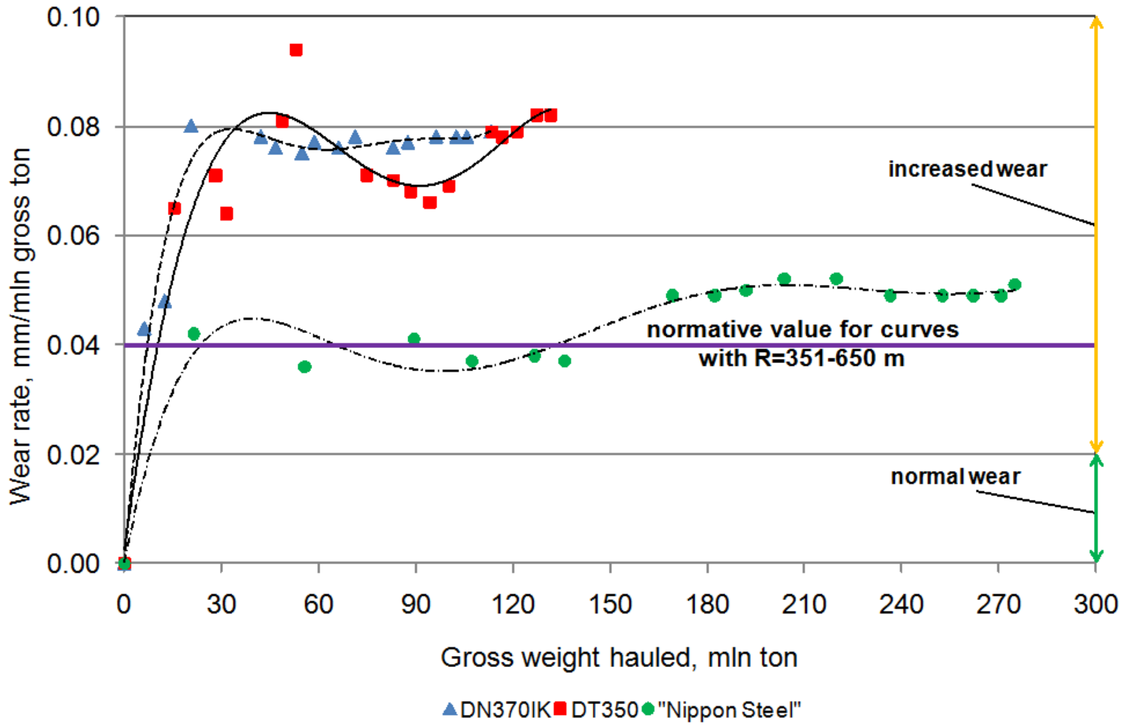

Figure 11. The side wear rate of DT350, DT370IK, and Nippon Steel rails on curves of 390- and 392-m radius

Table 4. Service life of Russian and Japanese rails for a curve of 390-m radius

Rail category	Mean side wear rate, mm/ mln gross tons	Load life, mln gross tons	
		to 15-mm side wear	to 20-mm side wear
DT350	0.070	195.8	261.1
DT370IK	0.069	193.8	258.4
Nippon Steel	0.043	309.9	413.2

The mean side wear rate of DT350 rails proved to be 0.070 mm/mln gross tons, and that of DT370IK rails, 0.069 mm/mln gross tons. It should be noted here that DT370IK rails are special-purpose ones, intended for laying heavy-traffic lines involving a multitude of small-radius curves. Following the transmission of 15 mln gross tons, those rails exhibit a more uniform rate of side wear.

The Nippon Steel rails also exhibit a uniform rate of side wear of their heads over the entire service life period. The mean side wear rate of Nippon Steel rails proved to be 0.043 mm/mln gross tons. Like for DT350 and DT370IK rails, the latter wear is classified as an increased wear. Yet, this indicator proved to be most closely meeting the rail quality standard in terms of side wear rate, 0.04 mm/mln gross tons, as recommended by JSC "RZD".

Prolongation of the service life of rails and other superstructure components presents an important strategic task for JSC "RZD" [22]. Ways toward the accomplishment of this task have been identified. This will allow prolongation of inter-repair periods and a reduction of the company operating costs.

A significant step toward the prolongation of the service life period of rail steels is the enhancement of rail-steel quality and the passage to long-length rails. Presently, the Central Infrastructure Directorate of JSC "RZD" has several times increased the order for 100-meter rails. Such rails were laid to form about five thousand kilometers of track.

For prolongation of rail service life, the current trends dictate further increase of rail resistance to wear.

Also, it is required to implement an optimal rate of rail side wear using lubrication and periodic rail grinding [23, 24]. This will exclude a necessity of early rail replacement by excessive side wear and, simultaneously, suppress the contact fatigue cracking [5, 15].

The desired value of the resistance of rails to wear can be obtained by raising the hardness of rail materials [25]. In view of the fact that the possibilities offered by pearlitic metal structure have proven to be nearly exhausted, it would be advantageous to use rails with the bainitic structure. Bainitic rail steels were examined in comparison with pearlitic rail-steel structures by E.A. Shur [5], H. Yokoyama [26], and by other workers. Bainitic rail steels exhibit higher fatigue-strength values, and they show fracture-toughness values nearly two times increased in comparison with pearlitic rail steels [10].

The use of bainitic steels in the production of rails with a load life of up to 2 bln gross tons was prospected in [22]. In combination with the differential quenching technology, this will enable the production of rails with high service durability.

Besides, a substantial contribution to the prolongation of the service life of rails and that of the whole track superstructure will be gained due to JSC "RZD" measures aimed at the improvement and prolongation of the guaranteed load life of spring rail fastening components to 1.5 bln gross tons.

Conclusions

From the performed study, the following conclusions can be drawn:

1. The modern differentially quenched rails possess a better service durability in comparison with volume-quenched rail and a lower service durability in comparison with Nippon Steel rails: the side wear rate of DT350 and DT370IK rails is 1.5 – 1.7 times lower than that of T1; yet, it is more than 1.5 times higher than the side wear rate of the Japan rails.
2. The load life of the differentially quenched rails to the formation of the maximally admissible 20-mm side wear is 29 % (in a curve of 353-m radius) and 45 % (in a curve of 390-m radius) greater than that of the T1 rails; simultaneously, it 27 % (in a curve of 353-m radius) and 36 % (in a curve of 390-m radius) lower than the load life of strings welded from the Japan rails.
3. In curved tracks, the T1 rails exhibit an extremely gross wear (in excess of 10 mm/mln gross tons), whereas the DT350, DT370IK, and Nippon Steel rails, an accelerated wear (2–10 mm/mln gross tons).
4. In a curve of 390-m radius, the general-purpose DT350 rails and the advanced DT370IK rails with enhanced wear resistance and enhanced contact endurance have demonstrated roughly identical wear rates.

References

1. Borts A.I. Issledovaniya innovatsionnoi rel'sovoi produktsii [Research of innovative rail products]. *Railway Transport*. 2015. No. 8. Pp. 54–58 (rus).
2. *EN 13674-1:2011* "Railway applications - Track - Rails - Part 1: Vignole railway rails 46 kg/m and above", NEQ.
3. *GOST R 51685-2013.* Natsional'nyi standart Rossiiskoi Federatsii. Rel'sy zheleznodorozhnye. Obshchie tekhnicheskie usloviya [Russian State Standard GOST R 51685-2013. Railway rails. General specifications]. Moscow: Standartinform, 2014. 95 p. (rus)
4. Gromov V.E., Yur'ev A.B., Morozov K.V., Ivanov Yu.F., Alsaraeva K.V. Structure, phase composition, and defect substructure of differentially quenched rail. *Steel in Translation.* 2014. Vol. 44. No. 12. Pp. 883–885.
5. Shur E.A. *Povrezhdeniya rel'sov* [Rail damage]. M.: Intekst, 2012. 192 p. (rus).
6. Konyukhov A.D., Shur E.A. Vliyanie ostatochnykh napryazhenij v zakalennykh rel'sah na vozniknovenie i rasprostranenie ustalostnykh treshchin pri tsiklicheskom izgibe [Influence of residual stresses in hardened rails on the occurrence and propagation of fatigue cracks in cyclic bending]. *Ostatochnye napryazheniya i prochnost' zheleznodorozhnykh rel'sov: Proceedings of CNII MPS*, Moscow: Transport, 1973. No. 491. Pp. 29–36. (rus).
7. Palkin S.V. Rel'sovaya produktsiya – osnova bezopasnosti i effektivnosti perevozok [Rail products as the basis of safety and efficiency of transportation]. *Railway Track and Facilities.* 2016. No. 5. Pp. 22–25. (rus)
8. Lisitsyn A.I. Kuznetsov I.A. Mart'yanov Yu.A. Analiz ekspluatatsii rel'sov na seti dorog Rossii i perspektivy povysheniya ikh nadezhnosti [Analysis of the operation of rails on the road network of Russia and the prospects for

Литература

1. Борц А.И. Исследования инновационной рельсовой продукции // Железнодорожный транспорт. 2015. № 8. С. 54–58.
2. ЕН 13674-1:2011 "Железные дороги. Путь. Рельсы. Часть 1. Рельсы Виньоля 46 кг/м и более.
3. ГОСТ Р 51685-2013. Национальный стандарт Российской Федерации. Рельсы железнодорожные. Общие технические условия. М.: Стандартинформ, 2014. 95 с.
4. Gromov V.E., Yur'ev A.B., Morozov K.V., Ivanov Yu.F., Alsaraeva K.V. Structure, phase composition, and defect substructure of differentially quenched rail // Steel in Translation. 2014. Vol. 44. № 12. Pp. 883–885.
5. Шур Е.А. Повреждения рельсов. М.: Интекст, 2012. 192 с.
6. Конюхов А.Д., Шур Е.А. Влияние остаточных напряжений в закаленных рельсах на возникновение и распространение усталостных трещин при циклическом изгибе // Остаточные напряжения и прочность железнодорожных рельсов: сб. тр. ЦНИИ МПС. М.: Транспорт, 1973. № 491.С. 29–36.
7. Палкин С.В. Рельсовая продукция – основа безопасности и эффективности перевозок // Путь и путевое хозяйство. 2016. № 5. С. 22–25.
8. Лисицын А.И. Кузнецов И.А. Мартьянов Ю.А. Анализ эксплуатации рельсов на сети дорог России и перспективы повышения их надежности // Путь и путевое хозяйство. 2016. № 12. С. 6–8.
9. Elyasi M, Hadinezhad M., Rajabi M., Abbasi M. Study on influence of effective parameters on wear behavior of wheel and rails through ADAMS-RAIL software // Manufacturing Science and Technology. 2015. Vol. 3(5).

improving their reliability]. *Railway Track and Facilities*. 2016. No. 12. Pp. 6–8. (rus)

9. Elyasi M, Hadinezhad M., Rajabi M., Abbasi M. Study on influence of effective parameters on wear behavior of wheel and rails through ADAMS-RAIL software. *Manufacturing Science and Technology*. 2015. Vol. 3(5). Pp. 308–315.
10. Zhai W., Gao J., Liu P., Kaiyun Wang K. Reducing rail side wear on heavy-haul railway curves based on wheel–rail dynamic interaction. *Vehicle System Dynamics.* 2014. Vol. 52. Pp. 440–454.
11. Santa J.F, Toro A., Lewis R. Correlations between rail wear rates and operating conditions in a commercial railroad. *Tribology International.* 2016. Vol. 95. Pp. 5–12.
12. Areiza Y.A., Garcés S.I., Santa J.F., Vargas G., Toro A. Field measurement of coefficient of friction in rails using a hand-pushed tribometer. *Tribology International.* 2015. Vol. 82. Part B. Pp. 274–279.
13. Ignesti M., Malvezzi M., Marini L., Meli E., Rindi A. Development of a wear model for the prediction of wheel and rail profile evolution in railway systems. *Wear.* 2012. Vol. 284–285. Pp.1–17.
14. Nia S.H., Carlos Casanueva C., Stichel S. Prediction of RCF and wear evolution of iron-ore locomotive wheels. *Wear.* 2015. Vol. 338–339. Pp. 62–72.
15. Famurewa S.M., Asplund M., Kumar U. Evaluation of rail wear characteristics on heavy haul track section using measurement data. *International Heavy Haul Association: The 11-th International Heavy Haul Association Conference.* 2015. Pp. 536–543.
16. Grassie S. Traction, curving and surface damage of rails, Part 2: Rail damage. *Journal of Rail and Rapid Transit.* 2014. Vol. 229(3). Pp. 330–339.
17. Wang W.J., Lewis R., Yang B., Guo L.C., Liu Q.Y., Zhu M.H.. Wear and damage transitions of wheel and rail materials under various contact conditions // Wear. 2016. Vol. 362-363. pp. 146–152.
18. Zhu Y., Sundh J., Olofsson U. A Tribological view of wheel-rail wear maps. *International Journal of Railway Technology.* 2013. Vol. 2. No. 10. Pp. 888–888.
19. Lye Y., Zhu Y., Olofsson U. Wear between wheel and rail: A pin-on-disc study of environmental conditions and iron oxides. *Wear.* 2015. Vol. 328–329. Pp. 277–285.
20. *Instruktsiya "Defekty rel'sov. Klassifikatsiya, katalog i parametry defektnykh i ostrodefektnyh rel'sov"* [Instruction manual "Defects of rails: classification, catalog and parameters of defective and highly defective rails"]: utv. rasporyazheniem JSCo "RZD" 23.10.2014 No. 2499r. (rus).
21. *Instruktsiya po ustrojstvu, ukladke, soderzhaniyu i remontu besstykovogo puti* [Instruction manual for the installation, laying, maintenance and repair of continuous welded paths]: utv. rasporyazheniem JSCo "RZD" 14.12.2016 No. 2544. (rus)
22. *Strategiya razvitiya zheleznodorozhnogo transporta v Rossiiskoi Federatsii do 2030 goda* [Strategy for the development of rail transport in the Russian Federation until 2030]: utv. rasporyazheniem Pravitel'stva RF ot 02.04.2014 No. 503-r // Sobranie zakonodatel'stva RF. 2008. No. 29 (Part. II). St. 3537. (rus)
23. Marich S. Rail grinding strategies adopted in Australia. *Rail Engineer International.* 2005. Vol. 9 (4). Pp. 4–6.
24. Zarembski A.M. The evolution and application of rail profile grinding. *Bulletin of the American Railway Engineering Association.* 1988. Bulletin 718. Vol. 89. Pp. 149–168.
25. Takahashi J., Kobayashi Y., Ueda M., Miyazaki T., Kawakami K. Nanoscale characterization of rolling contact wear surface of pearlitic steel. *Materials Science and Technology.* 2013. Vol. 10(29).
26. Yokoyama H., Mitao S., Yamamoto S., Kataoka Y., Sugiyama T. High strength bainitic steel rails for heavy haul

Pp. 308–315.

10. Zhai W., Gao J., Liu P., Kaiyun Wang K. Reducing rail side wear on heavy-haul railway curves based on wheel–rail dynamic interaction // Vehicle System Dynamics. 2014. Vol. 52. Pp. 440–454.
11. Santa J.F, Toro A., Lewis R. Correlations between rail wear rates and operating conditions in a commercial railroad // Tribology International. 2016. Vol. 95. Pp. 5–12.
12. Areiza Y.A., Garcés S.I., Santa J.F., Vargas G., Toro A. Field measurement of coefficient of friction in rails using a hand-pushed tribometer // Tribology International. 2015. Vol. 82. Part B. Pp. 274–279.
13. Ignesti M., Malvezzi M., Marini L., Meli E., Rindi A. Development of a wear model for the prediction of wheel and rail profile evolution in railway systems // Wear. 2012. Vol. 284–285. Pp.1–17.
14. Nia S.H., Carlos Casanueva C., Stichel S. Prediction of RCF and wear evolution of iron-ore locomotive wheels // Wear. 2015. Vol. 338–339. Pp. 62–72.
15. Famurewa S.M., Asplund M., Kumar U. Evaluation of rail wear characteristics on heavy haul track section using measurement data // International Heavy Haul Association: The 11th International Heavy Haul Association Conference. 2015. Pp. 536–543.
16. Grassie S. Traction, curving and surface damage of rails, Part 2: Rail damage // Journal of Rail and Rapid Transit. 2014. Vol. 229(3). Pp. 330–339.
17. Wang W.J., Lewis R., Yang B., Guo L.C., Liu Q.Y., Zhu M.H.. Wear and damage transitions of wheel and rail materials under various contact conditions // Wear. 2016. Vol. 362–363. Pp. 146–152.
18. Zhu Y., Sundh J., Olofsson U. A Tribological View of Wheel-Rail Wear Maps // International Journal of Railway Technology. 2013. Vol. 2. № 10. Pp. 888–888.
19. Lye Y., Zhu Y., Olofsson U.. Wear between wheel and rail: A pin-on-disc study of environmental Conditions and Iron Oxides // Wear. 2015. Vol. 328-329. Pp. 277–285.

20. Инструкция "Дефекты рельсов. Классификация, каталог и параметры дефектных и остродефектных рельсов": утв. распоряжением ОАО "РЖД" 23.10.2014 № 2499р.
21. Инструкция по устройству, укладке, содержанию и ремонту бесстыкового пути: утв. распоряжением ОАО "РЖД" 14.12.2016 № 2544.
22. Стратегия развития железнодорожного транспорта в Российской Федерации до 2030 года: утв. распоряжением Правительства РФ от 02.04.2014 № 503-р // Собрание законодательства РФ. 2008. № 29 (ч. II). Ст. 3537.

23. Marich S. Rail grinding strategies adopted in Australia // Rail Engineer International. 2005. Vol. 9 (4). Pp. 4–6.
24. Zarembski A.M. The evolution and application of rail profile grinding // Bulletin of the American Railway Engineering Association. 1988. Bulletin 718. Vol. 89. Pp. 149–168.
25. Takahashi J., Kobayashi Y., Ueda M., Miyazaki T., Kawakami K.. Nanoscale characterisation of rolling contact wear surface of pearlitic steel // Materials Science and Technology. 2013. Vol. 10(29).
26. Yokoyama H., Mitao S., Yamamoto S., Kataoka Y., Sugiyama T. High strength bainitic steel rails for heavy haul railways with superior damage resistance // NKK Technical Review. 2001. Vol. 84. Pp. 44–51.

27. Sharma S., Sangal S., Mondal K. Wear behaviour of bainitic rail and wheel steels // Materials Science and Technology. 2016. Vol. 4(32). Pp. 1–9.

railways with superior damage resistance. *NKK Technical Review.* 2001. Vol. 84. Pp. 44–51.

27. Sharma S., Sangal S., Mondal K. Wear behavior of bainitic rail and wheel steels. *Materials Science and Technology.* 2016. Vol. 4(32). Pp. 1–9.

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Kosenko S.A., Akimov S.S. Performance characteristics of differentially quenched rails. *Magazine of Civil Engineering*. 2017. No. 7. Pp. 94–105. doi: 10.18720/MCE.75.9.

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Water resistance of polymer compounds

Водостойкость полимерных компаундов

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Key words: polymer composites; epoxy resin; water resistance; polysulfone; polyetherimide

Ключевые слова: полимерные композиционные материалы; эпоксидная смола; водостойкость; полисульфон; полиэфиримид

Abstract. Composite materials have high physical and mechanical properties, but their widespread use is hampered by a lack of tightness (impermeability to moisture and gases). Mainly this is due to the stress-strain state of the composite structure at the micro level as a result of a comprehensive extension of the polymer owing to the adhesion of the matrix to the fiber. The development of cracks and epoxy binder swelling causes its shrinkage and thus, may change the size of distribution and the internal stress patterns in glass fiber wool and lead to a partial disruption of adhesive bonds at the interface, formation of pores, cracks and other defects in the boundary polymer layer. Finally, it can be the reason of its partial detachment from the fibers' surface. It should be noted that such changes depend mostly on the chemical structure and composition of the polymer binder. Watertight composite material development increases the possibilities of its using in many industries, including aerospace, shipbuilding, etc. In this paper the water resistance of polymer compounds was investigated by comparing the properties of the samples with modifying compounds based on epoxy resin. As a part of the study water saturation curves were determined for various compounds and curing agents, the optimal concentration of additives were measured. The results demonstrate the possibility of using of polysulfone as an additive in epoxy resin, improving the water resistance of fiber reinforced plastic.

Аннотация. Композитные материалы обладают высокими физико-механическими свойствами, однако их широкому применению препятствует отсутствие герметичности (непроницаемости для влаги и газов). Это в основном обусловлено напряженно-деформированным состоянием структуры композита на микроуровне в результате всестороннего растяжения полимера вследствие адгезии матрицы к волокну. Развитие трещин и набухание эпоксидного связующего вызывают его усадку, следовательно, могут изменить величину распределения и характер внутренних напряжений в стеклопластике, привести к частичному нарушению адгезионных связей на границе раздела, образованию пор, трещин и других дефектов в граничном полимерном слое, и, в конечном счете, вызвать его частичное отслоение от поверхности волокон. Следует отметить, что такие явления в значительной степени зависят от химического строения и состава полимерного связующего. Разработка герметичного по отношению к влаге композитного материала увеличивает возможности его применения во многих отраслях, включая космонавтику, судостроение и т.д. В настоящей работе исследована влагостойкость полимерных компаундов путем сопоставления свойств образцов с модифицирующими составами на основе эпоксидной смолы. В ходе исследования получены кривые водонасыщения для различных составов и отвердителей, определены оптимальные концентрации добавок. Полученные результаты свидетельствуют о возможности использования полисульфона в качестве добавки в эпоксидную смолу, тем самым повышая влагостойкость армированного пластика.

Introduction

Currently, the most promising materials with their strength, elastic and other properties are polymer materials [1–3]. They are used in almost all sectors of the national economy, and especially in the construction [4, 5], the automotive industry [6, 7], the chemical industry, the energy sector [8]. Polymeric

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materials are widely used in modern construction as a coverings of floors (linoleum, polyvinylchloride tiles, etc.), walls and ceilings, waterproofing materials, as well as in the manufacture of heat and sound insulation materials (porous plastics, foams), window blocks and doors, structural and finishing elements of buildings, varnishes, paints, enamels, adhesives, mastics (based on polymer binder) and for many other purposes.

The term "polymer materials" combines three large groups of synthetic materials: polymers [9], plastics [10] and their morphological variety – polymer composite materials (PCM) or, as they are also called, reinforced plastics [2, 11–13].

The binder (matrix) of the polymer composite material (PCM) performs the following functions: provides a material wholeness and promotes the uniform distribution of loads between the reinforcing elements that leads to inhibition of cracks growing, as well as transmission and distribution of stresses. It is matrix that determines the heat, moisture, fire and chemical resistance of PCM [14–16].

Ideal binder should have a high modulus of elasticity, relatively low elongation and high adhesion strength. One of the main requirements to the binder is to match the magnitude of its elongation in the solid state analogous to the deformation characteristics of the filler [17]. Elongation of polymer binder should be a little bit higher than one of the fiber [18]. When using carbon and glass fibers of large diameter (15–20 microns), the elongation is usually not more than 1.5–2%. The thin fibers (diameter is less than 10 microns) have considerably higher magnitude of elongation, which are 3–5%. Thus, the magnitude of the elongation of the binder should be in the range of 1.5–5% [16, 19].

The epoxy binders are widely used in the manufacture of structural parts from composite materials [20]. PCM on their basis are 15 times more durable than silicone ones and several times stronger than phenol based. The epoxy binders are slightly inferior to the epoxy phenolic ones in heat resistance [15]. The main advantage of epoxy resins is high adhesive strength, good manufacturability, low swelling and others [21]. However, epoxy resins are quite brittle, the magnitude of their elongation are typically less than 1% and therefore, it is very important to find effective ways to modify them to increase their deformation properties. Typical plasticizers, such as rubbers, can only a few improve toughness and crack resistance, however modulus and glass transition temperature are reduced by adding to material. One of the methods to significantly increase the deformation characteristics of epoxy resins, which do not cause their performance degradation, is the using of thermoplastics [22], they are added into the epoxy oligomer at its preparation stage and before injection of the herdening agent [23]. The most widely used thermoplastics are polyetherketones, polyetherimides [24–30] and polysulfones [31].

Polysulfone has a low degree of branching and stereoregular structure of macromolecules, but due to the high chain rigidity it is an amorphous transparent polymer. Polysulfone density is about 1240–1250 kg/m^3, the glass transition temperature is 190–195 ℃. The temperature of destruction is 420℃. Polysulfone is strong heat-resistant engineering thermoplastic with high toughness. Properties and sizes of products do not change in a wide temperature range; frost-resistance can be about 100 ℃. This is chemical, water, oil and petrol-resistant polymer. It has good anti-friction and dielectric properties; is non-toxic, and is sterilized by boiling. It is used in electrical engineering, mechanical engineering and medicine [10], and also in instrument making, machine-tool, diesel and automotive industry for the manufacture of structural, sealing and anti-friction parts operating at temperatures up to 150 ℃ [32, 33]. However, the possibility of polysulfone application to reduce the water permeability of material is poorly investigated [34].

Polyetherimide is an amorphous transparent (amber-transparent) polymer with high rigidity and strength even if continuously used at the temperature up to 170 ℃. It has improved mechanical, electrical insulating and thermal properties in comparison with other transparent amorphous plastics, rigidity at high temperatures is higher than this one of many semi-crystalline high-temperature polymers (glass transition point is +216 °C in short-term operating temperature 200 ℃). Polyetherimide has high inherent flame resistance without the addition of flame retardants and low smoke generation. Due to the excellent electrical characteristics and flame resistance it is often used for the manufacture of electrical and electronic insulators, contact strips, distributor hoods and other parts that require high strength and stability at elevated temperatures, and is also used in the manufacturing of the aircraft industry parts. Polyetherimide is physiologically inert. Good hydrolysis resistance and dimensional stability permit the use of polyetherimide not only in electrical engeneering, but also in medical devices, for example, in analytical devices. Its density is 1270 kg/m^3. Polyetheremide is chemically resistant to gasoline, oils, alcohols, weak acids. It has limited resistance to strong acids and is not resistant to alkalis.

Polyetherketone is a high-temperature thermoplastic polymer of taupe or brown-black colour. It differs by amount of ether and ketone groups. The main differences are the glass transition temperature and melting point due to increasing number of ketone groups of the polymer. The glass transition point is 160 ℃, melting point is 335 ℃, long-term operation temperature is 243 ℃, short-term is up to 315 ℃. In addition to resistance to high temperature and mechanical stress, polyether ketone has a very good chemical resistance, responds well to machining, has low water absorption and shows high dimensional stability. It is resistant to hydrolysis and hot steam. Polyetherketone density is about 1310–1320 kg/m^3.

Epoxy resins (ER) take place in various industries as a basis for adhesives, mastics, coatings, compounds and reinforced plastics [21, 24]. Recently investigations are conducted, they relate to the high penetrating abilities of epoxy oligomers in porous materials with subsequent structuring with the influence of different hardeners and giving of unique compositional properties to the final composite [4, 6, 35].

In this paper the possibilities of polysulfone additives to modify ER in order to improve the operating properties of the final product in high humidity environments or in direct contact with water were investigated. Adding polysulfone in material is justified by its very low shrinkage, resistance to high temperatures, chemical resistance in comparison with other thermoplastics, and also high resistance to hydrolysis. Currently, using of the binder epoxy resin-thermoplastic for the manufacture of composite materials has not been systematically investigated, data on the physical and mechanical properties, as well as the effect of polysulfone on the water resistance of polymer compounds, are practically absent both in foreign and domestic literature.

Methods (Experimental part)

Epoxy resin ED-20 is a viscous light yellow liquid [36, 37]. After **hardening**, the products based on ER can **be subjected** to the appearance of cracks due to its low elasticity. So additives are required to modify the properties of ER [38-41]. Polyetherimide (PEI) and polysulfone (PSK-1) were selected as modified additives, which are added to the composition of the polymer compounds with different mass fractions, but not more than 20%. The introduction of modifiers into the polymer compound composition with a concentration more than 20% leads to a marked increase in viscosity and complicates the production process of creating composite materials. Also, the concentration limit of 20% was determined as the most effective for increasing the impact e strength and fracture impact strength properties of composites [42], which may be an indirect indicator of the water resistance increase. These polymer compounds are produced with the addition of various hardeners: trietanolaminotitonate (TEAT) and diaminodifinilsulfone (DADFS).

The samples were prepared using the following technologies and the following composition (for example, sample №1):

1) 100g of resin Aradlite LY 556 containing 5$_{wt}$% of the PSK-1 (from the final weight of the polymer compound).
2) 90g of hardener Aradur 917.
3) 0.3g (6 drops) of accelerator Accelerator DY 070.

Curing took place in two stages in special silicone forms in the temperature cabinet SNOL 58/350:

Stage 1. Incubation for 3 hours at 90 ℃.

Stage 2. Incubation for 12 hours at a temperature of 120 ℃.

By similar techniques 13 samples were made of the resins of various compositions. All samples have dimensions of 18mm x 7.5mm x 5mm.

Thus, 14 samples were obtained:

1. 5%$_{wt}$ PSK-1 TEAT;
2. 10%$_{wt}$ PSK-1 TEAT;
3. 15%$_{wt}$ PSK-1 TEAT;
4. 20%$_{wt}$ PSK-1 TEAT;
5. Test ED-20 TEAT;
6. 5%$_{wt}$ PSK-1 DADFS;
7. 10%$_{wt}$ PSK-1 DADFS;
8. 15%$_{wt}$ PSK-1 DADFS;
9. 20%$_{wt}$ PSK-1 DADFS;
10. Test ED-20 DADFS;
11. 5%$_{wt}$ PEI DADFS;
12. 10%$_{wt}$ PEI DADFS;
13. 15%$_{wt}$ PEI DADFS;
14. 20%$_{wt}$ PEI DADFS;

After hardening each sample was placed in a sealed flask with distilled water and with index number corresponding to one of the above. During the 3 months before the full water saturation the weight of samples was measured. On the basis of the increase in weight, water saturation weight was determined for each sample.

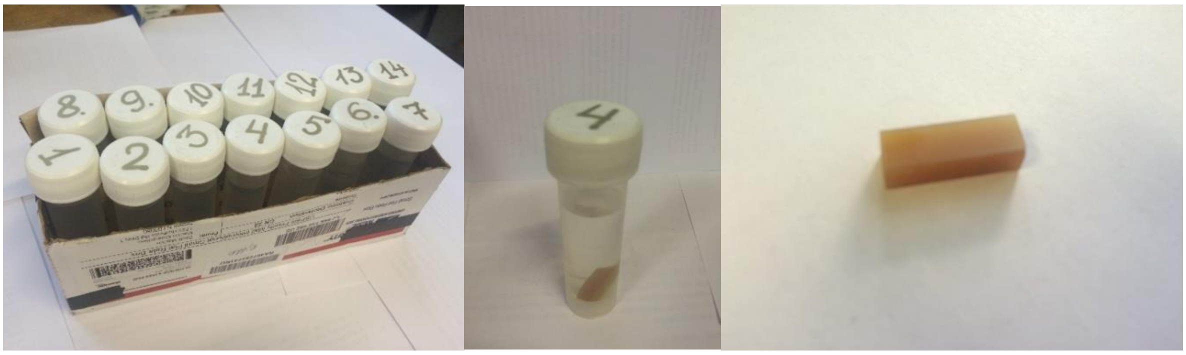

Figure 1. Exposure of polymer compounds samples in water

As a result of the experimental work water saturation values (ΔM/M) in time (T) were measured for each sample, the time of full water saturation of samples (T1) was determined. At the time of full water saturation of the samples the optimal additives concentration (C) for different hardeners was determined.

The experimental data was summarized in the tables and graphs were constructed on their basis (Figы. 2–5).

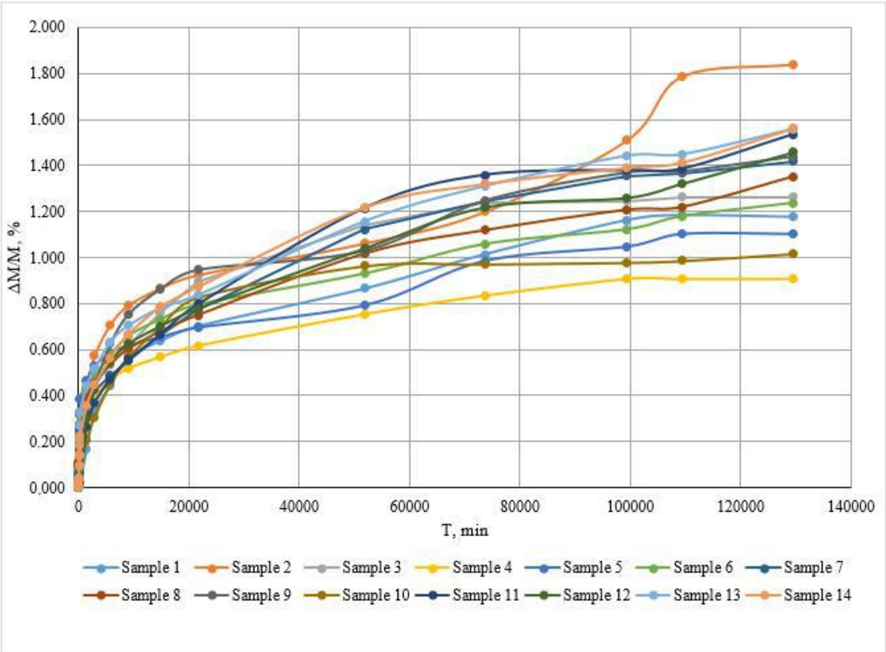

Figure 2. Water saturation of 14 samples - time graph

For each sample, water saturation – time graphs; water saturation - time logarithm graphs; water saturation – concentration (mass content) of additives in polymer compounds were constructed. As the examples of these graphs the data of 4, 5, 6, 10 and 12 samples are considered.

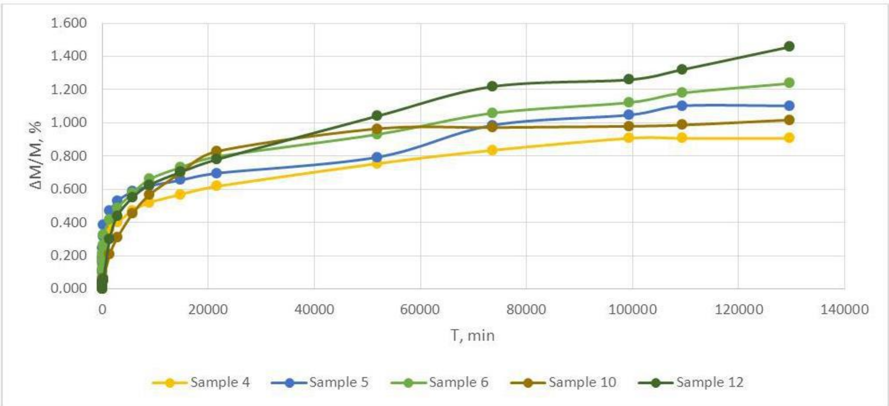

Figure 3. Water saturation of 4, 5, 6, 10, 12 samples - time graph

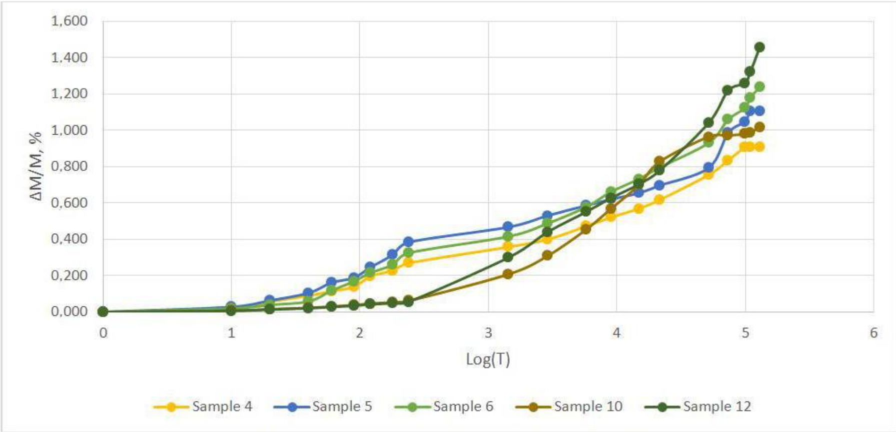

Figure 4. Water saturation of 4, 5, 6, 10, 12 samples – time logarithm graph

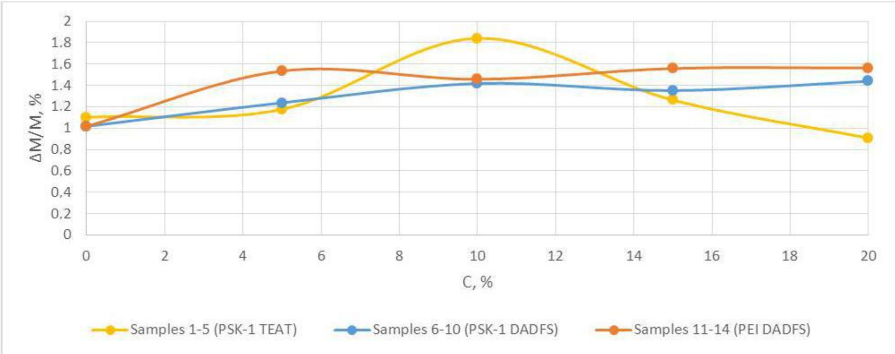

Figure 5. Water saturation of 1-5 (PSK-1 TEAT), 6-10 (PSK-1 DADFS), 10-14 (PEI DADFS) samples – the concentration (mass content) of polysulfone or polyetherimide in polymer compounds graph

After analyzing water saturation-time data of the samples the most water resistant compounds were identified. As a result of water saturation – time logarithm graphs analysis, the moment of total water saturation was determined for all samples, which was 69-76 days (of 90 days), depending on composition. For total water saturation time the full water saturation – concentration (mass content) of polysulfone and polyetherimide praphs (Fig. 5) was plotted.

Results and Discussion (Analysis of the experimental data)

The program of the investigation includes 14 samples. The basis of all the samples was the epoxy resin ED-20 with the addition of two hardeners TEAT and DADFS. As modified additives PSK-1 and were added to the polymer compounds at various concentrations from 5_{wt}% to 20_{wt}%.

As a result of this experiment the total water saturation data of the samples (Table 1) was gathered. The standard deviation is 0.00001.

Table 1. The experimental data. The full water saturation of samples

The full water saturation in total water saturation time, ΔM/M													
1	2	3	4	5	6	7	8	9	10	11	12	13	14
1.176	1.840	1.262	0.907	1.102	1.237	1.417	1.351	1.439	1.017	1.534	1.457	1.558	1.563

Table 1 shows the samples No. 5, No. 10 have sufficiently good water resistance despite the thermoplastics were not being injected in their composition. Among the samples with the addition of polysulfone (PSK-1) and trietanolaminotitonate (TEAT) №4 composition, which contained 20 % of polysulphone, had the highest water impermeability (Table. 1, Fig. 5). Among the samples with the addition of the polysulfone and diaminodifinilsulfone (DADFS) №6 composition, which contained 5 % of polysulfone, had the highest water impermeability (Table. 1, Fig. 5). Among the samples with the addition of polyetherimide (PEI) and diaminodifinilsulfone (DADFS) №12 composition, which contained 10 % of polyetherimide, had the highest water impermeability (Table. 1, Fig. 5).

Analyzing the graph of water saturation - concentrations of additives in the polymer (Fig. 5) we can notice that the water impermeability of the samples with added DADFS hardener is reduced by increasing weight content of polysulfone and polyetherimide additives. At the same time, the water resistance of the samples with added TEAT hardener is increased by increasing of polysulfone weight content. It is important that when we add TEAT hardener the water saturation – polysulfone concentration curve is at extreme if the polusulfone contents is 10 %, and is the least water resistant composition of the test samples.

In the paper [39] the problem of fracture strength of epoxy binders, modified with polysulfone and furfuralacetone resin, was considered. In the article [40], the influence of modifiers (polysulfone and / or carbon nanotubes (CNTs)) on the properties of organoplastics was investigated. The fracture strength and impact strength of reinforced plastics were considered. The article [41] presents data of the PEI and PSC-1 effects on the properties (fracture strength and impact strength) of epoxy binder based on ED-20. Thus, in the above studies [39–41] it was shown that the introduction of thermoplastic modifiers, such as PSC-1, PEI in epoxy resins, increases the fracture strength and impact strength of polymer compositions, that can be considered as an indirect indicator of increasing water resistance. In these studies, the water resistance of the polymers was not directly researched.

Conclusions

The result of this investigation is the determination of the water resistant composition and optimum concentration (weight content) of thermoplastic in polymer compound in order to increase the water resistance of reinforced plastics. The most water resistant sample No. 4 was identified with such composition: "20 $\%_{wt}$ of the PSK-1 TEAT." Thus, the resulting effect of polysulfone 20 $\%_{wt}$ application as the additive in epoxy resin permits the use of this thermoplastic material for the manufacture of composites and its application in high-humid conditions.

References

1. Goldman A.Ya. *Prognozirovaniye deformatsionno-prochnostnykh svoystv polimernykh i kompozitsionnykh materialov* [Forecasting the deformation-strength properties of polymer and composite materials]. Leningrad: Mashinostroyeniye, 1988. 272 p. (rus)
2. Rabinovich A.L. *Vvedeniye v mekhaniku armirovannykh polimerov* [Introduction to the mechanics of reinforced polymers]. Moscow: Nauka, 1970. 481 p. (rus)
3. Nilsen L. *Mekhanicheskiye svoystva polimerov i polimernykh kompozitsiy* [Mechanical properties of polymers and polymer compositions]. Moscow: Khimiya, 1978. 301 p. (rus)
4. Pan X., Webster D.C. Impact of structure and functionality of core polyol in highly functional biobased epoxy resins. *Macromolecular Rapid Communications.* 2011. No. 32(17). Pp. 1324–1330.
5. Kurlapov D.V., Kuvayev A.S., Rodionov A.V., Valeyev R.M. Usileniye zhelezobetonnykh konstruktsiy s primeneniyem polimernykh kompozitov [Injection as a method of strengthening stone structures]. *Magazine of Civil Engineering.* 2009. No. 3(5). Pp. 22–24. (rus)
6. Jaillet F., Desroches M., Auvergne R., Boutevin B., Caillol S. New biobased carboxylic acid hardeners for epoxy resins. *European Journal of Lipid Science and Technology.* 2013. No. 115(6). Pp. 698–708.
7. Loginova I.I., Artamonova D.A., Stolyarov O.N., Melnikov B.E. Vliyaniye struktury na vyazkouprugiye svoystva geosinteticheskikh materialov [Effects of structure type on viscoelastic properties of geosynthetics]. *Magazine of Civil Engineering.* 2015. No. 4(56). Pp. 11–18. (rus)
8. Tobol'skiy A. *Svoystva i struktura polimerov* [Properties and structure of polymers]. Moscow: Khimiya, 1964. 322 p. (rus)
9. Van-Krevelen D.V. *Svoystva i khimicheskoye stroyeniye polimerov* [Properties and chemical structure of polymers]. Moscow: Khimiya, 1976. 416 p. (rus)
10. Pakharenko V.A., Pakharenko V.V., Yakovleva R.A. Plastmassy v stroitel'stve [*Plastics in Construction*]. Saint-Petersburg: Nauchnyye osnovy i tekhnologii, 2010. 350 p. (rus)

Литература

1. Гольдман А.Я. Прогнозирование деформационно-прочностных свойств полимерных и композиционных материалов. Л.: Машиностроение, 1988. 272 с.
2. Рабинович А.Л. Введение в механику армированных полимеров. М.: Наука, 1970. 481 с.
3. Нильсен Л. Механические свойства полимеров и полимерных композиций. М.: Химия, 1978. 301 с.
4. Pan X., Webster D.C. Impact of structure and functionality of core polyol in highly functional biobased epoxy resins // Macromolecular Rapid Communications. 2011. № 32(17). Pp. 1324–1330.
5. Курлапов Д.В., Куваев А.С., Родионов А.В., Валеев Р.М. Усиление железобетонных конструкций с применением полимерных композитов // Инженерно-строительный журнал. 2009. № 3(5). С. 22–24.
6. Jaillet F., Desroches M., Auvergne R., Boutevin B., Caillol S. New biobased carboxylic acid hardeners for epoxy resins // European Journal of Lipid Science and Technology. 2013. № 115(6). Pp. 698–708.
7. Логинова И.И., Артамонова Д.А., Столяров О.Н., Мельников Б.Е. Влияние структуры на вязкоупругие свойства геосинтетических материалов // Инженерно-строительный журнал. 2015. № 4(56). С. 11–18.
8. Тобольский А. Свойства и структура полимеров. Пер. с англ. под ред. Слонимского Г.Л. и Бартенева Г.М. М.: Химия, 1964. 322 с.
9. Ван-Кревелен Д.В. Свойства и химическое строение полимеров. М.: Химия, 1976. 416 с.
10. Пахаренко В.А., Пахаренко В.В., Яковлева Р.А. Пластмассы в строительстве. СПб.: Научные основы и технологии, 2010. 350 с.
11. Мэнсон Дж., Сперлинг Л. Полимерные смеси и композиты. М.: Химия, 1979. 440 с.
12. Harper Ch.A. Modern plastics handbook. New York, McGraw-Hill, 2000. 1233 p.
13. Christensen R.M. Mechanics of Composite Materials. New York, John Wiley&Sons, 1979. 324 p.
14. Ванин Г.А. Микромеханика композиционных

11. Menson Dzh., Sperling L. *Polimernyye smesi i kompozity* [Polymer mixtures and composites]. M.: Khimiya, 1979. 440 p. (rus)
12. Harper Ch.A. *Modern plastics handbook.* New York, McGraw-Hill, 2000. 1233 p.
13. Christensen R.M. *Mechanics of Composite Materials.* New York, John Wiley&Sons, 1979. 324 p.
14. Vanin G.A. *Mikromekhanika kompozitsionnykh materialov* [Micromechanics of composite materials]. Kiyev: Naukova dumka, 1985. 304 p. (rus)
15. Lipatov Yu.S. *Budushcheye polimernykh kompozitsiy* [The future of polymer compositions]. Kiyev: Naukova dumka, 1984. 133 p. (rus)
16. Lipatov Yu.S. *Fizicheskaya khimiya napolnennykh polimerov* [Physical Chemistry of Filled Polymers]. Moscow: Khimiya, 1977. 387 p. (rus)
17. Kolosova A.S., Sokol'skaya M.K., Vitkalova I.A., Torlova A.S., Pikalov Ye.S. Fillers to modify the modern polymer composite materials. *Fundamental research.* 2017. No. 10. Pp. 459–465. (rus)
18. Perepelkin K.Ye. *Armiruyushchiye volokna i voloknistyye polimernyye kompozity* [Reinforcing fibers and fibrous polymeric composites]. Saint-Petersburg: Nauchnyye osnovy i tekhnologii, 2009. 386 p. (rus)
19. Korshak V.V. *Khimicheskoye stroyeniye i temperaturnyye kharakteristiki polimerov* [Chemical structure and temperature characteristics of polymers]. Moscow: Nauka, 1970. 420 p. (rus)
20. Lapitskiy V.A., Kritsuk A.A. *Fiziko-mekhanicheskiye svoystva epoksidnykh polimerov i stekloplastikov* [Physico-mechanical properties of epoxy polymers and fiberglass]. Kiyev: Naukova dumka, 1986. 96 p. (rus)
21. Chernin I.Z., Smekhov F.M., Zherdev Yu.V. *Epoksidnyye polimery i kompozitsii* [Epoxy polymers and compositions]. Moscow: Khimiya, 1982. 232 p. (rus)
22. Korshak V.V. *Termostoykiye polimery* [Heat Resistant Polymers]. Moscow: Nauka, 1969. 411 p. (rus)
23. Kestelman V.M. *Fizicheskiye metody modifikatsii polimernykh materialov* [Physical methods for the modification of polymeric materials]. Moscow: Khimiya, 1980. 224 p. (rus)
24. Glazkov S.S. Stabilizatsiya tortsevoy parketnoy shashki s ispol'zovaniem epoksidnoy smoly [Stabilizing parquet blocks with epoxy resin]. *Magazine of Civil Engineering.* 2015. No. 7(59). Pp. 57–65. (rus)
25. Bartolomucci J.R. *Polyetherimide.* Modern Plastics encyclopedia, 1986-1987. 50 p.
26. Beland S. *High performance thermoplastic resins and their composites.* New Jersey, Noyes Data Corporation, 1990. 177 p.
27. Fink J.K. *High performance polymers.* Norwich, William Andrew Inc., 2008. Pp. 475–519.
28. Gallucci R.R. Thermoplastic polyetherimide (PEI). *Engineering plastics handbook.* The McGraw-Hill Companies Inc., 2006. Pp. 155–180.
29. Kim J.H. Polyetherimide substrates for future high density optical data storage. *Polym. Eng. Sci.* 2008. No. 1(48). Pp. 97–101.
30. Wagner A.H., Yu J.S., Kalyon D.M. Microstructure and ultimate properties of injection molded amorphous engineering plastics: poly(ether imide) and poly (2,6-dimethyl-1,4 phenylene ether). *Polym. Eng. Sci.* 1989. No. 18(29). Pp. 1298–1307.
31. Oudian Dzh. *Osnovy khimii polimerov* [Basics of polymer chemistry]. Moscow: Mir, 1974. 416 p.
32. Lee H., Stoffey D., Neville K. *New Linear Polymers.* New York, McGraw-Hill, 1967. 176 p.
33. Savina M.Ye., Chernova N.K., Pinayeva N.K. et al. Kompozitsiya na osnove polisulfona [Composition based

материалов. Киев: Наукова думка, 1985. 304 с.

15. Липатов Ю.С. Будущее полимерных композиций. Киев: Наукова думка, 1984. 133 с.
16. Липатов Ю.С. Физическая химия наполненных полимеров. М.: Химия, 1977. 387 с.
17. Колосова А.С., Сокольская М.К., Виткалова И.А., Торлова А.С., Пикалов Е.С. Наполнители для модификации современных полимерных композиционных материалов // Фундаментальные исследования. 2017. № 10. С. 459–465.
18. Перепелкин К.Е. Армирующие волокна и волокнистые полимерные композиты. СПб.: Научные основы и технологии, 2009. 386 с.
19. Коршак В.В. Химическое строение и температурные характеристики полимеров. М.: Наука, 1970. 420 с.
20. Лапицкий В.А., Крицук А.А. Физико-механические свойства эпоксидных полимеров и стеклопластиков. Киев: Наукова думка, 1986. 96 с.
21. Чернин И.З., Смехов Ф.М., Жердев Ю.В. Эпоксидные полимеры и композиции. М.: Химия, 1982. 232 с.
22. Коршак В.В. Термостойкие полимеры. М.: Наука, 1969. 411 с.
23. Кестельман В.М. Физические методы модификации полимерных материалов. М.: Химия, 1980. 224 с.
24. Глазков С.С. Стабилизация торцевой паркетной шашки с использованием эпоксидной смолы // Инженерно-строительный журнал. 2015. № 7(59). С. 57–65.
25. Bartolomucci J.R. Polyetherimide. Modern Plastics encyclopedia, 1986-1987. 50 p.
26. Beland S. High performance thermoplastic resins and their composites. New Jersey, Noyes Data Corporation, 1990. 177 p.
27. Fink J.K. High performance polymers. Norwich, William Andrew Inc., 2008. Pp. 475–519.
28. Gallucci R.R. Thermoplastic polyetherimide (PEI) // Engineering plastics handbook. The McGraw-Hill Companies Inc., 2006. Pp. 155–180.
29. Kim J.H. Polyetherimide substrates for future high density optical data storage // Polym. Eng. Sci. 2008. № 1(48). Pp. 97–101.
30. Wagner A.H., Yu J.S., Kalyon D.M. Microstructure and ultimate properties of injection molded amorphous engineering plastics: poly(ether imide) and poly(2,6-dimethyl-1,4 phenylene ether) // Polym. Eng. Sci. 1989. № 18(29). Pp. 1298–1307.
31. Оудиан Дж. Основы химии полимеров. М.: Мир, 1974. 416 с.
32. Lee H., Stoffey D., Neville K. New Linear Polymers. New York, McGraw-Hill, 1967. 176 p.
33. Патент № 747867. СССР. Композиция на основе полисульфона. Савина М.Е., Чернова Н.К., Пинаева Н.К. и др. Заявлено 07.02.78. Опубл.15.07.1980.
34. Баском В. Химия поверхности композитов, подвергнутых воздействию влаги // Композиционные материалы. Т. 6. Поверхности раздела в полимерных композитах. М.: Мир, 1978. С. 88–118.
35. Sindu B.S., Sasmal S. Evaluation of mechanical characteristics of nano modified epoxy based polymers using molecular dynamics // Computational Materials Science. 2015. № 1(96). Pp. 146–158.
36. Мошинский Л. Эпоксидные смолы и отвердители. Тель-Авив: Аркадия пресс Лтд, 1995. 370 с.
37. Пакен А.М. Эпоксидные соединения и эпоксидные смолы. Л.: Госхимиздат, 1962. 964 с.
38. Благонравова А.А., Непомнящий А.И. Лаковые эпоксидные смолы. М.: Химия, 1970. 248 с.
39. Копицына М.Н., Бессонов И.В., Котомин С.В. Трещиностойкость эпоксидных связующих,

on polysulfone]. *Patent USSR no. 747867*, 1980. (rus)

34. Baskom V. *Khimiya poverkhnosti kompozitov, podvergnutykh vozdeystviyu vlagi* [Surface chemistry of composites exposed to moisture]. Kompozitsionnyye materialy [Composite materials]. Vol. 6. Poverkhnosti razdela v polimernykh kompozitakh [The interfaces in polymer composites]. Moscow: Mir, 1978. Pp. 88–118. (rus)
35. Sindu B.S., Sasmal S. Evaluation of mechanical characteristics of nano modified epoxy based polymers using molecular dynamics. *Computational Materials Science.* 2015. No. 1(96). Pp. 146–158.
36. Moshinskiy L. *Epoksidnyye smoly i otverditeli* [Epoxy resins and hardeners]. Tel'-Aviv: Arkadiya press Ltd, 1995. 370 p. (rus)
37. Paken A.M. *Epoksidnyye soyedineniya i epoksidnyye smoly* [Epoxy compounds and epoxy resins]. Leningrad: Goskhimizdat, 1962. 964 p. (rus)
38. Blagonravova A.A., Nepomnyashchiy A.I. *Lakovyye epoksidnyye smoly* [Lacquered epoxy resins]. Moscow: Khimiya, 1970. 248 p. (rus)
39. Kopitsyna M.N., Bessonov I.V., Kotomin S.V. Fracture strength of epoxy binders modified by thermoplastic polysulfone and furfural-acetone resin. *Engineering Journal: Science and Innovation.* 2016. No. 12(60). Pp. 1–9. (rus)
40. Solodilov V.I., Korokhin R.A., Gorbatkina Yu.A., Kuperman A.M. Organoplastiki na osnove slozhnykh gibridnykh matrits, vklyuchayushchikh v kachestve modifikatorov epoksidnykh smol polisulfon i uglerodnyye nanotrubki [Organoplastics based on complex hybrid matrices, including polysulfone and carbon nanotubes as epoxy resin modifiers]. *Khimicheskaya fizika.* 2012. No. 6(31). Pp. 63–71. (rus)
41. Kolozhvari Ye.A., Gorbunova I.Yu., Kerber M.L., Gorbatkina Yu.A. Uluchsheniye svoystv epoksiaminnogo svyazuyushchego putem modifitsirovaniya termoplastami [Improving the properties of epoxyamine binder by modifying with thermoplastics]. *Uspekhi v khimii i khimicheskoy tekhnologii.* 2007. No. 5(73). Pp. 64–68. (rus)
42. Solodilov V.I. *Armirovannyye plastiki na osnove epoksipolisulfonovykh svyazuyushchikh, poluchennyye metodom namotki* [Reinforced plastics based on epoxy-polysulfone binders, obtained by winding]. PhD Thesis 02.00.06. Moscow, 2005. 148 p. (rus)

модифицированных термопластичным полисульфоном и фурфуролацетоновой смолой // Инженерный журнал: наука и инновации. 2016. № 12(60). С. 1–9.

40. Солодилов В.И., Корохин Р.А., Горбаткина Ю.А., Куперман А.М. Органопластики на основе сложных гибридных матриц, включающих в качестве модификаторов эпоксидных смол полисульфон и углеродные нанотрубки // Химическая физика. 2012. № 6(31). С. 63–71.
41. Коложвари Е.А., Горбунова И.Ю., Кербер М.Л., Горбаткина Ю.А. Улучшение свойств эпоксиаминного связующего путем модифицирования термопластами // Успехи в химии и химической технологии. 2007. № 5(73). С. 64–68.
42. Солодилов В.И. Армированные пластики на основе эпоксиполисульфоновых связующих, полученные методом намотки: дис. … канд. тех. наук: 02.00.06. Москва, 2005. 148 с.

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The phase composition and properties of aluminate cements after early loading

Фазовый состав и свойства алюминатных цементов при раннем нагружении

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Ключевые слова: глиноземистый цемент; высокоглиноземистый цемент; раннее нагружение; прочность на сжатие; прочность на растяжение при изгибе; рентгенофазовый анализ; дифференциально-термический анализ

Abstract. It is widely recognized that the effect of loading at the early stages of hardening enables to increase strength characteristics of cement systems and composites based on them. Of particular interest is a study on the effect of compression of aluminate cements on physicomechanical characteristics, hydration process and phase transformations. The research focuses on maximum compressive and flexural strength, the peak intensity of the main phases and hydrate products, characteristics of DTA curves after early loading, studied by means of physical and chemical methods. The authors note an increased flexural strength of specimens exposed to loading at an early age. The comparison of diffractograms showed that the peaks of the main phases were reduced during the compression stage, as well as the changes in the amorphous structure of the stone. The differential thermal analysis showed no change in bound water content.

Аннотация. Воздействие нагрузки на ранних этапах твердения позволяет получать прирост прочностных показателей цементных систем и композитов на их основе, что является общеизвестным фактом. Интерес представляет исследование влияние сжатия структуры алюминатных цементов на физико-механические характеристики, процесс гидратации и изменение фазового состава. В работе при помощи физико-химических методов изучались предел прочности на сжатие и растяжение при изгибе, интенсивность пиков основных фаз и гидратных продуктов, характер эффектов на кривых ДТА при раннем нагружении. Авторами отмечается увеличение прочности на изгиб образцов, подвергнутых нагрузке в раннем возрасте. Путем сравнения дифрактограмм установлено снижение пиков основных фаз при сжатии структуры, а также различия в аморфности структуры камня. Несмотря на отмеченные изменения, дифференциально-термический анализ показал отсутствие изменений связанной воды.

Introduction[1]

Calcium aluminate cements (CAC), known as Fondu cements, have become widespread in construction industry, including the production of high-performance concretes (HPC) [1], because of the following properties [2–5]:

- Rapid strength development;
- High resistance to corrosion;
- Stability at high temperatures and flame resistance.

[1] Notation: C=CaO; A=Al_2O_3; H=H_2O; S=SiO_2.

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An intensive hardening is accompanied by an increased heat release during hydration, and within 24 hours about 70–90 % of all heat should be released, while the temperature of material can reach up to 1000°C [6]. The development of structure is mainly occurs through the hydration of calcium monoaluminate CA [7]. The most important hydroaluminates are CAH_{10}, C_2AH_8, C_4AH_x (x = 13-19), C_3AH_6 [8–9] (cubic phase), and AH_3 [10] as an amorphous gel which crystallizes to gibbsite.

The following chemical equations demonstrate the effect of temperature on the composition of hydration products [11–12]:

- ($T \leq 15^0C$) $CA+10H \rightarrow CAH_{10}$;
- ($15^0C<T\leq 30^0C$) $2CA+11H \rightarrow C_2AH_8 + AH_3$;
- ($30^0C<T$) $3CA+12H \rightarrow C_3AH_6 + 2AH_3$.

Over time, crystallization of metastable CAH_{10} and C_2AH_8 leads to their conversion to a thermodynamically stable cubic C_3AH_6. As a result of conversion reactions, some of the bound water within the crystal structure is liberated resulting in an increase in porosity of CAC matrix and consequently in a decrease in strength, which limits the scope of application of the CAC [2].

An opportunity to apply pre-stress to cement and concrete composites at an early stage of hardening and to achieve design requirements (along with the obvious acceleration of construction works [13]) without any loss of performance characteristics is particularly relevant. The review [14] presents some data on changes of properties after early loading, and many studies have been devoted to the application of this method (for example, [15–24]), in the course of which Portland cement silicates (with crystalline, submicrocrystalline and amorphous structure) silicates had been exposed to compression.

The study of the effect of early loading of a structure consisting mostly of calcium aluminates (CA) will allow us to gain a better understanding of how the aluminate component affects the effectiveness in comparison with silicate component, to investigate the nature of the changes, taking into account the properties and structure of aluminate cements. Rapid strength development makes it possible to apply a significant loading at the earliest stages of hardening (24–72 hours from the moment of molding). Obtaining data on the effects of loading (changes of compressive and bending tensile strength, hydration, bound water content, composition and number of new formations) is of particular interest.

The main purpose of the study is to analyze the changes occurring in the structure of aluminate cement after early loading, which determines the following tasks:

- obtain values of compressive and bending tensile strength after preliminary short-term compression;
- conduct X-ray phase analysis and describe the changes of intensity of the peaks of crystalline phases, take an assessment of amorphousness of the deformed structure;
- analyze differences in weight loss using the DTA methods.

Materials and Methods

High aluminate cement GC-50 (according to Russian State Standard GOST RF 969-91) produced by Pashiya Metallurgical Cement Plant was used (Table 1) as a binding component. The X-ray phase analysis of initial cement stone (before mixing with water) was carried out to identify main phases. The main mineralogical phase is calcium monoaluminate CA. There are also $C_{12}A_7$, C_2AS, C_4AF and CA_2 to be found. The size of the prism is 40 × 40 × 160 mm. The samples have been molded from cement-sand grout with the following shares of components: cement : sand : water = 2.5 : 2.5 : 1, using fractional sand 0–0.63 mm, purified from any foreign and clay particles as a fine aggregate. Bending under tension tests were conducted on prisms, while compressive strength was determined by testing prism halves. Thus, each point of the generated strength curve indicates average value obtained from 3 measurements of bending tensile strength and 6 measurements of compressive strength values.

Table 1 Chemical composition of cements applied (%)

Al_2O_3	CaO	SiO_2	Fe_2O_3	MgO	TiO_2
38–42	27–29	10–12	5–8	<5	<10

Correct load distribution was provided through the use of hinged bearings. Samples that were not exposed to loading (hereinafter referred to as "control samples"), as well as samples before and after loading, were stored under the same normal conditions of humidity.

Curing duration of samples subjected to loads is 24-hours from the moment of their manufacture. For the experiment, the value of the compressive load was taken as a constant and was equal to 10% of the daily strength of the sample. It was expected that effect of earlier loading would be the most significant at the stage of formation of composite structures. Cracks were not allowed, as well as the eccentricities corresponding to the points of load application. Taking into account rapid strength development of aluminous cement (grade strength R = 50.3 MPa is achieved after 72 hours), a period of short-term loading should be 24 hours. Bending under tension test for prisms was performed in accordance with Russian State Standard GOST RF 310.4, for cubes – according to Russian State Standard GOST RF 10180. Calculation and statistical methods were applied for analysis of the obtained data. The accuracy rate (the ratio of the mean error to the arithmetic mean) did not exceed 2.6 % for bending under tension test, and it reached the value 3.9 % for compression test.The results were assessed for 5 % significance level. The total test duration was 15 days. After 10 days, the increase of strength did not exceed 5 %, and the slope of a line tangent to strength curves tended to a constant, therefore, in the present work, changes in compressive and bending strength are presented for 10 days.

X-ray phase analysis was performed using an XRD-7000 Shimadzu diffractometer (Japan). The peaks identification in the diffractograms was carried out using the PDWin 4.0 and Crystallographica Search-Match software, integrated into the hardware software complex of the device. The shooting conditions were the following: copper anode, the wavelength of radiation Kα 1.54051Å, 40kV, 30mA, the angle range 5 to 70 degrees, the shooting speed 1 deg/min.

The differential thermal analysis was performed by Netzsch STA-409 PC Luxx, temperature range of 25–1000 °C. The test has been carried out under the air atmosphere conditions in platinum crucibles at a heating rate of 10 °C/min.

Observational data are presented as strength curves (Fig. 1), X-ray diffraction patterns of alumina cement before mixing with water (Fig. 2a), comparision of overlaid diffractograms of loaded and control samples (Fig. 2b, 2c), and comparision of their derivatograms (Fig. 3).

Results and Discussion

The effect of a 24-hour static compression on the strength development of aluminate cement for 10 days is shown in Figure 1. The prisms subjected to the loading showed an increase of ultimate tensile strength (up to 29 % on the second day). Observations indicate that in the course of aging an increase in strength development is reducing.

At the early loading, an increase in the compressive strength was not detected (Fig. 1). Comparison of diagrams demonstrates that the increase rate reduces, and it becomes close to a constant after 3 days. The fact that the compression strength of aluminate cement decreases when loaded for 1 day is very much in line with the data of [25].

A comparative analysis of X-ray diffraction patterns of the sample subjected to early short-term compression (drawn in blue color in the Fig. 2b) and control sample (drawn in red color in the Fig. 2b) has been carried out. Nine days later, another X-ray phase analysis of the same samples was carried out (Fig. 2c).

Figure 2b shows the peaks of control and pre-loaded samples related to hydroaluminates of CAH_{10} type (2Θ = 6.1°; 12.3°), whose intensity decreases by the 10^{th} day due to recrystallization (Fig. 2c), as well as because of the formation of C_2AH_8 and aluminum hydroxide (amorphous gel). The arc of amorphous cement without any load in the angle range 2Θ = 5-17 ° is higher, which indicates an increasing content of loosely bound water in its structure. The authors of [26–31] drew attention to the almost instantaneous change in moisture content during the compression of cement systems, relating it to the intense shrinkage and redistribution of water in capillaries and interlayer space under compressive loading.

As it is seen in X-ray diffraction patterns of cement stone which was subjected to early short-term compression (Fig. 2b), the peak intensity of the main phase – calcium monoaluminate CA is decreasing (2Θ=16.1°; 18.8°; 22.8°; 24.07°; 28.85°; 31.14°; 40.14°; 41.01°; 59.14°), as well as peak intensity of $C_{12}A_7$ (2Θ = 18.02°; 36.56°) и CA_2 (2Θ = 28.85°; 34.27°). However, the number of peaks related to CAH_{10}, as well as the peak intensity, is higher in the X-ray patterns of the control sample (Fig. 2b). This is true for C_2AH_8 (hexagonal phase) and for AH_3 (microcrystalline phase). Recorded decrease of the number of peaks of crystalline hydration products in the cement stone after early short-term compression whilst decreasing peak intensity of main mineralogical phases in the X-ray pattern is probably due to increase of the amorphous content in the pre-loaded samples, which is difficult to identify with X-ray phase analysis. In this case the structure of the cement stone subjected to load at an early stage can be more amorphous, which is consistent with the conclusions [32, 33], and it may have more specific surface area [34]. Data on crystallinity decreasing of water containing structures subjected to compression are given in the sources [35, 36].

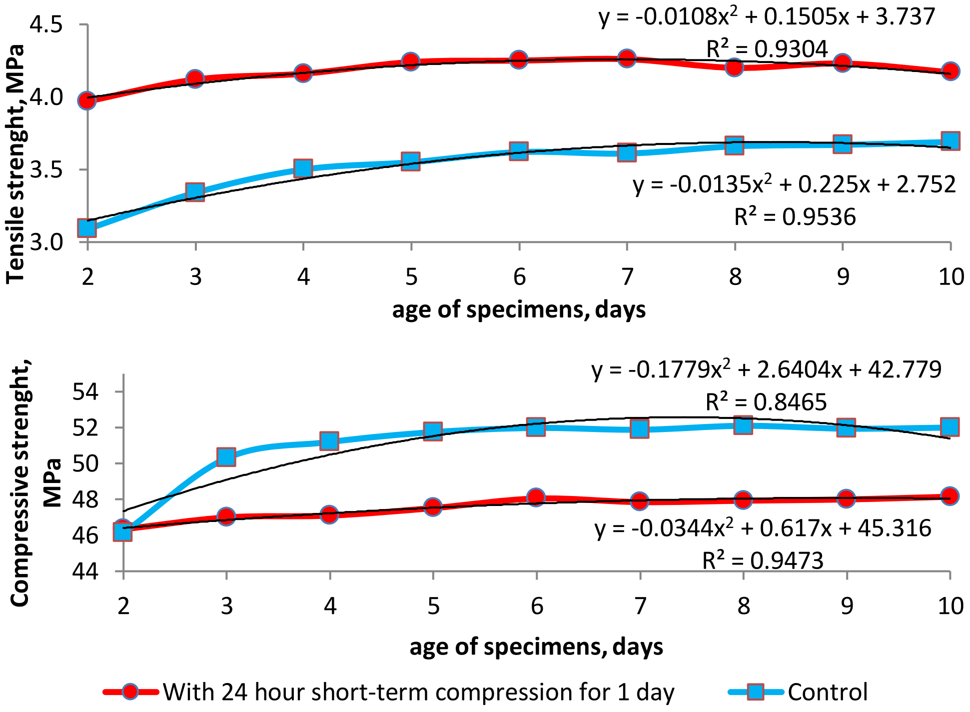
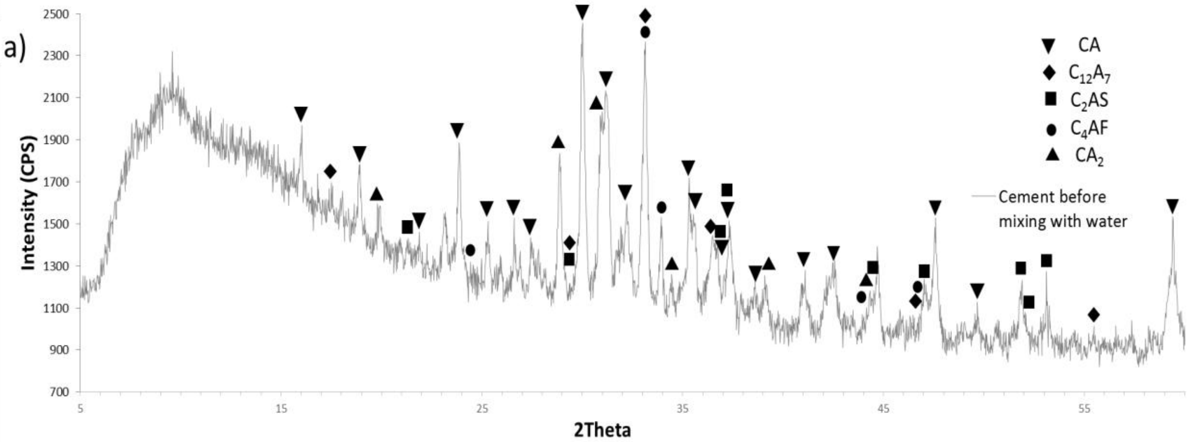


Figure 1. Effect of short-term compression on increase of compressive and bending strength

X-ray diffraction patterns of sample which was not exposed to loading (see red X-ray diffraction pattern in the Fig. 2b) indicates the presence of hydroaluminate C_3AH_6 (cubic phase) (2Θ = 19.89°; 22.6°; 26.8°; 39.1°; 44.47°), while the peak intensity related to C_3AH_6 in pre-loaded samples is lower. The aluminum hydroxide gel affects the stability of hexagonal hydroaluminates and reduces the tendency of recrystallization into cubic crystals [37]. Perhaps the increasing amorphousness (which was noted above) because of early compression leads to slowing down of recrystallization process and to the formation of cubic hydroaluminates, which caused these differences of X-ray diffraction patterns. AH_3 gel plays an important role in the strength increasing [36]. Higher tensile strength in bending (Figure 1) may result from changes in the structure occurred under load (the increasing amorphousness noted above due to the gel component). Interconnection between layers [34], compaction and change in porosity [39–43], which accompany deformation of cement systems, probably affect the bending strength characteristics.



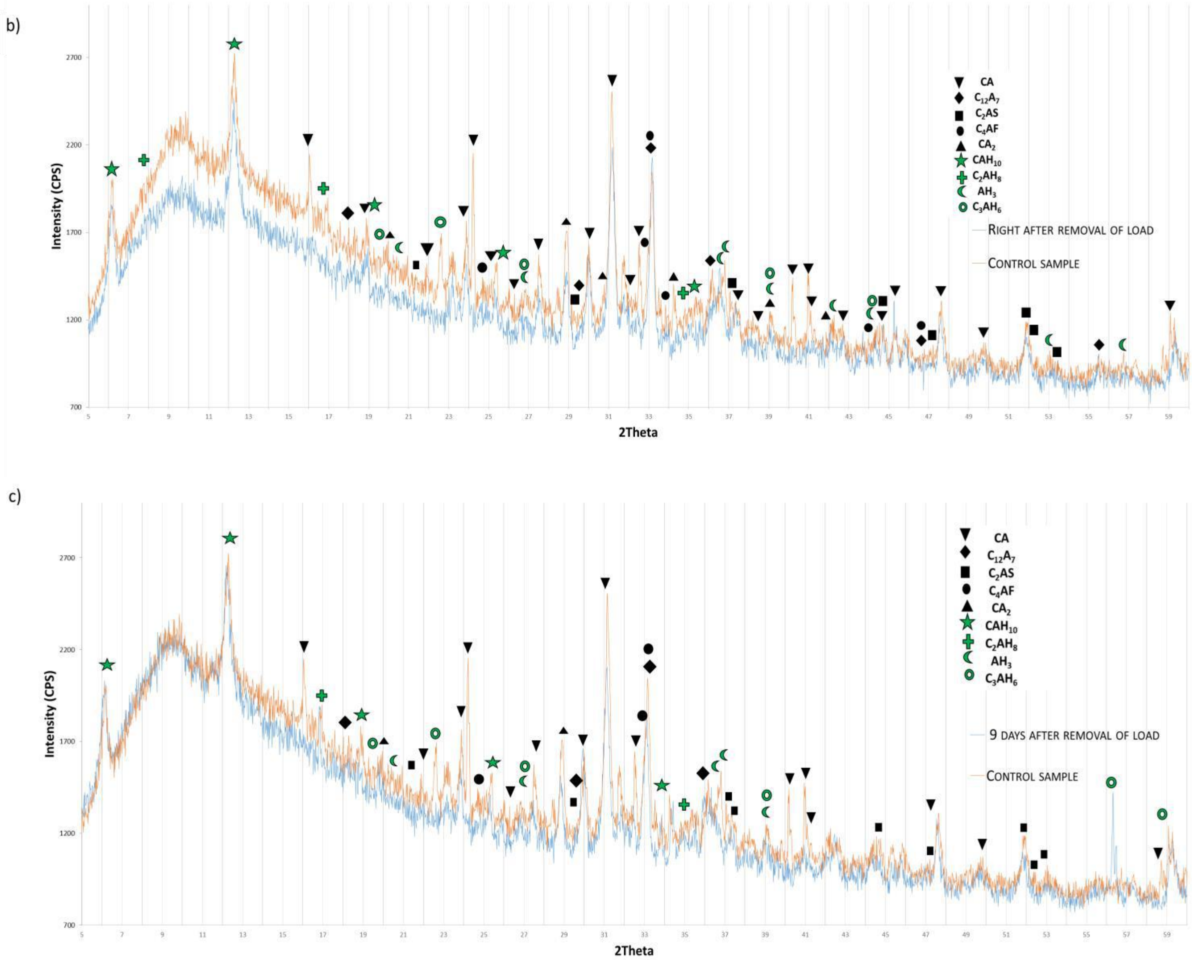


Figure 2. X-ray diffraction patterns of the cement GC-50 before mixing with water (a), immediately after removal of the load (b) and 9 days later (c)

A second analysis of the phase composition 9 days later showed that recrystallization of hexagonal hydroaluminates occurred in the cement stone under compression. Load removal and hardening of samples subjected to preliminary compression under absence of compressive stresses contributed to formation of cubic structure. It is confirmed by the appearance of a peak of cubic C_3AH_6 (peak at $2\Theta = 56.3°$ in Figure 2c) and increasing intensity of hydrate peaks (for example, microcrystalline gibbsite from $2\Theta = 20.58°$; $26.8°$). It is also appropriate to assume that the recrystallization rate under compressive stresses should be lower than in uncompressed structures. The delay in this process led, apparently, to a strength decrease of the specimens after 10 days of compression [2] (Fig. 1). An almost instantaneous partial transformation of the amorphous structure into a crystalline structure after removal of the load was indicated in researches [35–37, 42]. Taking the above into account, it should be noted that the short-term load at an early age changes the nature of structure formation processes that occur in aluminate systems.

According to the data of DTA (Fig. 3), it can be seen that the thermogravimetric curves of the samples practically coincide with each other. Despite the changes mentioned above (noted in the analysis of X-ray diffraction patterns), water content values in the structures of the pre-loaded and control samples are quite close. Coincidence of endo-effects at 275 °C (which corresponds to the temperature of boehmite formation [38]) may indicate that the weight percentage of the amorphous component of the control sample and pre-compressed sample is equalized within 10 days.

It is confirmed by overlaying of amorphous phase arcs in the angular range $2\Theta = 5\text{-}17°$ 9 days later (Fig. 2c). As S.V. Aleksandrovsky [44] indicated earlier, the water molecules in crystalline structures are loosely bound. Under certain conditions, they can be removed again, and then to be re-absorbed without changing the crystal structure.

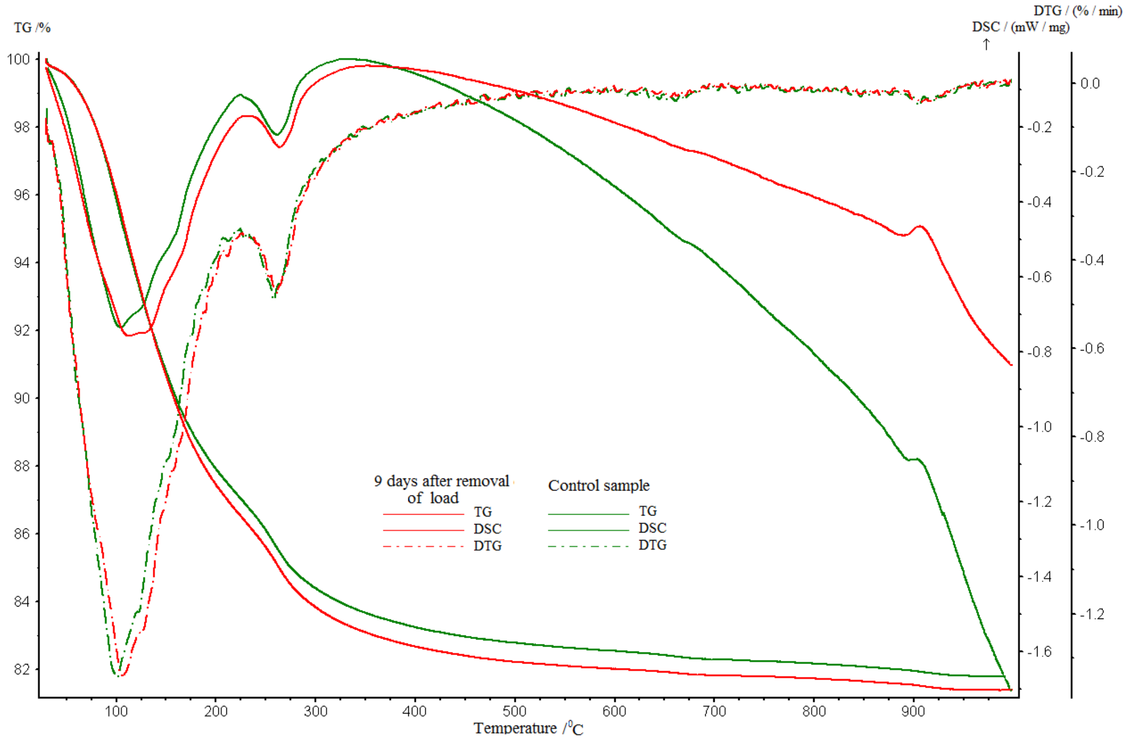


Figure 3. Comparison of the compressed and control specimens through differential thermal analysis 9 days after removal of the load

Conclusions

1. Preliminary short-term compression at an early age contributes to increase of bending tensile strength.

2. At the same time the compressive strength of the samples loaded at early age decreases, which may be due to the later recrystallization of hexagonal hydroaluminates into cubic phase and dumping of strength accompanying this process.

3. The peak intensity of the main phases of aluminate cement reduces in the case of load application. On the basis of the literature review, an assumption has been made that the amorphousness of the hydrated aluminate structure tends to increase under compressive stress.

4. Comparison of the derivatograms of the samples did not reveal any changes in the bound water content. The absence of such changes and simultaneous formation of a different crystal structure indicated by the X-ray analysis is of particular interest for the further study of aluminate cements after early loading.

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References

1. Scrivener K., Cabiron J., Letourneux R. High-perfomance concretes from calcium aluminate cements. *Cement and Concrete Research.* 1999. No. 29(8). Pp. 1215–1223.
2. Han B., Wang P., Ke C., Yan W., Wei Y., Li N. Hydration behavior of spinel containing high alumina cement from high titania blast furnace slag. *Cement and Concrete Research*. .2016. Vol. 79. Pp. 257–264.
3. Taylor H. *Himija cementa* [The Chemistry of Cement]. Moscow: Mir, 1996. 560 p. (rus).
4. Kuznetsova T.V. Glinozemistyy tsement [Alumina cement]. *Altinform: Tsement. Beton. Sukhiye smesi.* 2008. No. 2. Pp. 8–24. (rus).
5. Khaliq W., Khan H. High temperature material properties of calcium aluminate cement concrete. *Construction and*

Литература

1. Scrivener K., Cabiron J., Letourneux R. High-perfomance concretes from calcium aluminate cements // Cement and Concrete Research. 1999. № 29(8). Pp. 1215–1223.
2. Han B., Wang P., Ke C., Yan W., Wei Y., Li N. Hydration behavior of spinel containing high alumina cement from high titania blast furnace slag // Cement and Concrete Research. 2016. Vol. 79. Pp. 257–264.
3. Тейлор Х. Химия цемента. М.: Мир, 1996. 560 с.
4. Кузнецова Т.В. Глиноземистый цемент // Altinform: Цемент. Бетон. Сухие смеси. 2008. №2. С. 8–24.
5. Khaliq W., Khan H. High temperature material properties of calcium aluminate cement concrete // Construction and Building Materials. 2015. № 94. Pp. 475–487.

Building Materials. 2015. No. 94. Pp. 475–487.

6. Ukrainczyk N., Matusinovic T. Thermal properties of hydrating calcium aluminate cement pastes. *Cement and Concrete Research*. 2010. Vol. 40. Pp. 128–136.
7. Fujii K., Kondo W., Ueno H. Kinetics of hydration of monocalcium aluminate. *Journal of the American Ceramic Society*. 1986. Vol. 69. Pp. 74–82.
8. Abzayev Yu. A., Sarkisov Yu. S., Kuznetsova T. V., Samchenko S. V., Khlopotov V.D., Afanasyev D.A. Analiz strukturno-fazovogo sostoyaniya monoalyuminata kaltsiya [Analysis of the structural-phase state of calcium monoaluminate]. Magazine of Civil Engineering. 2014. No. 3. Pp. 56–62. (rus).
9. Ukrainczyk N. Kinetic modeling of calcium aluminate cement hydration. *Chemical engineering science*. 2010. Vol. 65. Pp. 5605–5614.
10. Guesta A. et al. Aluminum hydroxide gel characterization within a calcium aluminate cement paste by combined Pair Distribution Function and Rietveld analyses. *Cement and Concrete Research*. 2017. Vol. 96. Pp. 1–12.
11. Richard N., Lequewx N., Boch P. An X-ray absorption study of phases formed in high-alumina cements. *Advances in cement research*. 1995. No. 7. Pp. 159–169.
12. Antonovic V., Keriene J., Boris R., Aleknevicius M. The effect of temperature on the formation of the hydrated calcium aluminate cement structure. *Procedia Engineering*. 2013. No. 57. Pp.99–106.
13. Baiburin A. Technology of the early age concrete loading. *Procedia Engineering*. 2016. Vol. 150. Pp. 2157–2162.
14. Nehdi M., Soliman A. M. Early-age properties of concrete: overview of fundamental concepts and state-of-the-art research. *Construction materials, procc. of the Inst. of civ. eng*. 2011. Vol. 164. Pp. 1–21.
15. Claisse P., Dean C. Compressive strength of concrete after early loading. *Construction materials*. 2013. Vol. 166. Pp. 152–157.
16. Helmi M., Hall M., Stevens L., Rigby S. Effects of high-pressure / temperature curing on reactive powder concrete microstructure formation. *Construction and Building Materials*. 2016. Vol. 105. Pp. 554–562.
17. Shukenov I. I., Chalabayev B.M., Yerkinbekov A. Ye. Stend dlya stupenchatogo otpuska predvaritelnykh napryazheniy v protsesse teplovoy obrabotki pri proizvodstve zhelezobetonnykh konstruktsiy [Stand for the stepwise release of prestressing in the process of heat treatment in the manufacture of reinforced concrete structures]. *Tekhnologii betonov*. 2009. No. 2. Pp. 39–41. (rus)
18. Babich Ye. N., Makarenko L.P. Eksperimentalnyye issledovaniya modulya uprugosti betonnykh obraztsov pri razlichnoy intensivnosti szhimayushchikh nagruzok. [Experimental studies of the modulus of elasticity of concrete samples at different intensities of compressive loads]. *Izvestiya vuzov. Stroitelstvo i arkhitektura*. 1966. No. 7. Pp. 29–30. (rus).
19. Bezgodov I. M., Andrianov A. A. Vliyaniye dlitelnogo zagruzheniya na fiziko-mekhanicheskiye kharakteristiki vysokoprochnogo keramzitobetona [Influence of long loading on physicomechanical characteristics of high-strength expanded clay concrete]. *Tekhnologii betonov*. 2008. No. 8. Pp. 54–56. (rus).
20. Justs J., Bajare D., Korjakins A., Mezinskis G., Locs J., Bumanis G. Microstructural investigations of UHPC obtained by pressure application within the first 24 hours of hardening. *Construction Science*. 2013. No. 8. Pp. 50–57.
21. Liu G., Gao H., Chen F. Microstudy on creep of concrete at early age under biaxial compression. *Cement and Concrete Research*. 2002. Vol. 32. Pp. 1865–1870.
22. Koval S. B., Molodtsov M. V. Ranneye nagruzheniye betona v usloviyakh razlichnoy vlazhnosti [Early loading of

6. Ukrainczyk N., Matusinovic T. Thermal properties of hydrating calcium aluminate cement pastes // Cement and Concrete Research. 2010. Vol. 40. Pp. 128–136.
7. Fujii K., Kondo W., Ueno H. Kinetics of hydration of monocalcium aluminate // Journal of the American Ceramic Society. 1986. Vol. 69. Pp. 74–82.
8. Абзаев Ю.А., Саркисов Ю.С., Кузнецова Т.В., Самченко С.В., Хлопотов В.Д., Афанасьев Д.А. Анализ структурно-фазового состояния моноалюмината кальция // Инженерно-строительный журнал. 2014. № 3. С. 56–62.
9. Ukrainczyk N. Kinetic modeling of calcium aluminate cement hydration // Chemical engineering science. 2010. Vol. 65. Pp. 5605–5614.
10. Guesta A. et al. Aluminum hydroxide gel characterization within a calcium aluminate cement paste by combined Pair Distribution Function and Rietveld analyses // Cement and Concrete Research. 2017. Vol. 96. Pp. 1–12.
11. Richard N., Lequewx N., Boch P. An X-ray absorption study of phases formed in high-alumina cements // Advances in Cement Research. 1995. № 7. Pp. 159–169.
12. Antonovic V., Keriene J., Boris R., Aleknevicius M. The effect of temperature on the formation of the hydrated calcium aluminate cement structure // Procedia Engineering. 2013. № 57. Pp. 99–106.
13. Baiburin A. Technology of the early age concrete loading // Procedia Engineering. 2016. Vol. 150. Pp. 2157–2162.
14. Nehdi M., Soliman A. M. Early-age properties of concrete: overview of fundamental concepts and state-of-the-art research // Construction materials, procc. of the Inst. of civ. eng. 2011. Vol. 164. Pp. 1-21.
15. Claisse P., Dean C. Compressive strength of concrete after early loading // Construction materials. 2013. Vol. 166. Pp. 152–157.
16. Helmi M., Hall M., Stevens L., Rigby S. Effects of high-pressure / temperature curing on reactive powder concrete microstructure formation // Construction and Building Materials. 2016. Vol. 105. Pp. 554–562.
17. Шукенов И.И., Чалабаев Б.М., Еркинбеков А. Е. Стенд для ступенчатого отпуска предварительных напряжений в процессе тепловой обработки при производстве железобетонных конструкций // Технологии бетонов. 2009. № 2. С. 39–41.
18. Бабич Е. Н., Макаренко Л.П. Экспериментальные исследования модуля упругости бетонных образцов при различной интенсивности сжимающих нагрузок // Известия вузов. Строительство и архитектура. 1966. № 7. С. 29–30.
19. Безгодов И. М., Андрианов А. А. Влияние длительного загружения на физико-механические характеристики высокопрочного керамзитобетона // Технологии бетонов. 2008. № 8. С. 54–56.
20. Justs J., Bajare D., Korjakins A., Mezinskis G., Locs J., Bumanis G. Microstructural investigations of UHPC obtained by pressure application within the first 24 hours of hardening // Construction Science. 2013. № 8. Pp. 50–57.
21. Liu G., Gao H., Chen F. Microstudy on creep of concrete at early age under biaxial compression // Cement and Concrete Research. 2002. Vol. 32. Pp. 1865–1870.
22. Коваль С.Б., Молодцов М.В. Раннее нагружение бетона в условиях различной влажности // Вестник ЮУрГУ. 2011. № 16. С. 15–17.
23. Макаренко Л. П. Сопротивление бетона сжатию и растяжению после кратковременного и длительного сжатия различной интенсивности // Известия вузов. Строительство и архитектура. 1985. № 2. С. 8–11.
24. Кубанешвили А.С., Пирадов А. Б., Журатин А. М. Физико-механические свойства бетона, твердеющего под давлением в замкнутом пространстве // Бетон и железобетон. 2004. № 5. С. 11–13.

concrete in conditions of different humidity]. *Vestnik YuUrGU*. 2011. No. 16. Pp. 15–17. (rus).

23. Makarenko L. P. Soprotivleniye betona szhatiyu i rastyazheniyu posle kratkovremennogo i dlitelnogo szhatiya razlichnoy intensivnosti [Resistance of concrete to compression and stretching after short-term and long-term compression of various intensities]. *Izvestiya vuzov. Stroitelstvo i arkhitektura*. 1985. No. 2. Pp. 8–11. (rus).
24. Kubaneshvili A.S., Piradov A. B., Zhuratin A. M. Fiziko-mekhanicheskiye svoystva betona, tverdeyushchego pod davleniyem v zamknutom prostranstve [Physical and mechanical properties of concrete hardening under pressure in a confined space]. *Beton i zhelezobeton*. 2004. No. 5. Pp. 11–13. (rus).
25. Swamy R. N., Anand K.L. Behaviour of high alumina cement concrete under sustained loading. *Proceedings of ICE*. 1974. Vol. 57. Pp. 651–671.
26. Galkin Yu.Yu., Udodov S.A. Izucheniye poteri massy tsementnykh sistem pri szhatii [The study of mass loss of cement systems under compression]. *Inzhenernyy vestnik Dona.* 2017. No. 1. URL: ivdon.ru/en/magazine/archive/n1y2017/4064. (date of reference: 09.09.2017) (rus).
27. Wyrzykowski M., Lura P. The effect of external load on internal relative humidity in concrete. *Cement and concrete research*. 2014. Vol. 65. Pp. 58–63.
28. Altoubat S.A. Early-age stresses and creep–shrinkage and Interaction of Restrained Concrete. *PhD thesis, University of Illinois at Urbana-Champaign, United States.* 2002. 125 p.
29. Feldman R.F. Mechanism of creep of hydrated Portland cement paste. *Cement and concrete research*, 1972. Vol. 2. Pp. 521–540.
30. Moon J., Oh J., Balonis M., Glasser F., Clark S.M. Pressure induced reactions amongst calcium aluminate hydrate phases. *Cement and concrete research*. 2011. Vol. 41. Pp. 571–578.
31. Acker P. Swelling, shrinkage and creep: a mechanical approach to cement hydration. *Concrete science and engineering*. 2004. Vol. 37. Pp. 237–243.
32. Bentur A., Milestone N.B., Young J.F. Creep and drying shrinkage of calcium silicate pastes II. Induced microstructural and chemical changes. *Cement and concrete research*. 1978. Vol. 8. Pp. 721–732.
33. Muller H. S., Eckhardt, Haist M. New experimental approach to study creep and shrinkage mechanisms of concrete on the nano-scale level. Mechanics and physics of creep, shrinkage and durability mechanics. *Procc. of 9th inter. conference. ASCE*. 2013. 507p.
34. Glukhovskiy V. D., Runova R. F., Maksunov S. Ye. *Vyazhushchiye i kompozitsionnyye materialy kontaktnogo tverdeniya*. [Knitting and composite materials of contact hardening]. K.: Vishcha shkola, 1991. 243 p. (rus).
35. Clark S. M., Colas B., Kunz M., Speziale S., Monteiro P.-J. M. Effect of pressure on the crystal structure of ettringite. *Cement and concrete research*. 2008. Vol. 38. Pp. 19–26.
36. Cuesta A., Rejmak P., Ayuela A. Experimental and theoretical high pressure study of calcium hydroxyaluminate phases. *Cement and concrete research*. 2017. Vol. 97. Pp. 1–10.
37. Lothenbach B., Pelletier- Chaignat L., Winnefeld F. Stabilty in the system CaO-Al_2O_3-H_2O. *Cement and concrete research*. 2012. Vol. 42. Pp. 1621–1634.
38. Kuznetsova T. V. *Alyuminatnyye i sulfoalyuminatnyye tsementy* [Aluminate and sulphoaluminous cements]. M.: Stroyizdat, 1986. 207 p. (rus).
39. Hope B.B. A model for the creep of concrete. *Cement and concrete research*. 1975. Vol. 5. Pp. 577–586.
40. Pignatelli I., Kumar A., Alizabeh R., Pape Y. L. Dissolution – precipitation mechanism is at the origin of concrete creep in moist environments. *The journal of chemical*

25. Swamy R.N., Anand K.L. Behaviour of high alumina cement concrete under sustained loading // Proceedings of ICE. 1974. Vol. 57. Pp. 651–671.
26. Галкин Ю.Ю., Удодов С.А. Изучение потери массы цементных систем при сжатии [Электронный ресурс] // Инженерный вестник Дона. 2017. № 1. URL: ivdon.ru/en/magazine/archive/n1y2017/4064. (дата обращения: 09.09.2017)
27. Wyrzykowski M., Lura P. The effect of external load on internal relative humidity in concrete // Cement and concrete research. 2014. Vol. 65. Pp. 58–63.
28. Altoubat S.A. Early-age stresses and creep–shrinkage and Interaction of Restrained Concrete. PhD thesis, University of Illinois at Urbana-Champaign. United States. 2002. 125 p.
29. Feldman R.F. Mechanism of creep of hydrated Portland cement paste // Cement and concrete research. 1972. Vol. 2. Pp. 521–540.
30. Moon J., Oh J., Balonis M., Glasser F., Clark S. M. Pressure induced reactions amongst calcium aluminate hydrate phases // Cement and concrete research. 2011. Vol. 41. Pp. 571–578.
31. Acker P. Swelling, shrinkage and creep: a mechanical approach to cement hydration // Concrete science and engineering. 2004. Vol. 37. Pp. 237–243.
32. Bentur A., Milestone N.B., Young J.F. Creep and drying shrinkage of calcium silicate pastes II. Induced microstructural and chemical change // Cement and concrete research. 1978. Vol. 8. Pp. 721–732.
33. Muller H. S., Eckhardt, Haist M. New experimental approach to study creep and shrinkage mechanisms of concrete on the nano-scale level // Mechanics and physics of creep, shrinkage and durability mechanics. Procc. of 9th inter. conference. ASCE. 2013. 507p.
34. Глуховский В.Д., Рунова Р.Ф., Максунов С.Е. Вяжущие и композиционные материалы контактного твердения. К.: Вища школа, 1991. 243 с.
35. Clark S.M., Colas B., Kunz M., Speziale S., Monteiro P.-J.M. Effect of pressure on the crystal structure of ettringite // Cement and concrete research. 2008. Vol. 38. Pp. 19–26.
36. Cuesta A., Rejmak P., Ayuela A. Experimental and theoretical high pressure study of calcium hydroxyaluminate phases // Cement and concrete research. 2017. Vol. 97. Pp. 1–10.
37. Lothenbach B., Pelletier- Chaignat L., Winnefeld F. Stabilty in the system CaO-Al_2O_3-H_2O // Cement and concrete research. 2012. Vol. 42. Pp. 1621–1634.
38. Кузнецова Т. В. Алюминатные и сульфоалюминатные цементы. М.: Стройиздат, 1986. 207 с.
39. Hope B.B. A model for the creep of concrete // Cement and concrete research. 1975. Vol. 5. Pp. 577–586.
40. Pignatelli I., Kumar A., Alizabeh R., Pape Y. L. Dissolution – precipitation mechanism is at the origin of concrete creep in moist environments // The journal of chemical physics. 2016. Vol. 145. http://dx.doi.org/10.1063/1.4955429. дата обращения: 09.09.2017)
41. Justs J., Bajare D., Korjakins A. Microstructural investigations of UHPC obtained by pressure application within the first 24 hours of hardening // Construction science. 2013. Vol. 8. Pp. 50–57.
42. Helmi M., Hall M. R., Stevens L. A., Rigby S., Effects of high pressure/temperature curing on reactive powder concrete microstructure formation // Construction and building materials. 2016. Vol. 105. Pp. 554–562.
43. Brooks J.J. Concrete and masonry movements. Oxford: Elsevier, 2015. 599 p.
44. Александровский С.В. Расчет бетонных и железобетонных конструкций на температурные и

physics. 2016. Vol. 145. http://dx.doi.org/10.1063/1.4955429. (date of reference: 09.09.2017)

41. Justs J., Bajare D., Korjakins A. Microstructural investigations of UHPC obtained by pressure application within the first 24 hours of hardening. *Construction science*. 2013. Vol. 8. Pp. 50–57.
42. Helmi M., Hall M. R., Stevens L. A., Rigby S., Effects of high pressure/temperature curing on reactive powder concrete microstructure formation. *Construction and building materials*. 2016. Vol. 105. Pp. 554–562.
43. Brooks J.J. *Concrete and masonry movements*. Oxford: Elsevier, 2015. 599p.
44. Aleksandrovskiy S.V. *Raschet betonnykh i zhelezobetonnykh konstruktsiy na temperaturnyye i vlazhnostnyye vozdeystviya* [Calculation of concrete and reinforced concrete structures for temperature and humidity effects]. M.: Stroyizdat, 1966. 443 p. (rus).

влажностные воздействия. М.: Стройиздат, 1966. 443 с.

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Dome houses made of soil-concrete based on local raw materials

Купольные дома из грунтобетона на основе местного сырья

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Key words: soil-concrete; magnesia binder; dome houses; strengthening of soil foundation; water resistance

Ключевые слова: грунтобетон; магнезиальное вяжущее; купольные дома; укрепление основания; водостойкость

Abstract. The work presents the results of research on the strengthening of soil basement in the center of the urban nucleus of Vladimir, which are loam, sandy loam in soft-plastic state, and also sandy loam with silt. The technology of applying the strengthening composition in the soil has been developed. The results of the investigation of physical and mechanical properties of soils and their water resistance have been presented. The microstructure of the concrete has been studied and compositions of masonry material suitable for the construction of residential premises have been developed. The advantages and prospects of development of domed buildings have been studied. The expediency of using domed buildings in terms of energy efficiency has been substantiated. The possibility of using the developed building materials for the construction of domed buildings has been considered.

Аннотация. В работе представлены результаты исследований по укреплению грунтового основания в центре городского ядра Владимира, который представляет собой суглинки, супеси в мягкопластическом состоянии, а также супесь с илом. Разработана технология применения упрочняющей композиции в почве. Приведены результаты исследования физико-механических свойств полученного материала и его водостойкости. Изучена микроструктура грунтобетона и разработаны материалы кладки, подходящие для строительства жилых зданий. Изучены достоинства купольных зданий, перспективы развития. Обоснована целесообразность применения куполообразных зданий с точки зрения энергоэффективности. Рассмотрена возможность применения разработанных строительных материалов при строительстве купольных зданий.

Introduction

The paper presents research data on strengthening of soil foundation (such as loam, sandy loam in soft-plastic state, and also sandy loam with silt) in the center of the urban nucleus of Vladimir city. This research includes manufacturing technology development of applying the strengthening composition in the soil and compositions for masonry materials suitable for the construction of residential premises. The scientific work shows research data of physical and mechanical properties of soils as well as their water resistance. Additionally, there has been studied concrete microstructure and justified the perspective of domed buildings from the standpoint of energy efficiency.

There are a lot of examples of housing made of soil in almost all countries of the world, including Russia (Priory Palace in Gatchina, built in 1798). The application of such material is governed by its strength, environmental friendliness and cost effectiveness [1].

Закревская Л.В., Любин П.А., Авдеев С.Н., Гандельсман И.А., Филиппов С.В. Купольные дома из грунтобетона на основе местного сырья // Инженерно-строительный журнал. 2017. № 7(75). С. 123–128.

Nowadays, when it seems that there is nothing left to invent in the construction field, the global issue to improve the energy performance of buildings and structures remains open. From the energy point of view, the determining factor is the shape of the building. As it is known, dome is one of the most stable shapes in nature. To increase the adhesion of the dome to the base surface, the spoiler effect is used. The dome (bubble) house, erected from compact-grained soil-concrete is a stable structure [2].

Energy performance of the domed house lies in the shape of the spherical structure. There is an inverse dependence between the area of the external face of the structure and energy consumption for room heating. The less the area of walls and roof is, the higher the performance coefficient is. The heat loss through the building footing depends on the ratio of the perimeter of the floor to its area. In this case, the dome, having a smaller perimeter-to-area ratio, in comparison with a conventional house, gains the upper hand [3–5].

One of the advantages of domed buildings is that wall structures and roofing are made of the same materials. It gives the possibility to put them up in a single cycle. This fact in combination with full mechanization and automation makes it possible to reduce the construction period and the cost of the erected building. As a result, one of the promising areas of domed construction is the erection of above-ground structures by means of 3D printing. In the course of this study, there was considered a possibility of a single cycle construction with the use of a 3D printer. While the roofing is being assembled, the mortar layers will be compressed and will not allow the entire roof to collapse even in the process of construction without auxiliary devices such as formwork and other supporting tools. However, in sloping roof structures the mortar has to have an extremely high setting speed to prevent local dropout of the material. In addition, a precise adjustment of the printer speed is required [6–8].

There have been created a number of models, which confirmed the possibility of building construction by means of a 3D printer in a single cycle.

Figure 1 shows a dome house model printed on a 3D printer.

Figure 1. Dome house model printed on a 3D printer

One of the main factors affecting durability and reliability of buildings is the stability of the footing on which the building is erected. The reason for footing instability is the state of the soil, depending on its origin [9, 10].

Nowadays, cities are developing at a quick rate, and the most favorable areas have already been "covered" (built up) with industrial and civil buildings. However, having examined a city map in details, almost everywhere one can see "white spots" i.e. city areas that do not have any buildings. The reason is the occurrence in these territories of soft soils such as clay, loam, sandy loam, and peat. These types of soils are subject to mandatory strengthening before building any structures on it. In Vladimir city there are a number of undeveloped sites that are vivid examples of such "white spots" [11–13].

Methods

In this paper there was set a task of integrated use of soil and a magnesia binder derived from dolomite raw materials to construct spherical buildings. There was made an attempt to strengthen foundation soil as well as to create a structural material for the erection of the aboveground part of the dome building.

Man-made soil formed on the territory of the former brick factory in the central part of Vladimir (in the vicinity of the urban nucleus) was selected as weak soil for strengthening. Due to the low bearing capacity of the foundation, it is impossible to build by traditional methods on this territory. Therefore, there was made an attempt to use the best properties of the magnesia binder and clay soil [14–16].

Table 1 shows characteristics of the investigated foundation at one of the construction sites.

Table 1. Characteristics of the investigated foundation at one of the construction sites

Number of engineering-geological element	Classification of soils according to National State Standard 25100-2011	Normative values										
		Moisture, u.s.			Plastic index, Ip, u.s.	Soil density, ρ, G / cm^3	Porosity fractions, e	Saturation degree, Sr, u.s.	Soil permeability, M/day	Shearing strength		Stiffness modulus
		Natural, W	At the yield point, WL	At the rolling edge, Wp						Angle of internal friction	Specific cohesion	
1	Made-up soil	No limitation										
2	Fine sand, quartz, loose	0.186	Wet			1.79	0.76	0.65	1.0-4.7	27	-	17
		0.257	Water-saturated			1.90		0.90				
3	Stiff loam	0.219	0.314	0.163	0.151	1.89	0.654	0.86	<0.1	14	0.0210	11
3a	Very soft loam	0.286	0.302	0.157	0.145	1.97	0.648	1.0	<0.1	19	0.0119	8

Dolomite is a mineral from the carbonate class with the following chemical composition CaCO3 • MgCO3. The authors obtained a magnesia binder by the semi-sintering of dolomite powder (which is a mining waste) from the Melekhovsky deposit. The obtained binder contains 28 % of MgO [17, 18].

The introduction of semi-sintered dolomite waste into the clay soil makes it possible to change the colloidal and chemical properties of the foundation for strengthening due to pore saturation with magnesium cations. The hydration process of the mixture proceeds under the most favorable conditions for hardening. The use of semi-sintered dolomite waste can improve the strength of the foundation. [19, 20]

In the course of the work there were investigated compositions of soils strengthened by the magnesia binder (semi-sintered dolomite). To obtain an optimal clay-magnesia ratio there was developed an array of compositions which were tested for strength, water and frost resistance.

Results and Discussion

Table 2 presents the compositions of the synthesized samples.

Table 2. The compositions of the synthesized samples

Soil-concrete composition	Clay soil, g	Semi-sintered dolomite, g	Bishofite, ml	Water, ml	KH2PO4, g	Water resistance
SC-50	500	500	100	145	55	0.78
SC-40	600	400	75	170	60	0.81
SC-30	700	300	60	205	65	0.84
SC-20	800	200	40	205	70	0.93

Bar chart 1 illustrates the strength test data for the investigated compositions on the 28th day of their hardening, depending on clay-magnesia binder ratio.

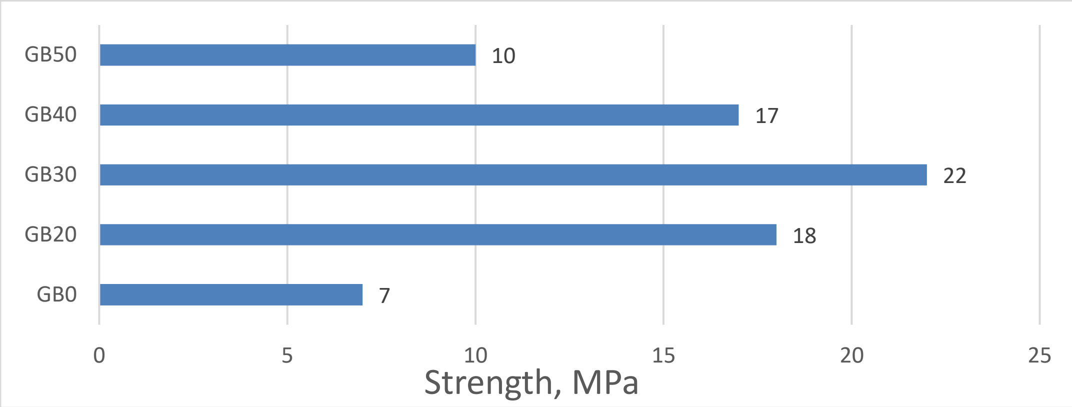

Figure 2. Bar chart 1. The strength test data for the investigated compositions on the 28th day of their hardening, depending on clay-magnesia binder ratio

As we can see from Table 2 and bar chart 1, the most optimal composition is SC-30 containing 700 g of clay soil, 300 g of semi-sintered dolomite, 60ml of bischofite, 205 ml of water, 65 g of potassium dihydrogen phosphate.

The strengthened foundation is a soil conglomerate containing hydro silicates and hydro alliumates. Potassium hydrogenphosphate binding of water, which is found in the soil leads to an increase in water resistance of the soil and, as a result, increases its strength.

In the course of the work there was set a task to obtain a structural material from the soil to construct spherical buildings. For this purpose, the authors carried out studies on the creation of wall materials both for concrete blocks and mortars for building technology using a 3D printer. The composition GB-30 is the most suitable for 3D printer technologies. With the aim to obtain soil-blocks as a masonry material there were developed compositions listed in Table 3.

Table 3. Composition of soil blocks

Soil-concrete composition	Clay, g	Semi-sintered dolomite, g	Bischofite with the addition of asbestos, ml	Wood aggregate, g	KH2PO4, g.
SC-25	700	250	200	50	60
SC-20	700	200	200	100	65
SC-15	700	150	200	150	70
SC-0	700	0	0	300	0

The composition of masonry soil-concrete except for clay soil includes:

Magnesia binder, containing 28 % of MgO.

Bishofite with the density of 1.2 g/cm^3, which is MgSO4 as a grouting fluid

Bun or sawdust as a filler

Single-substituted potassium dihydrogen phosphate used as an additive that increases water resistance.

The main criteria to evaluate the synthesized compounds are mechanical strength (Rc) and water resistance. Bar chart 2 presents the investigated properties. The obtained samples, with the exception of SC-0, show water resistance higher than 0.8.

The results of strengthening man-made soils completely consistent with the concept of authors [13, 17–19] and served as the basis for the creation construction material which was not used in 3D technology before. With the aim of creating environmentally friendly building materials on the basis of the developed compositions with the introduction of an organic lightweight aggregate, in 2017-18 the expansion of the study is planned.

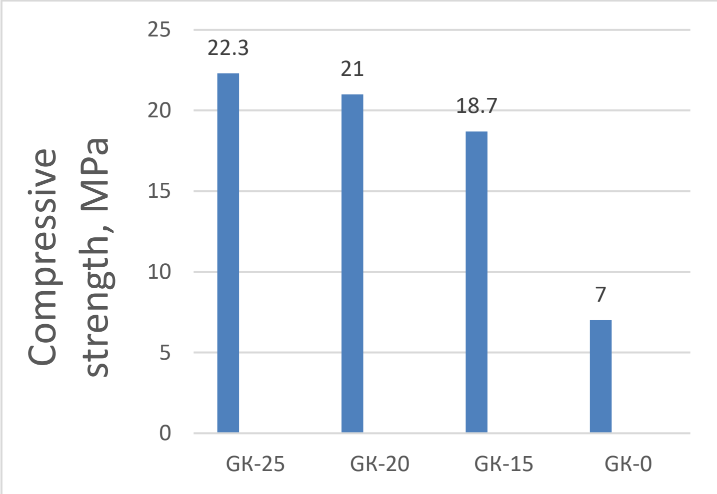

Figure 3. Bar chart 2. The investigated properties

Conclusions

This paper investigates the possibility to use soft man-made clay soils as a natural foundation, strengthened by semi-sintered dolomite waste; there have been selected components for composite mixing and improvement of its water resistance. There have been created samples of constructional materials as well as provided examples of complex approach to use domestically-produced raw materials to create non-waste technologies for the construction of energy-efficient housing using the technology of erecting buildings on a 3D printer. There have been considered the possibility of erecting buildings on a 3D printer in a single cycle. This research makes it possible to predict the availability to expand the resource base in the construction industry and to use previously unsuitable sites for construction purposes.

References

1. Gardiner J.B. *PhD Thesis – Exploring the Emerging Design Territory of Construction 3D Printing*. RMIT University, 2011. 382 p.
2. Müller E. Building Blocks as an Approach for the Planning of Adaptable Production Systems. *Advanced Manufacturing and Sustainable Logistics.* Springer Berlin Heidelberg, 2010. Vol. 46. Pp. 37–45.
3. Johnston W.D. Design and Construction of Concrete Formwork. *Concrete Construction Engineering Handbook*. 2008. Pp. 7–49.
4. Lloret E. et al. Complex concrete structures: Merging existing casting techniques with digital fabrication. *ComputerAided Design.* 2015. No. 60. Pp. 40–49.
5. Di Carlo T., Khoshnevis B., Carlson A. Experimental And numerical techniques to characterize structural properties of fresh concrete. *ASME 2013 International Mechanical Engineering Congress and Exposition. American Society of Mechanical Engineer.* 2013. Pp. 62–64.
6. Warszawski A., Navon R. Implementation of robotics in building: Current status and future prospects. *Journal of Construction Engineering and Management.* 1998. No. 124(1). Pp. 31–41.
7. Ibrahim M.I.M. Estimating the sustainability returns of recycling construction waste from building projects. *Sustainable Cities and Society.* 2016. No. 23. Pp. 78–93.
8. Chen L. et al. The research status and development trend of additive manufacturing technology. *The International Journal of Advanced Manufacturing Technology.* 2016. Pp. 1–10.
9. Gálvez J. A. Not just a pretty face: three-dimensional printed custom airway management devices. *3D Printing and Additive Manufacturing.* 2016. Vol. 3. No. 3. Pp. 160–165.
10. Kumar L.J., Nair C.G. Current Trends of Additive

Литература

1. Gardiner J.B. PhD Thesis – Exploring the Emerging Design Territory of Construction 3D Printing. RMIT University, 2011. 382 p.
2. Müller E. Building Blocks as an Approach for the Planning of Adaptable Production Systems // Advanced Manufacturing and Sustainable Logistics. Springer Berlin Heidelberg, 2010. Vol. 46. Pp. 37–45.
3. Johnston W.D. Design and Construction of Concrete Formwork // Concrete Construction Engineering Handbook. 2008. Pp. 7–49.
4. Lloret E. et al. Complex concrete structures: Merging existing casting techniques with digital fabrication // ComputerAided Design. 2015. № 60. Pp. 40–49.
5. Di Carlo T., Khoshnevis B., Carlson A. Experimental And numerical techniques to characterize structural properties of fresh concrete // ASME 2013 International Mechanical Engineering Congress and Exposition. American Society of Mechanical Engineer. 2013. Pp. 62–64.
6. Warszawski A., Navon R. Implementation of robotics in building: Current status and future prospects // Journal of Construction Engineering and Management. 1998. № 124(1). Pp. 31–41.
7. Ibrahim M.I.M. Estimating the sustainability returns of recycling construction waste from building projects // Sustainable Cities and Society. 2016. № 23. Pp. 78–93.
8. Chen L. et al. The research status and development trend of additive manufacturing technology // The International Journal of Advanced Manufacturing Technology. 2016. Pp. 1–10.
9. Gálvez J. A. Not just a pretty face: three-dimensional printed custom airway management devices // 3D Printing and Additive Manufacturing. 2016. Vol. 3. № 3. Pp. 160–165.
10. Kumar L.J., Nair C.G. Current Trends of Additive

Manufacturing in the Aerospace Industry. *Advances in 3D Printing & Additive Manufacturing Technologies.* Springer Singapore. 2016. Pp. 39–54.

11. Kreiger M. A., MacAllister B. A., Wilhoit J. M., Case M. P. The current state of 3D printing for use in construction. *The Proceedings of the 2015 Conference on Autonomous and Robotic Construction of Infrastructure*. Ames. Iowa. 2015. Pp. 149–158.
12. Chernykh T.N. Osobennosti tverdeniya magnezialnogo vyazhushchego [Features of hardening magnesia astringent]. *Cement and its Applications*. 2006. No. 5. Pp. 58–61. (rus)
13. Zvezdina Ye.V., Treskova N.V. Povysheniye vodostoykosti teploizolyatsionnykh izdeliy na osnove kausticheskogo dolomita [Increase of water resistance of heat-insulating products on the basis of caustic dolomite]. *Nauchno-prakticheskiy Internet-zhurnal «Nauka. Stroitelstvo. Obrazovaniye».* 2011. No. 1. (rus)
14. Korjakins A., Shakhmenko G., Bajare D., Bumanis G. Application a dolomite waste as filler in expanded clay lightweight concrete. *10th International Conference Modern Building Materials, Structures and Techniques.* 2010. Pp. 156–161.
15. Chekhovskiy Yu.V., Spitsyn A.N., Kardash Yu.A., Aliyev A.D., Chalykh A.Ye. Issledovaniye kontaktnoy zony tsementnogo kamnya s krupnym zapolnitelem [Investigation of the contact zone of cement stone with a large aggregate]. *Colloid Journal.* 1988. No. 6. Pp. 1216–1218. (rus)
16. Chernykh, T., Nosov A., Kramar L. Dolomite magnesium oxychloride cement properties control method during its production. *IOP Conference Series: Materials Science and Engineering.* 2015. Vol. 71. No. 1. Article number 012045.
17. Chernykh T. Energy-saving magnesium oxychloride cement intensifier. *SGEM International Multidisciplinary Scientific Conference on Social Sciences and Arts.* 2015. Pp. 359–363.
18. Chernykh T., Orlov A., Kramar L. Hydration features of magnesium sulfate compositions. *SGEM International Multidisciplinary Scientific Conference on Social Sciences and Arts.* 2015. Pp. 463–468.
19. Sizikov A.M., Shapovalova Ye.V. Puti povysheniya kachestva magnezialnykh betonov [Ways to improve the quality of magnesia concrete.]. Omsk: SibADI, 2009. 92 p. (rus)
20. Ip K., Miller A. Life cycle greenhouse gas emissions of hemp – lime wall constructions in the UK. *Resources, Conserv. Recycl.* 2012. Vol. 69. Pp. 1–9.

Manufacturing in the Aerospace Industry // Advances in 3D Printing & Additive Manufacturing Technologies. Springer Singapore. 2016. Pp. 39–54.

11. Kreiger M. A., MacAllister B. A., Wilhoit J. M., Case M. P. The current state of 3D printing for use in construction // The Proceedings of the 2015 Conference on Autonomous and Robotic Construction of Infrastructure. Ames. Iowa. 2015. Pp. 149–158.
12. Черных Т.Н. Особенности твердения магнезиального вяжущего // Цемент и его применение. 2006. №5. С. 58–61.
13. Звездина Е.В., Трескова Н.В. Повышение водостойкости теплоизоляционных изделий на основе каустического доломита // Научно-практический Интернет-журнал «Наука. Строительство. Образование». 2011. № 1.
14. Korjakins A., Shakhmenko G., Bajare D., Bumanis G. Application a dolomite waste as filler in expanded clay lightweight concrete // 10th International Conference Modern Building Materials, Structures and Techniques. 2010. Pp. 156–161.
15. Чеховский Ю.В., Спицын А.Н., Кардаш Ю.А., Алиев А.Д., Чалых А.Е. Исследование контактной зоны цементного камня с крупным заполнителем // Коллоидный журнал. 1988. № 6. С. 1216–1218.
16. Chernykh T., Nosov A., Kramar L. Dolomite magnesium oxychloride cement properties control method during its production // IOP Conference Series: Materials Science and Engineering. 2015. Vol. 71. № 1. Article number 012045
17. Chernykh T. Energy-saving magnesium oxychloride cement intensifier // SGEM International Multidisciplinary Scientific Conference on Social Sciences and Arts. 2015. Pp. 359–363.
18. Chernykh T., Orlov A., Kramar L. Hydration features of magnesium sulfate compositions // SGEM International Multidisciplinary Scientific Conference on Social Sciences and Arts. 2015. Pp. 463–468.
19. Сизиков А.М., Шаповалова Е.В. Пути повышения качества магнезиальных бетонов. Монография. Омск: СибАДИ, 2009. 92 с.
20. Ip K., Miller A. Life cycle greenhouse gas emissions of hemp – lime wall constructions in the UK // Resources, Conserv. Recycl. 2012. Vol. 69. Pp. 1–9.

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Zakrevskaya L.V., Lubin P.A., Avdeev S.N., Gandelsman I.A., Filippov S.V. Dome houses made of soil-concrete based on local raw materials. *Magazine of Civil Engineering*. 2017. No. 7. Pp. 123–128. doi: 10.18720/MCE.75.12.

doi: 10.18720/MCE.75.13

Modeling the design seismic input in conditions of limiting seismological information

Моделирование расчетного сейсмического воздействия в условиях ограниченной сейсмологической информации

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Key words: seismic input; input characteristics; peak ground acceleration; harmonious ratio; Arias intensity; cumulative absolute velocity; seismic energy density

Ключевые слова: сейсмическое воздействие; генерация воздействия; характеристики землетрясения; пиковые ускорения основания; коэффициент гармоничности; интенсивность по Ариасу; кумулятивная абсолютная скорость; плотность сейсмической энергии

Abstract. The object of the investigations is seismic input models used in structure designing under the conditions of limited seismological information. The aim of the given investigation is to propose a new variant of a seismic input model, which should be generated for the structure under consideration, taking into account main peculiarities of actual seismic excitations. The method of design seismic input generation by means of its presentation as a sum of velocity impulse and multi frequency excitation has been developed. The duration and peak value are parameters of velocity impulse. They can be presented as function of possible earthquake magnitude and hypocentral distance. Multi frequency excitation can be presented as a product of sinusoid and some envelope function. Parameters of the impulse and multi frequency excitation are determined to provide accordance of generated input characteristics with characteristics of past earthquakes. Characteristics of past earthquakes were estimated using the joined database including more than 100 records of strong earthquakes presented by Chinese and Russian experts.

Аннотация. Предмет исследований – модели сейсмических воздействий, используемые при проектировании сооружений в условиях ограниченной сейсмологической информации. Цель выполненного исследования – предложить новый вариант модели сейсмического воздействия, которая должна генерироваться для рассматриваемой конструкции с учетом основных особенностей реальных сейсмических воздействий. Развит метод генерации расчетного сейсмического воздействия, использующий его представление в виде суммы импульса скорости и

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поличастотного возмущения. Параметрами импульса скорости являются его продолжительность и пиковое значение. Они могут быть представлены как функции возможной магнитуды землетрясения и гипоцентрального расстояния. Полигармоническое возмущение представляет собой произведение синусоиды на некоторую огибающую. Параметры импульса и поличастотного воздействия определяются так, чтобы характеристики сгенерированного воздействия соответствовали натурным характеристикам прошлых землетрясений. Характеристики реальных воздействий оценены с использованием общей базы данных российских и китайских специалистов, включающей более 100 записей 9-ти бальных землетрясений.

Introduction

At present two models of seismic input are used in earthquake engineering practice [1]. The first model is based on generating the input for the building site, and the second model is based on generating the input for structure. There is also a group of intermediate models in which the input is generated for a given response spectrum. If this spectrum is set for a building site, then these models belong to the first group and if the spectrum is given for a construction, then they belong to the second group. Finally, the spectral curve of Guidelines can be taken as the spectrum. Then the input model will have all the errors inherent in the normative spectral curve. This question was discussed in detail in [2, 3].

Input models for the building site were built by many experts [5–11]. These models are advanced by seismologists and require seismological information for the building site. Although this approach to generating the input seems quite logical, it raises some objections from engineers.

First, in engineering practice, the necessary seismological information is not always available. Generally it is used in designing nuclear power plants and large dams. Even in designing the world largest cable-stayed bridge across the Easten Bosporus to the Russian Island, there was no complete seismological information, and when designing transport Olympic facilities in Sochi, the necessary information had been received only by the end of the designing process, when all design decisions have been made.

Secondly, building engineers do not always trust seismological information. This is due to the fact that out of 27 strong earthquakes that have been place in the former USSR since 1948, 24 occurred in areas previously considered to be non-seismic or weakly seismic.

Thirdly, structures can be rather sensitive to small changes in the response spectrum. Sometimes, a 5% change in the input prevailing period of the spectrum, which is quite possible within the framework of seismological studies, leads to a twofold change of seismic loads. Metal structures are particularly sensitive to these changes.

Fourthly, the generation of input for the building site is completely unfit for a typical designing, when the structure is to be earthquake-proof on any sites.

It is these shortcomings that led to the development of input models with a spectrum that repeats the normative spectral curve. It should be noted here that for the given spectrum one can generate infinitely many inputs [1, 8]. Besides it should be capped in mind the normative spectral curve itself contains errors which in the Guidelines are balanced by errors in specifying the design peak accelerations and errors in the damping task [1, 2], but not balanced in design accelerograms. For these reasons, this approach can lead to unpredictable results.

An alternative to generating input for the building site is the second model of seismic input, i.e. generation of input for the construction. In this case, the properties of the building site are ignored completely or partially, and the most hazardous input is determined for the structure. For linear structure models, the input model is given as a resonant one for the structure. For structures in the inelastic stage of behavior, dangerous input frequencies are determined iteratively. This process is described, for example, in [12].

The aim of the given investigation is to propose a new variant of a seismic input model, which should be generated for the structure under consideration, taking into account main peculiarities of actual seismic excitations.

The authors develop a well-known approach to the generation of seismic input for structures used for a standardized design, when the structure must be earthquake-proof on any site with a given seismicity [1, 11]. In the proposed version of the method, it is possible to take into account some seismological features of the building site. First of all, it is situational seismicity determined by using general seismic zoning maps and Guidelines in law [13]. Information about possible sources of earthquakes is often available and one can take into account the possible magnitude and hypocentral distance for the structure under consideration. These data had been available, for example, in [14].

Methods

Description of the input model.

In this investigation the authors set the predominant input frequencies equal to the eigen structure ones. It makes the input dangerous for the structure under consideration. Other input parameters are indefinite ones and can be set by the way of providing the correspondence of the input model to actual earthquakes. The design input is represented as the sum of the speed pulse and the polyharmonic process, as it was used in [15].

In this case the input velocigram is written in the form

$$\dot{y}_0 = V(t-\phi)\cdot\eta(t-\phi)+\sum_{i=1}^{3} a_i e^{-\alpha_i t}\left(1-e^{-\beta_i t}\right)\sin\omega_i t \qquad (1)$$

,

where V (t) is the velocity impulse separated out the seismic input,

ϕ is phase shift from the beginning of the earthquake to the moment when the speed pulse arrives to the structure;

H (z) is the Heaviside function;

a_i, α_i, β_i and ω_i are the parameters of the polyharmonic component of the process.

The velocity pulse is described by the following dependences of the acceleration *acc*, the velocity v, and the displacement u on time

$$acc(t)=\begin{cases} u_{\max}/t_0^2 \\ -u_{\max}/t_0^2 \\ 0 \end{cases} \qquad v(t)=\begin{cases} u_{\max}\,t/t_0^2 \\ \dfrac{u_{\max}}{t_0}\left(2-\dfrac{t}{t_0}\right) \\ 0 \end{cases} \qquad u(t)=\begin{cases} \dfrac{u_{\max}}{2}\left(\dfrac{t}{t_0}\right)^2 \\ \dfrac{u_{\max}}{2}\left[-\left(\dfrac{t}{t_0}\right)^2+4\cdot\left(\dfrac{t}{t_0}\right)-2\right] \\ u_{\max} \end{cases} . \qquad (2)$$

Graphical interpretation of expressions (2) is presented in Figure 1.

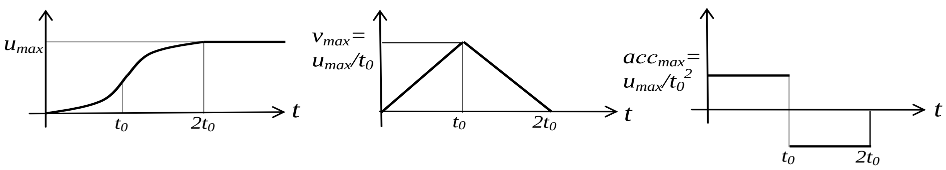

Figure 1. Time charts of displacements, velocities and accelerations for the speed pulse

The presence of a velocity pulse in a seismic input was fined by Italian seismologists [16]. They obtained the relationship between the pulse parameters and the magnitude of the earthquake Mw and the hypocentral distance R.

In particular, the half-pulse duration t_0 is uniquely determined by the magnitude of the action

$$t_0 = 10^{\left(-3.471+0.5\cdot M_w\right)}; \qquad (3)$$

And the residual displacement of the action depends on the magnitude and hypocentral distance

$$u_{\max} = 10^{\left(-6.3+M_w-\ell og(R)\right)} \qquad (4)$$

In this case, the velocity pulse is represented in the accelerogram as a step with an amplitude

$$acc_{\max} = \frac{u_{\max}}{t_0^2} = \frac{4.35}{R} \quad (5)$$

In formulas (3–5), obtained in [16], the value of R is substituted in "km", and the displacements, velocities and accelerations are obtained respectively in "m", "m/s" and "m/s^2". It is shown in [17] that the velocity pulse, theoretically obtained in [16], is presented in real accelerograms. This allows us to consider the input model in the form (1), as sufficiently universal. It has 12 indeterminate parameters: three parameters determining the momentum of the speed (Mw, R and ϕ) and 9 parameters (a_i, α_i, β_i, $i=3$) defining the polyharmonic process. Frequencies ω_i are set hazardous for the structure. Undefined parameters are set so that the accelerogram model has properties of real accelerograms. This statement of the problem is available in the works of A.A. Dolgaya [18, 19]. In the works mentioned, the input model was brought to conformity with real peak ground accelerations and Arias intensity. These proposals were included in the Recommendation [11]. Below they are developed taking into account new data about past earthquakes.

The main characteristics of seismic input

Three types of real accelerograms characteristics are distinguished in the literature [1, 20]: kinematic, spectral and energy.

The kinematic characteristics include peak accelerations (PGA), peak velocities (PGV), maximum displacements, residual displacements y_{rez}, duration of the action τ_{eq}, and also the process harmonic index κ. The quantity κ is determined by the formula

$$\kappa = \frac{\ddot{y}_0^{(\max)} \cdot y_0^{(\max)}}{\left(\dot{y}_0^{(\max)}\right)^2} \quad (6)$$

According to the American NPP calculation standards [21], the value of κ is assumed to be 5. In studies [22], it is noted that this parameter decreases with increasing the prevailing earthquake input period and it varies from 3 to 7. For harmonic action $\kappa = 1$. The larger κ, the more the process differs from the harmonic one. Large values of κ for the model do not allow one to concentrate the input energy at one frequency.

Kinematic characteristics of the impact are very important for elastic calculations under the loads caused by design earthquake (DE).

Input spectral characteristics, i.e. input spectral composition, are not considered in the framework of the proposed approach. Excitation frequencies ωi are set to be hazardous for the structure. This approach allows one of the frequencies ωi to be set equal to the prevailing frequency of the excitation predicted by seismologists, but in this case calculations will not be conservative.

Energy characteristics of the excitation determine the structure behavior under the impact of the MDE. Among these characteristics are

1) Arias intensity

$$I_A = \frac{\pi}{2g}\int_0^T \ddot{y}_0^2 dt \quad (7)$$

2) Absolute cumulative velocity CAV

$$CAV = \int_0^T \left|\ddot{y}_0\right| dt \quad (8)$$

3) Seismic energy density SED

$$SED = \int_0^T \dot{y}_0^2 dt \quad (9)$$

In the given investigation five parameters of the seismic input PGA, κ, I_A, CAV and SED are taken into account.

Results and Discussion

To estimate the values of input parameters corresponding to real accelerograms, more than 100 accelerogram records included in the combined database of Russian and Chinese accelerograms have been analyzed. During the research the dependence of the input parameters on the prevailing input frequency was analyzed.

The dependence $\ddot{y}_0(t)$ had been considered earlier in papers [18, 20], and the decrease of the PGA value was found with the growth of the predominant period T_{eq} increasing.

The dependence $\kappa(T_{eq})$ is shown in Figure 2, where the points indicate the data of the records of past earthquakes available for the authors. In this case for high-frequency inputs with the predominant period T_{eq}≈0.05…0.015 s, the value of κ can be about 10–15, and for long-period inputs the value of κ decreases down to 2–3. For some impacts, e.g., for the Bucharest earthquake κ≈1.5. It should be noted that a decrease in κ makes the input dangerous. Therefore, in order to generate hazardous input on the structure, it seems justified to set the value of κ according to the proposed schedule $\kappa(T_{eq})$ or somewhat lower, taking as a design value the follows

$$\kappa_{calc} = \kappa - \alpha \cdot \sigma, \tag{10}$$

where σ is the standard deviation of κ from the mean value,

α is the reliability index of the assumed design value.

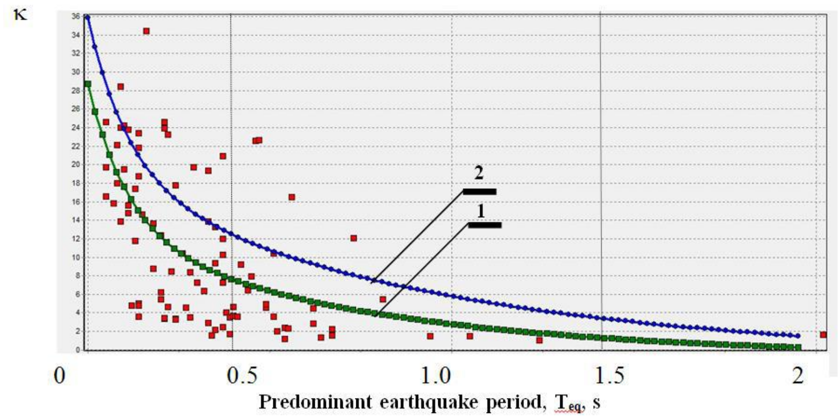

Figure2. Dependence $\kappa(T_{eq})$ obtained on the basis of the available records of real accelerograms. 1 – average value; 2 – $\kappa(T_{eq})$ - σ

Energy indicators are sufficiently stable characteristics of seismic input. Figure 3 shows the dependence I_A(Teq). The records of past earthquakes available to authors are denoted in the picture by points, and the mathematical expectation of the value and the sum of the mathematical expectation and standard deviation are denoted by lines. According to our data, the I_A value decreases slightly from 5 to 4 m/s with the growth of T_{eq}. In this case, the quantity $I_A + \sigma_I \approx 8$. It can be noted, that according to the data of [23], there is also a slight decrease in the value of I_A with an increase in the prevailing period. In the range of T_{eq} from 0.1 to 0.5 s $I_A \approx 2.53$ m/s, and for Teq> 0.5 s $I_A \approx 2$. If we take into account that the intensity of earthquakes considered in [23] is within the range of 8–9 degrees on the MSK scale, and the earthquakes considered in this paper have intensity of 9 or more degree, then the results obtained are quite compatible. Analogous dependencies can be obtained for other energy characteristics, in particular, for the CAV value. Between the quantities under consideration there is clear-cut correlation dependence, shown in Figure 4. This makes it possible to use any of the parameters under consideration when generating the design input. In Recommendations [11], the intensity according to Arias is given as the energy characteristic, and in the paper of American experts [23] the preference is given to the value of CAV.

$$Error = \sum_{i=1}^{5} p_i \cdot Er_i$$

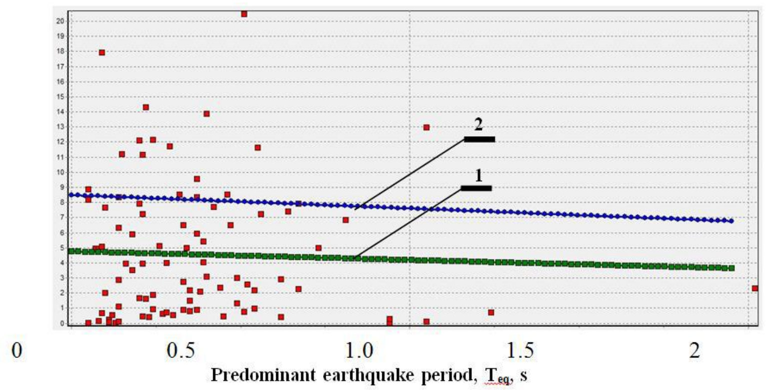

Figure 3. Dependence I_A(Teq). 1 – average value; 2 – $I_A(T_{eq})$ +σ_I

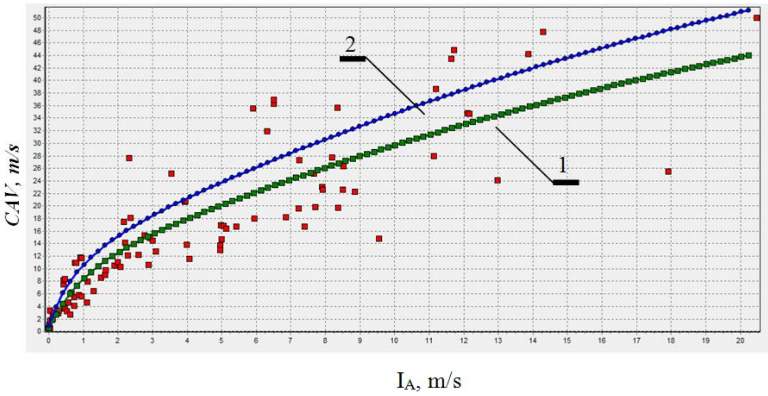

Figure 4. Dependence of CAV (I_A). 1 – average value of CAV (IA); 2 – CAV(I_A)+σ_{CAV}

The data of real accelerograms are denoted by dots

In paper [24], it is proposed to use the SED as the basic energy parameter. This value, according to the data available to the authors, does not correlate with the two energy characteristics considered before. It is sufficiently stable and in the first approximation it can be assumed to be equal to 1.5.

The obtained empirical dependencies are used to specify 12 indeterminate parameters of formula (1). To do this, some weight is assigned to each of the five parameters (PGA, κ, I_A, CAV, SED), and an error between the given empirical values and the analogous values obtained from formulas (6–9) is calculated at each grid point of the variable parameters.

By assigning different weight coefficients we get a set of dangerous inputs, with a close spectral composition. When choosing the coefficients, it seems possible to start with the following considerations.

Calculations for the impact of the DE are force ones, and the result of calculation, strains and stresses in the elements of the construction are proportional to the value of PGA. Therefore, when generating a DE, one should assign a large weighting factor for PGA.

Calculations for the impact of the MDE are energy ones. The strength condition of the elements is not met, but it is necessary to exclude progressive collapse and low cycle fatigue of the main load-bearing elements. To do this, it is necessary to get a limit to work of plastic deformation forces [25–28]. In this case energy characteristics are most important for the model input.

It is also important to limit the concentration of all seismic energy at one frequency both for the DE and for the MDE. Therefore, the weighting factor of κ must be taken into account for both inputs.

As an example, Figure 5 shows two generated accelerograms for calculating a building with suspended members [29], a seismically isolated structure with periods of natural oscillations T1 = 4.243 s, T2 = 3.145 s, T3 = 0.054 s. Table 1 shows the characteristics of these accelerograms. When generating the first accelerogram, three weight coefficients are assumed to be nonzero: p_{PGA} = 0.6, p_κ = 0.3, p_{IA} = 0.1, and for the second accelerogram p_{PGA} = 0.1; p_κ = 0.3; p_{IA} = 0.6. The generated accelerograms are quite similar, although they have some differences, for example, in residual displacements. This confirms the validity of the hypothesis expressed in [1] which supposes that taking into account the dependence of the peak acceleration on the prevailing acceleration period ensures that the parameters of the model and real excitations coincide.

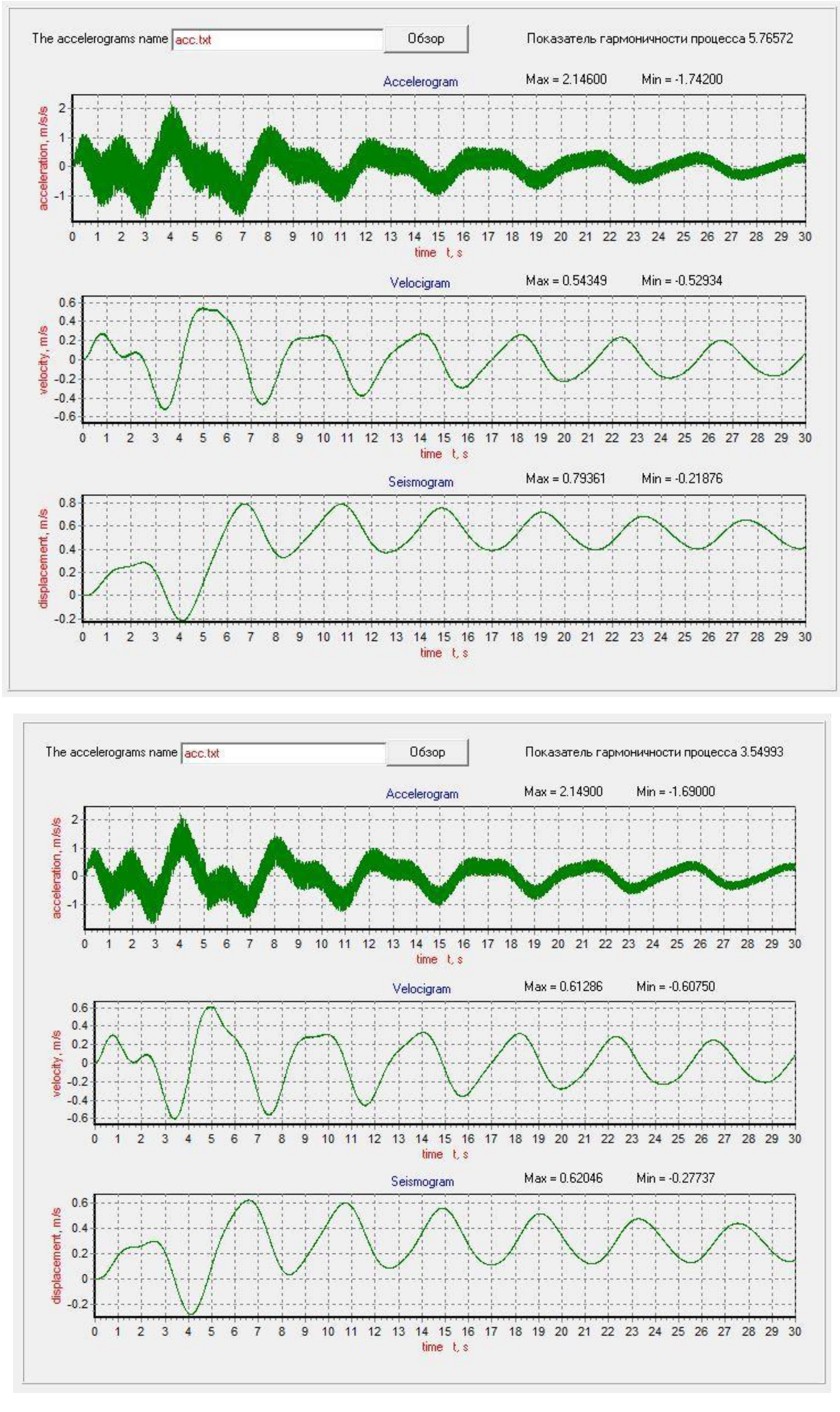

Fig.5. Oscillograms of artificial input for pPGA = 0.1 (on top) and pPGA = 0.6 (on bottom)

Table 1. Artificial input characteristics

Input charcteristics	**The type of input**	
	p_{PGA} = 0.6	p_{PGA} = 0.1
PGA	2.146	2.149
κ	3.55	5.76
I_A, m/s	1.368	1.343
CAV, m/s	12.02	12.23
SED, m^2/s	1.55	2.0

Conclusions

As the result of the investigations a new input model with a spectral composition that is dangerous for the structure under consideration, and with kinematic and energy characteristics corresponding to real seismic excitations has been generated. This new input model can be considered as a certain compromise between the model for the building site and the model for the structure.

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References

1. Uzdin A.M. Zadaniye seysmicheskogo vozdeystviya. Vzglyad inzhenera-stroitelya [The task of seismic action. The view of a civil engineer]. *Earthquake engineering. Constructions safety.* 2005. No. 1. Pp. 27–31. (rus)
2. Uzdin A.M. Chto skryvayetsya za lineyno-spektralnoy teoriyey seysmostoykosti [What lies behind the linear-spectral theory of seismic resistance]. *Earthquake engineering. Constructions safety.* 2009. No. 2. Pp. 18–23. (rus)
3. Aptikayev F.F. Prognoz parametrov seysmicheskikh kolebaniy, postroyeniye lokalnogo spektra i sinteticheskoy akselerogrammy [Forecast of seismic oscillation parameters, construction of local spectrum and synthetic accelerogram]. *Proceedings "Seysmostoykoye stroitelstvo v epokhu mogushchestva i schastya".* Ashkhabad, Ylym, 2013. Pp. 285–304. (rus)
4. Vakhrina G.N., Smirnov V.I. Razvitiye modeley raschetnykh akselerogramm seysmicheskikh vozdeystviy [Development of models of calculated accelerograms of seismic effects]. *Earthquake engineering. Constructions safety.* 2013. No. 1. Pp. 29–39. (rus)
5. Bani-Hani K.A., Malkawi A.I. A multi-step approach to generate response-spectrum-compatible artificial earthquake accelerograms. *Soil Dynamics and Earthquake Engineering.* 2017. Vol. 97. Pp. 117–132.
6. Vrochidou E., Alvanitopoulos P., Andreadis I., Elenas A., Mallousi K. HHT-Based artificial seismic accelerograms generation. *IFIP International Conference on Artificial Intelligence Applications and Innovations. AIAI 2014: Artificial Intelligence Applications and Innovations.* 2014. Pp. 476–486.
7. Li Zh., Kotronis P., Wu H. Simplified approaches for Arias Intensity correction of synthetic accelerograms. *Bulletin of Earthquake Engineering.* March 2017. Pp. 1–21.
8. Zentner I., Allain F., Humbert N., Gaudron M. Generation of spectrum compatible ground motion and its use in regulatory and performance-based seismic analysis. *Proceedings of the 9th International Conference on Structural Dynamics. EURODYN 2014.* Porto, Portugal, 2014. Pp. 381–384.
9. Kuznetsova I.O., Uzdin A.M., Zhgutova T.V., Shulman S.A. Seismic protection of railway bridges in Sochi. Proceedings of workshop "Bridges seismic isolation and large-scale modeling". Saint-Petersburg, 2010. Pp. 39–41.
10. Guzeyev R.V. Algoritm generatsii sinteticheskikh akselerogramm iz usloviya maksimalnogo sovpadeniya spektra otklika i normativnoy krivoy dinamichnosti [Algorithm for generating synthetic accelerograms from the condition of maximum coincidence of the response spectrum and the normative dynamic curve]. *Earthquake engineering. Constructions safety.* 2010. (rus)
11. *Rekomendatsii po zadaniyu seysmicheskikh vozdeystviy dlya rascheta zdaniy raznoy stepeni otvetstvennosti* [Recommendations for assigning seismic impacts for the

Литература

1. Уздин А.М. Задание сейсмического воздействия. Взгляд инженера-строителя // Сейсмостойкое строительство. Безопасность сооружений. 2005. № 1. С. 27–31.
2. Уздин А.М. Что скрывается за линейно-спектральной теорией сейсмостойкости // Сейсмостойкое строительство. Безопасность сооружений. 2009. № 2. С. 18–23.
3. Аптикаев Ф.Ф. Прогноз параметров сейсмических колебаний, построение локального спектра и синтетической акселерограммы // Сб. Сейсмостойкое строительство в эпоху могущества и счастья. Ашхабад, Ылым, 2013. С. 285–304.
4. Вахрина Г.Н., Смирнов В.И. Развитие моделей расчетных акселерограмм сейсмических воздействий // Сейсмостойкое строительство. Безопасность сооружений. 2013. № 1. С. 29–39.
5. Bani-Hani K.A., Malkawi A.I. A multi-step approach to generate response-spectrum-compatible artificial earthquake accelerograms // Soil Dynamics and Earthquake Engineering. 2017. Vol. 97. Pp. 117–132.
6. Vrochidou E., Alvanitopoulos P., Andreadis I., Elenas A., Mallousi K. HHT-Based artificial seismic accelerograms generation // IFIP International Conference on Artificial Intelligence Applications and Innovations. AIAI 2014: Artificial Intelligence Applications and Innovations. 2014. Pp. 476–486.
7. Li Zh., Kotronis P., Wu H. Simplified approaches for Arias Intensity correction of synthetic accelerograms // Bulletin of Earthquake Engineering. 2017. March. Pp. 1–21.
8. Zentner I., Allain F., Humbert N., Gaudron M. Generation of spectrum compatible ground motion and its use in regulatory and performance-based seismic analysis // Proceedings of the 9th International Conference on Structural Dynamics. EURODYN 2014. Porto, Portugal, 2014. Pp. 381–384.
9. Kuznetsova I.O., Uzdin A.M., Zhgutova T.V., Shulman S.A. Seismic protection of railway bridges in Sochi // Proceedings of workshop "Bridges seismic isolation and large-scale modeling". Saint-Petersburg, 2010. Pp. 39–41.
10. Гузеев Р.В. Алгоритм генерации синтетических акселерограмм из условия максимального совпадения спектра отклика и нормативной кривой динамичности // Сейсмостойкое строительство. Безопасность сооружений. 2010. С. 17–19.
11. Рекомендации по заданию сейсмических воздействий для расчета зданий разной степени ответственности. С.-Петербург–Петропавловск-Камчатский. КамЦентр. 1996. 12 с.
12. Никонова Н.В. Особенности задания воздействия и расчета нелинейных систем сейсмоизоляции // Известия Петербургского университета путей сообщения. 2016. № 3. С. 430–438.

calculation of buildings of different degrees of responsibility] S.-Peterburg–Petropavlovsk-Kamchatskiy. KamTsentr. 1996. 12 p. (rus)

12. Nikonova N.V. Osobennosti zadaniya vozdeystviya i rascheta nelineynykh sistem seysmoizolyatsii [Features of the task of impact and calculation of nonlinear seismic isolation systems]. *Izvestiya Peterburgskogo universiteta putey soobshcheniya.* 2016. No. 3. Pp. 430–438 (rus)
13. SP 14.13330.2011 Stroitelstvo v seysmicheskikh rayonakh [Russian Set of Rules SP 14.13330.2011 Construction in seismic regions]. (rus)
14. *Seysmicheskaya sotryasayemost territorii SSSR* [Seismic shaking of the territory of the USSR]. Ed. Yu.V.Riznichenko. Moscow: Nauka, 1979. 192 p. (rus)
15. Dmitrovskaya L.N., Uzdin A.M. Ob odnoy forme predstavleniya seysmicheskogo vozdeystviya dlya otsenki korrelyatsii kolebaniy tochek dnevnoy poverkhnosti pri raschete mnogoopornykh konstruktsiy [On one form of representation of seismic action for estimating the correlation of fluctuations of the points of the day surface in the calculation of multi-support structures]. *Earthquake engineering. Constructions safety.* 2006. No. 2. Pp. 22–25. (rus)
16. Faccioli E., Paolicci R., Rey Ju. Displacement spectra for long periods. *Earthquake Spectra.* Vol. 20. No. 2. Pp. 347–376.
17. Uzdin A.M., Dmitrovskaya L.N., Sakharov O.A. Setting the level of design acceleration on the basis of the energy theory of earthquake engineering. *Fourteen European Conference on Earthquake Engineering.* Macedonia, Ochrid, 2010. CD-paper No. 189. Abstract Book. R. 98.
18. Dolgaya A.A. Modelirovaniye seysmicheskogo vozdeystviya korotkim vremennym protsessom [Simulation of seismic action by a short time process]. *Ekspress-informatsiya VNIINTPI. Ser. “Seysmostoykoye stroitelstvo”.* 1994. No. 5-6. Pp. 56–63. (rus)
19. Dolgaya A.A., Indeykin A.V. Statisticheskiy analiz intensivnosti po Ariasu i skorosti dlya realnykh zemletryaseniy [Statistical analysis of Arias intensity and velocity for real earthquakes]. *Seysmostoykoye stroitelstvo.* 2002. No. 2. Pp. 32–33. (rus)
20. Bogdanova A.M., Nesterova O.P., Nikonova N.V., Tkachenko A.S., Uzdin A.M., Ravkhmanova M., Azayev T.M., Zaynulabidova Kh.R. Chislovyye kharakteristiki seysmicheskikh vozdeystviy [Numerical characteristics of seismic actions]. *Nauka i mir.* 2017. Vol. 1. No. 3(43). Pp. 49–55. (rus)
21. Birbrayer A.N. *Raschet konstruktsiy na seysmostoykost* [Calculation of structures for seismic resistance]. Saint-Petersburg: Nauka, 1998. 254 p. (rus)
22. Bogdanova G.A., Dolgaya A.A., Ivanova J.V, Sakharov O.A., Uzdin A.M. The model of seismic impact as a short temporary process for calculating of the seismoisolated systems. *12th World Conference on Earthquake Engineering.* New Zealand, 2000. Paper No. 1358.
23. Campbell K.W., Bozorgnia Y. Cumulative absolute velocity (cav) and seismic intencity based on the peer-nga database. *Earthquake Spectra.* 2012. Vol. 28. No. 2. Pp. 457–485.
24. Rutman Yu.L., Shiwua J. Otsenki seysmicheskoy energii, postupivshey v uprugo-plasticheskuyu sistemu s odnoy stepenyu svobody [Estimates of seismic energy received in an elastic-plastic system with one degree of freedom]. *Vestnik grazhdanskikh inzhenerov SPbGASU*. 2015. No. 2(49). Pp. 64–74. (rus)
25. Goldenblat I.I., Nikolayenko N.A., Polyakov S.V., Ulyanov S.V. *Modeli seysmostoykosti sooruzheniy* [Models of seismic stability of structures]. Moscow: Stroyizdat,1979. 251 p. (rus)

13. СП 14.13330.2011 Строительство в сейсмических районах. Актуализированная редакция СНиП II-7-81*.
14. Сейсмическая сотрясаемость территории СССР. Под ред. Ю.В. Ризниченко. М.: Наука, 1979. 192 с.
15. Дмитровская Л.Н., Уздин А.М. Об одной форме представления сейсмического воздействия для оценки корреляции колебаний точек дневной поверхности при расчете многоопорных конструкций // Сейсмостойкое строительство. Безопасность сооружений. 2006. № 2. С. 22–25.
16. Faccioli E., Paolicci R., Rey Ju. Displacement spectra for long periods // Earthquake Spectra. Vol. 20. № 2. Pp. 347–376.
17. Uzdin A.M., Dmitrovskaya L.N., Sakharov O.A. Setting the level of design acceleration on the basis of the energy theory of earthquake engineering // Fourteen European Conference on Earthquake Engineering. Macedonia.Ochrid. 2010. CD-paper №189. Abstract Book. P. 98.
18. Долгая А.А. Моделирование сейсмического воздействия коротким временным процессом // Экспресс-информация ВНИИНТПИ. Сер. “Сейсмостойкое строительство”. 1994. № 5-6. С. 56–63.
19. Долгая А.А., Индейкин А.В. Статистический анализ интенсивности по Ариасу и скорости для реальных землетрясений // Сейсмостойкое строительство. № 2. 2002. С. 32–33.
20. Богданова А.М., Нестерова О.П., Никонова Н.В., Ткаченко А.С., Уздин А.М., Равхманова М., Азаев Т.М., Зайнулабидова Х.Р. Числовые характеристики сейсмических воздействий // Наука и мир. № 3(43). 2017. Т. 1. С. 49–55.
21. Бирбраер А.Н. Расчет конструкций на сейсмостойкость. СПб.: Наука. 1998. 254 с.
22. Bogdanova G.A., Dolgaya A.A., Ivanova J.V, Sakharov O.A., Uzdin A.M. The model of seismic impact as a short temporary process for calculating of the seismoisolated systems // 12th World Conference on Earthquake Engineering. New Zealand, 2000. Paper No. 1358.
23. Campbell K.W., Bozorgnia Y. Cumulative Absolute velocity (cav) and seismic intencity based on the PEER-NGA database // Earthquake Spectra. 2012. Vol. 28. № 2. Pp. 457–485.
24. Рутман Ю.Л., Шивуа А.Дж. Оценки сейсмической энергии, поступившей в упруго-пластическую систему с одной степенью свободы // Вестник гражданских инженеров СПбГАСУ. 2015. № 2(49). С. 64–74.
25. Гольденблат И.И., Николаенко Н.А., Поляков С.В., Ульянов С.В. Модели сейсмостойкости сооружений. М.: Стройиздат,1979.251 с.
26. Москвитин В.В. Циклические нагружения элементов конструкций. М.: Наука, 1981. 344 с.
27. Erberik M.A., Sucuoglu H. Energy-based low-cycle fatigue characteristics of degrading structures // Proc. of 12-th European Conference on Earthquake Engineering. Paper Reference 118.
28. Shiwua J., Rutman Y. Assessment of seismic input energy by means of new definition and the application to earthquake resistant design // Architecture and Engineering. 2016. Vol. 1. № 4. Pp. 26–35.
29. Белаш Т.А., Рыбаков П.Л. Здания с подвесными конструкциями в сейсмических районах // Инженерно-строительный журнал. 2016. № 5(65). С. 17–26.

Иванова Т.В., Гуань Ю., Нестерова О.П., Прокопович С.В., Смирнова Л.Н., Уздин А.М., Ивашинцов Д.А. Моделирование расчетного сейсмического воздействия в условиях ограниченной сейсмологической информации // Инженерно-строительный журнал. 2017. № 7(75). С. 129–138.

26. Moskvitin V.V. *Tsiklicheskiye nagruzheniya elementov konstruktsiy* [Cyclic loading of structural elements]. Moscow: Nauka, 1981. 344 p. (rus)
27. Erberik M.A., Sucuoglu H. Energy-based low-cycle fatigue characteristics of degrading structures. *Proc. of 12-th European Conference on Earthquake Engineering.* Paper Reference 118.
28. Shiwua J., Rutman Y. Assessment of seismic input energy by means of new definition and the application to earthquake resistant design. *Architecture and Engineering.* 2016. Vol. 1. No. 4. Pp. 26–35.
29. Belash T.A., Rybakov P.L. Buildings with suspended structures in seismic areas. *Magazine of Civil Engineering*. 2016. No.5. Pp. 17–26.

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Determination of soil deformation moduli after National Building Codes of Russia and Germany

Расчетные модули деформации грунта согласно национальным стандартам России и Германии

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Key words: compression tests; compression modulus; oedometer deformation modulus; design deformation modulus; transition coefficients; comparison of reference standards

Ключевые слова: компрессионные испытания; компрессионный модуль деформации; одометрический модуль деформации; расчетный модуль деформации; переходные коэффициенты; сравнение нормативных стандартов

Abstract. The paper analyzes the differences in carrying out compression tests in accordance with the regulatory documents of Russia and Germany and subsequent processing of the obtained data. The paper gives the reasons for significant differences in the values of compression and design deformation moduli obtained from the compression test data in accordance with Russian and German National Building Codes. Approximate differences of the above-mentioned moduli have been revealed for various types of soil. The transition coefficients have been calculated for the compression and design moduli obtained from the compression test data in accordance with Russian and German National Building Codes. The paper describes the applicability of these transition coefficients. The paper presents the research results carried out within the framework of international cooperation of two universities: Industrial University of Tyumen (Russia) and Beuth University of Applied Sciences, Berlin (Germany). The results of the research prove the correctness of the calculated transition coefficients.

Аннотация. Произведен анализ отличий проведения компрессионных испытаний в соответствии с нормативными документами России и Германии и последующей обработки данных. Описаны причины значительных различий получаемых значений компрессионных и расчетных модулей деформации, вычисляемых по данным компрессионных испытаний в соответствии с российскими и немецкими национальными стандартами. Выявлены приблизительные отличия вышеупомянутых модулей для различных видов грунта. На основе этого вычислены переходные коэффициенты для компрессионных и расчетных модулей деформации, полученных по данным компрессионных испытаний согласно российским и немецким национальным стандартам. Описаны условия применения данных коэффициентов. Представлены результаты исследований, проведенных в рамках международного сотрудничества двух университетов: Тюменского индустриального университета (Россия) и университета прикладных наук им. Бойта, Берлин (Германия). Результаты исследований подтверждают корректность применения приведенных переходных коэффициентов.

Introduction

International cooperation in various fields between Russia and Germany is a question of present interest; there is a growing interest in solving common scientific problems and executing joint industrial projects which can be implemented both in Russia and Germany. The lack of uniformity of terms, concepts, their interpretation and compliance with national technical (industrial) standards is a certain obstacle to effective joint scientific work and productive cooperation.

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Unification of the initial data for calculations, consideration of the methods for determining various characteristics and parameters and verification of the results obtained are topical issues in the sphere of construction.

Great attention is being paid to unification of the regulatory documents: National Building Codes are being updated with regard to the requirements of European (EN) and International (ISO) Standards [1–5]; various aspects of misbalance between the foreign and Russian regulatory framework are being studied [4–7], as well as the relevance of the changes introduced to the updated standards [5–8]. But in addition to the adopted International and European regulatory documents, Germany has its own Building Regulations (DIN). Available studies comparing the National Building Codes of the Russian Federation and Germany are not sufficient for possible unification of geotechnical norms.

The study focuses on the principal deformation characteristic of soil – soil deformation modulus, which is the basis for foundation analysis after the second group of ultimate limit states.

E.V. Lega [9] made a general comparative analysis of the obtained values of the main physical and mechanical characteristics of silts in accordance with the Russian and German norms including the deformation modulus. However, there is no clear and detailed description and comparison of methods for determining the deformation moduli obtained. In addition, it should be noted that the studies were carried out between 1991 and 1994 in accordance with regulatory documents irrelevant at the moment. At that time, Germany lacked a document to determine the deformation characteristics of soil in the universal regulatory. Therefore, the verification and the unification of the geotechnical norms of Russia and Germany to determine the deformation modulus cannot be considered as studied and thus, further studies are needed to take the diversity of soil types into account.

At present, a number of different methods can be used to determine the deformation modulus [5–8, 10–19]; here, the resulting values of the deformation modulus for one and the same soil can vary significantly depending on the method of its determination (several times for clay soils), as it is shown in the works of the authors [5–8, 13, 14, 19–24].

When calculating the settlements of buildings and structures, especially in complex soil and groundwater conditions, the question arises of whether the design deformation modulus is chosen correctly. Table 1 presents the methods for determining the deformation modulus and the reference standards of Russia and Germany regulating them.

Table 1. Reference Standards of RF and Germany regulating determination of the deformation modulus

Methods to determine the deformation modulus	**GOST**	**DIN**
Oedometer compressibility testing	GOST 12248-2010	DIN 18135
Triaxial deformability testing	GOST 12248-2010	absent
Flat plate load tests	GOST 20276-2012	DIN 18134
Screw plate load tests	GOST 20276-2012	absent
Pressuremeter tests	GOST 20276-2012	DIN EN ISO 22476-4
Dilatometer tests	absent	DIN EN ISO 22476-5
Cone penetration testing	GOST 19912-2012	DIN EN ISO 22476-1
Dynamic testing	GOST 19912-2012	DIN EN ISO 22476-2
Borehole dynamic probing	absent	DIN 4094-2

The existing regulatory documents offer some recommendations for choosing and determining the design deformation modulus, since great varieties of the deformation moduli are obtained depending on the method used.

According to Regulation – SP 22.13330.2016 [25], a plate load test is the priority method for determining the soil deformation modulus in Russia, pressuremeter tests (in case of isotropy of the properties of the tested soils). In order to interpret the data of other methods, correlation dependencies, based on the above mentioned experiments, are used. The applicability of the complex approach to determine the deformation modulus is also confirmed by the research of Russian scientists, e.g. G.G Boldyrev [7]. For structures which fall into the limits of the Geotechnical Category 1, and in some cases 2 (if statistically based data according the regional normative regulations is available), the design deformation modulus can be calculated from the compression test data taking the multiplying transition coefficient from the oedometer modulus to the modulus from plate loading test m_{oed} [25, Table. 5.1] into account; its values are correct for the compression modulus calculated in the pressure range of

0.1–0.2 MPa and they can lie in the range from 1.2 to 3 for silty-clayed soils. According the previous version of the same regulation SP 22.13330.2011 [26] the caclulation of the design deformation modulus is performed in the same pressure range of 0.1–0.2 MPa using the transition coefficient from the compression modulus to the modulus from plate loading test m_k in the range of 2 to 6 [26, Table. 5.1]. At this point it should be noted that the deformation modulus determined using both the transition coefficient m_k and the coefficient m_{oed} has th esame value. For sandy soils, the multiplying coefficient is absent. These transition coefficients are widely used in Russia, but compression tests are compulsory and are often conducted together with cone penetration tests and less often with dynamic tests. The priority methods are rarely used due to their high labour intensity and cost, only by agreement with the client.

The design deformation modulus is also an urgent problem in Germany. There is a lack of any clear recommendations concerning the design deformation modulus in the regulatory document DIN 4019: 2015-05 [27] which regulates the settlement calculation; it can be chosen by a design engineer after comparison of the results of laboratory and in-situ tests and monitoring the settlements of structures, but some recommendations still exist. Thus, in accordance with paragraph 8.4 of the DIN standard [27], in the absence of specific soils and settlement monitoring data, the design deformation modulus E^* may be assumed to be approximately equal to the oedometer modulus E_s determined in accordance with DIN 18135: 2012-04 [28], taking into account the state of the soil formation at a corresponding pressure. It is necessary to consider that in laboratory conditions a sample of cohesionless soil of sufficiently good quality intended for accurate determination of the soil deformation characteristics can be obtained only in exceptional cases; when testing cohesive soil in laboratory conditions, reliability of the deformation modulus depends on the quality of the taken sample, its treatment and the experiments, taking the soil stressed state in the massif into account.

In order to conduct the research, the basic method for determining the deformation modulus in Russia and Germany was considered – compression tests performed in accordance with GOST 12248-2010 [29] and DIN [28].

Research objectives: to identify the principal differences in conducting compression tests and factors affecting the results; possible differences between the compression and design deformation moduli obtained in accordance with Russian and German National Building Codes; search for transition coefficients.

Methods

The principal differences in carrying out compression tests in accordance with GOST [29] and DIN [28] are expressed in the duration of the increment load. To consider unsaturated soil, according to GOST [29], the increment load is maintained until the conditional stabilization of the soil sample deformation occurs. Incremental portion is taken as a criterion of conditional stabilization of deformation, which does not exceed 0.05% for the time indicated in [29, Table. 5.3] and depends on the type of soil. With water-saturated clay, organic or organic-mineral soil, the end of compaction at this increment load is determined as the end of 100% filtration consolidation. In accordance with DIN [28], each portion of increment load must be maintained at the same time interval, at least until the completion of 100 % primary consolidation. In here, these differences do not affect the results, since in both cases the increment load is maintained up to a certain stabilized state of the soil. The done research also proves this [30].

Processing of the experimental data greatly affects the value of the deformation modulus calculated from the compression test data. According to DIN [28], an oedometer deformation modulus is obtained as a result of compression tests. It is evaluated by the formula:

$$E_s = \frac{\Delta\sigma'}{\dfrac{\Delta s}{h_i}} = \frac{\Delta\sigma'}{\Delta s'}(1 - s'), \tag{1}$$

where $\Delta\sigma'$ – interval of axial stress variation in which the deformation modulus is determined;

Δs – change of *compressive deformation* of the sample in dependence of the variation in axial stress $\Delta\sigma'$;

h_i – height of the sample which agrees with the mean axial stress of the considered interval:

$$h_i = h_0 - s_i = h_0 - h_0 s' = h_0(1 - s'), \tag{2}$$

h_0– initial height of the sample;

s – *compressive deformation* of the sample which refers to the mean axial stress of the considered interval;

s' – *relative compressive deformation* of the sample which refers to the mean axial stress of the considered interval:

$$s' = \frac{s}{h_0}, \tag{3}$$

$\Delta s'$ – change of *relative compressive deformation* of the sample in dependence of the variation in axial stress $\Delta\sigma'$:

$$\Delta s' = \frac{\Delta s}{h_0}, \tag{4}$$

In accordance with the Russian State Standard GOST [29], the oedometer and compression deformation moduli can be obtained from the data of compression tests. The oedometer deformation modulus is evaluated by the formula (5), compression one – by the formula (6):

$$E_{oed} = \frac{\Delta p}{\Delta \varepsilon}, \tag{5}$$

$$E_k = E_{oed}\beta, \tag{6}$$

where Δp – pressure interval at which the deformation modulus is determined;

$\Delta\varepsilon$ – variations in the relative deformation which agree with Δp:

$$\Delta\varepsilon = \frac{\Delta h}{h}, \tag{7}$$

h – initial height of the sample;

β – coefficient that takes the absence of lateral expansion of soil in the compression equipment into account:

$$\beta = 1 - \frac{2\nu^2}{1-\nu}, \tag{8}$$

ν – Poisson's ratio.

In real values of Poisson's ratio within 0.1–0.45 for soils (the smallest value corresponds to coarse soil, the largest one – to clay soil), β takes the values within 0.26–0.98 (the smallest value corresponds to clay soil, the largest one – to coarse soil). In the absence of experimental data in engineering calculations, the value of Poisson's ratio can be taken within 0.27–0.45 depending on the type of soil in accordance with the Russian regulatory document [25, Table. 5.10]; in here, β is of values of 0.26–0.8.

In order to determine the compression modulus, β can be chosen according to [29, paragraph 5.4.6.4] equal to 0.8 for sands, 0.7 – for clayey sands, 0.6 – for silts and 0.4 – for clays.

When comparing formulas (1) and (5), it is obvious that the oedometer modulus E_s of DIN [28] is approximately equal to the oedometer modulus of GOST [29] E_{oed}. This is because the only difference is the factor *(1-s')* in the formula (1), which is close to U, since the relative compressive deformation of the sample – s' is very small. Thus, the coefficient β can be used as a transition coefficient from the oedometer deformation modulus obtained by DIN [28] E_s to the compression modulus obtained by GOST [29] E_k. In here, it is necessary to take into account that the types of soil do not always match together in Russian and German classifications (for example, very soft silt in Russian classification can be weakly low plasticity clay in German classification; this is proved by the studies [31]). However, in most cases β values as transition values are acceptable. Thus, it is possible to calculate the transition coefficients from the compression modulus obtained by GOST [29] E_k to the oedometer deformation modulus obtained by DIN [28] E_s, which can be applied regardless of matching of the soil types in German and Russian classifications: 2.5 for clays; 1.67 for silts, 1.43 for clayey sands and 1.25 for sands.

Based on the assumption that the oedometer deformation moduli obtained by GOST [29] and DIN [28] are equal and taking into account the multiplying coefficient m_{oed} ($m_{oed} = \beta \cdot m_k$), it is possible to determine the approximate transition coefficients from the design deformation modulus taken in accordance with SP [25,26], i.e. $E = m_{oed} \cdot E_{oed}$ or $E = m_k \cdot E_k$, to the design deformation modulus taken in accordance with DIN [27], i.e. $E^* \approx E_s$, (Table 2). The coefficient m_{oed} can be used as the transition coefficient from $E^* \approx E_s$ after DIN [27] to $E = m_{oed} \cdot E_{oed}$ after SP [25] or $E = m_k \cdot E_k$ after SP [26] (Table 3).

Table 2. Transition coefficients from $E = m_{oed} \cdot E_{oed}$ after Building Regulations (SP) [25] to $E^* \approx E_s$ after DIN [27]

Soil type	Voids ratio, e					
	0.45–0.55	0.65	0.75	0.85	0.95	1.05
Clayey sands	0.36	0.40	0.48	0.71	-	-
Silts	0.33	0.37	0.42	0.56	0.67	0.83
Clays	-	0.42	0.42	0.45	0.50	0.56

Table 3. Transition coefficients from $E^* \approx E_s$ after DIN [27] to $E = m_{oed} \cdot E_{oed}$ after Building Regulations (SP) [25]

Soil type	Voids ratio, e					
	0.45–0.55	0.65	0.75	0.85	0.95	1.05
Silts	3.0	2.7	2.4	1.8	1.5	1.2
Clays	-	2.4	2.4	2.2	2	1.8

The values given in Tables 2 and 3 are determined by interpolation for intermediate values of the void ratio *e*.

To confirm the theoretical prerequisites in the framework of international cooperation of two universities: the Beuth University of Applied Sciences, Berlin (Germany) and the Industrial University of Tyumen (Russia), a number of experiments were carried out on various types of soils. The investigated soils and their basic physical characteristics are presented in Table 4 in accordance with Russian Building Regulation / German Standards. The physical characteristics of the soils have been determined in accordance with the Russian document GOST 5180-2015 [32] and the regulatory framework of Germany: DIN EN ISO 17892-1: 2015-03 [33], DIN EN ISO 17892-2: 2015-03 [34], DIN 18122-1: 1997-07 [35]. Classification of the soil is presented in accordance with GOST 25100-2011 [36] and DIN 18196: 2011-05 [37], DIN EN ISO 14688-1: 2013-12 [38], DIN EN ISO 14688-2: 2013-12 [39].

Table 4. Physical characteristics of tested soils

Soil type	ρ, g/cm³	ρ_s, g/cm³	ρ_d, g/cm³	n, %	e, unit fraction	W, %	W_P, %	W_L, %	I_p, unit fraction	I_L, unit fraction
Dense sand of medium size / Even-graded sand of medium size coarse sandy, fine sandy, fine gravelley	1.89	2.67	1.83	0.31	0.46	3	-	-	-	-
Firm silt of undisturbed structure / intermediate plasticity clay firm of undisturbed structure	1.92	2.71	1.48	0.46	0.83	30	22.0	$\frac{34.4}{43.0}$	$\frac{0.12}{0.21}$	$\frac{0.65}{0.38}$
Stiff clay of undisturbed structure / high plasticity clay stiff of undisturbed structure	1.96	2.77	1.50	0.46	0.85	31.1	22.8	$\frac{51.0}{66.7}$	$\frac{0.28}{0.44}$	$\frac{0.29}{0.19}$
Very stiff clay of undisturbed structure / high plasticity clay stiff of undisturbed structure	2.10	2.97	1.63	0.45	0.82	28.9	25.1	$\frac{50.2}{66.4}$	$\frac{0.25}{0.41}$	$\frac{0.15}{0.09}$
Firm silt of disturbed structure / low plasticity clay soft of disturbed structure	2.0	2.71	1.64	0.40	0.65	22.0	15.0	$\frac{28.0}{33.7}$	$\frac{0.13}{0.19}$	$\frac{0.54}{0.37}$

Note to the Table: the values of W_L, I_p, I_L given above the line are obtained after Russian Building Regulations, under the line - after German Standards.

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The soil types given below are referred only to the Russian classification in accordance with GOST [36].

In order to obtain statistical data for each type of soil, several compression tests were performed in accordance with the GOST [29] and DIN [28] methods. Sand of medium size, firm silt undisturbed structure and stiff clay of undisturbed structure with medium consistency were tested in the geotechnical laboratory of the Beuth University (Germany) in a mechanical compression device with an internal diameter of the ring of 100 mm and a height of 30 mm, clay with undisturbed structure and very stiff consistency – in the laboratory of the Geotechnical Department of TIU (Russia) in an automated compression device ASIS with an internal diameter of the operating ring of 87 mm and a height of 25 mm. In the laboratory of the TIU samples of the same homogeneous soil (silt of disturbed structure with firm consistency) were also prepared and tested to avoid errors by comparing the results obtained with the GOST [29] and DIN [28] methods, caused by the inherent heterogeneity of a soil from natural deposition.

Results and Discussion

As a result of testing the twin samples, the compression curves almost completely coincided. According to the experiments with the other soils, the compression curves of the same soil type have permissible deviations. The obtained statistical average compression curves of the tested soils are shown in Fig. 1.

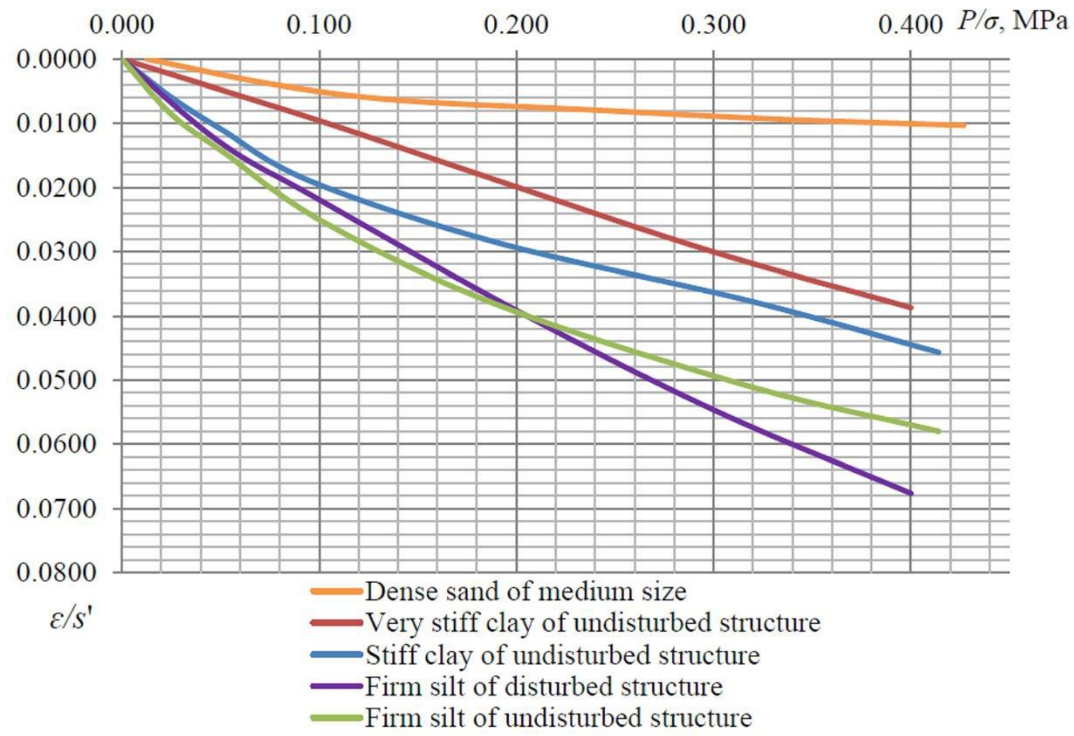

Figure 1. Plot of the relative deformation versus pressure / axial stress after compression tests according to GOST [29] / DIN [28] under primary loading

The tests resulted in determining the deformation moduli in the pressure range of 0.1–0.2 MPa (Table 5).

Table 5. Moduli of deformation of tested soils

Soil type in Russian/ German classification	Oedometer modulus E_{oed}, GOST [29], MPa	Compression modulus E_k, GOST [29], MPa	Oedometer modulus E_s, DIN [28], taken as design modulus E^*, DIN [27], MPa	Differences between E_{oed} and E_s/ E_k and E_s %	Multiplying coefficient m_k/m_{oed}, Building Regulations (SP) [26,25]	Design modulus of deformation $E = m_k \cdot E_k$, $E = m_{oed} \cdot E_{oed}$, Building Regulations (SP) [26,25]	Differences in design moduli of deformation by SP [25,26] and DIN [27], %
Medium sand	50.00	40.00	49.6	0.8 24	-	40.00	-24
Very stiff clay of undisturbed structure	9.67	3.87	9.53	1.5 146	5.65 2.26	21.87	+129
Stiff clay of undisturbed structure	9.88	3.95	9.74	1.4 147	5.5 2.2	21.73	+123
Firm silt of undisturbed structure	6.86	4.12	6.63	3.5 61	3.2 1.92	13.18	+99
Firm silt of disturbed structure	6.14	3.68	5.90	4.1 60	4.5 2.7	16.61	+182

As expected, the differences between the oedometer deformation moduli obtained by DIN [28] and GOST [29] are insignificant (1–4 %) and depend on the value of the deformation modulus itself. If the difference is smaller, the disagreement is greater, since relative settlement *s'* is also of greater importance for a weaker soil.

As expected, the least difference between the compression modulus obtained by GOST [29], oedometer modulus obtained by DIN [28] and design deformation modulus taken by the recommendations of SP [25, 26] and DIN [27] is observed for sand, the greatest – for clays.

Thus, the oedometer deformation modulus of sand determined by DIN [28] exceeds the compression modulus determined by GOST [29] by 24 %, silt – 1.6 times and clay – 2.5 times.

According to the results of sand testing, the design deformation modulus $E^* \approx E_s$ – DIN [27] exceeds the design deformation modulus $E = m_{oed} \cdot E_{oed}$ – [25] or $E = m_k \cdot E_k$ – SP [26] by 24%; for clay soil the design deformation modulus $E^* \approx E_s$ – DIN [27] is much lower than that of $E = m_{oed} \cdot E_{oed}$ – SP [25] or $E = m_k \cdot E_k$ – SP [26], namely: nearly 2.2-2.3 times – for clays, 2 times – for silts of the undisturbed structure and 2.8 times – for silts of the disturbed structure. Such a significant excess of the design deformation modulus in clays $E = m_{oed} \cdot E_{oed}$ – SP [25] or $E = m_k \cdot E_k$ – SP [26] is due to the multiplying coefficient m_k/m_{oed} which depends not only on the soil type, but also on its state.

The results obtained prove the applicability of the transition coefficients presented in Tables 2 and 3, but in transition from the design deformation modulus $E^* \approx E_s$, taken by DIN [27] to the design deformation modulus $E = m_{oed} \cdot E_{oed}$, taken by SP [25], or $E = m_k \cdot E_k$, taken by SP [26], the transition coefficients are relevant only in cases when the soil types match together by GOST [36] and DIN [37–39].

It is necessary to note that the values of the coefficient m_k or m_{oed},obtained by direct tests can significantly differ from those recommended by SP [25, 26], as seen from various studies including [19, 23]. There is a viewpoint that the values of m_k and also of m_{oed}, must be calculated taking into account the regional soil features, e.g. as in [40]. A.V. Pilyagin considers that it is generally incorrect to set the transition coefficients from the compression modulus to the plate one (m_k) due to the difference in the types of stress state of soils during compression and plate load tests [24]. Moreover, there are doubts about the applicability of the compression modulus obtained by GOST [29]. In world practice and in Germany, it is the oedometer deformation modulus that is obtained after the compression test data. In addition, in the current version of the SP [25] the increasing coefficients for the design deformation modulus are presented for the oedometer modulus which is indirectly applied in various geotechnical programs (for example, PLAXIS). The authors agree with G.G. Boldyrev that the oedometer deformation modulus is more accurate, since it is determined in direct measurements regardless of lateral expansion of soil and that it is easier to use it as a reference for finding various correlation dependences [7]. When using the oedometer deformation modulus instead of the compression modulus, it is easier to adapt Russian Building Regulations compared to the foreign regulations.

Conclusions

1. It has been stated that the principal differences in carrying out compression tests in accordance with GOST 12248 and DIN 18135 are expressed in the duration of the increment load, which do not affect the results of the experiments. In turn, processing of the experimental data significantly influences the value of the deformation modulus resulting from the compression tests. This is due to the presence of the coefficient β in the formula for the compression modulus E_k (GOST). The coefficient considers the absence of lateral deformations of the soil in the compression equipment, which underestimates the value of the compression modulus as compared to the oedometer deformation modulus E_s (DIN).

2. The oedometer modulus E_s (DIN) is approximately equal to the one E_{oed} (GOST). The difference lies in the presence of the factor $(1\text{-}s')$ in the formula for determining E_s, which is close to U, since the relative compressive deformation of the sample – s' is very small. Therefore, the deviations depend on soil deformability, i.e. weaker soil will have a higher value of s' in comparison to harder soil. The difference between the values of oedometer moduli obtained by DIN and GOST will be greater in weak soil testing. The test results showed small deviations – from 1 to 4%.

3. The compression tests have resulted in the values of the oedometer deformation modulus E_s (DIN), compression modulus $E_к$ (GOST) and design deformation modulus $E = m_k \cdot E_к = m_{oed} \cdot E_{oed}$ (SP 22.13330) and $E^* \approx E_s$ (DIN 4019) which significantly differ depending upon the soil type. The smallest difference is characteristic for sands, the largest one – for clays. The following dependencies have been revealed: $E^{(SP)} > E^{*(DIN)} \approx E_s \approx E_{oed} > E_k$ – silty-clayey soils, $E^{(SP)} = E_k < E^{*(DIN)} \approx E_s \approx E_{oed}$. – for sands.

4. Based on the assumption that the oedometer deformation moduli E_{eod} (GOST) and E_s (DIN) are equal and taking into account the available dependencies of oedometer E_{eod} and compression E_k moduli (coefficient β), compression E_k and design E deformation moduli (coefficient m_k), oedometer E_{oed} and the calculated E deformation moduli (coefficient m_{oed}), the approximate transition coefficients have been determined from the design deformation modulus taken in accordance with SP $E = m_k \cdot E_к = m_{oed} \cdot E_{oed}$, to the design deformation modulus taken in accordance with DIN $E^* \approx E_s$ and vice versa, and from the compression modulus E_k (GOST) to the oedometer deformation modulus E_s (DIN) and vice versa.

References

1. Ignatova O.I., Svertilov A.A. Implementation of GOST 25100-2011 "Soils. Classification". *Soil Mechanics and Foundation Engineering.* 2013. Pp. 1–3.
2. Ignatova O.I., Bondarenko G.I., Trufanov A.N. Publication of GOST 12248-2010 "Soils. Methods for laboratory determination of strength and deformability characteristics". *Soil Mechanics and Foundation Engineering.* 2012. Vol. 49 No. 3. Pp. 115–118.
3. Larina T.A. Novye problemy v tehnicheskom normirovanii inzhenernyh izyskanij v stroitel'stve [New challenges in technical rate setting of engineering investigations in construction]. *Inzhenernye izyskanija.* 2011. No. 1. Pp. 10–12. (rus)
4. Gamsahurdija G.R., Larina T.A. Sovremennye problemy tehnicheskogo normirovanija v inzhenernyh izyskanijah [Contemporary issues in technical rate setting of engineering investigations]. *Inzhenernye izyskanija.* 2016. No. 1. Pp. 18–19. (rus)
5. Kashirskij V.I. Osobennosti ispytanij chetvertichnyh i dochetvertichnyh dispersnyh gruntov polevymi i laboratornymi metodami [Testing of quaternary and pre-quaternary dispersed soils by field and laboratory methods]. *Inzhenernye izyskanija.* 2013. No. 5. Pp. 34–42. (rus)
6. Boldyrev G.G., Idrisov I.H., Barvashov V.A. Vtoraja chast' Evrokoda 7 i sootvetstvujushhie standarty Rossii: shodstvo i razlichija [Second part of Eurocode 7 and corresponding standards of Russia: similarities and differences]. *Inzhenernye izyskanija*. 2010. No. 3. Pp. 22–26. (rus)
7. Boldyrev G.G., Mel'nikov A.V., Merkul'ev E.V., Novichkov G.A. Sravnenie metodov laboratornyh i polevyh ispytanij

Литература

1. Ignatova O.I., Svertilov A.A. Implementation of GOST 25100-2011 "Soils. Classification" // Soil Mechanics and Foundation Engineering. 2013. Pp. 1–3.
2. Ignatova O.I., Bondarenko G.I., Trufanov A.N. Publication of GOST 12248-2010 "Soils. Methods for laboratory determination of strength and deformability characteristics" // Soil Mechanics and Foundation Engineering. 2012. Vol.49. № 3. Pp. 115–118.
3. Ларина Т.А. Новые проблемы в техническом нормировании инженерных изысканий в строительстве // Инженерные изыскания. 2011. № 1. С. 10–12.
4. Гамсахурдия Г.Р., Ларина Т.А. Современные проблемы технического нормирования в инженерных изысканиях // Инженерные изыскания. 2016. № 1. С. 18–19.
5. Каширский В.И. Особенности испытаний четвертичных и дочетвертичных дисперсных грунтов полевыми и лабораторными методами // Инженерные изыскания. 2013. № 5. С. 34–42.
6. Болдырев Г.Г., Идрисов И.Х., Барвашов В.А. Вторая часть Еврокода 7 и соответствующие стандарты России: сходство и различия. // Инженерные изыскания. 2010. № 3. С. 22–26.
7. Болдырев Г.Г., Мельников А.В., Меркульев Е.В., Новичков Г.А. Сравнение методов лабораторных и полевых испытаний грунтов // Инженерные изыскания. 2013. № 14. С. 28–46.
8. Болдырев Г.Г., Скопинцев Д.Г. Методические вопросы определения модулей деформации дисперсных грунтов // Инженерные изыскания. 2016. № 10–11. С. 24–37.

gruntov [Comparison of laboratory and in-situ soil tests]. *Inzhenernye izyskanija.* 2013. No. 14. Pp. 28–46. (rus)

8. Boldyrev G.G., Skopincev D.G. Metodicheskie voprosy opredelenija modulej deformacii dispersnyh gruntov [Methodical problems of determining moduli of deformation in dispersed soils]. *Inzhenernye izyskanija.* 2016. No. 10-11. Pp. 24–37. (rus)
9. Lega E.A. *Issledovanie kachestva laboratornoj inzhenerno-geologicheskoj informacii, poluchaemoj v Rossii i za rubezhom: dis. … kand. geol.-min. nauk: 04.00.07* / Lega Elena Arkad'evna [Study on the quality of laboratory engineering and geological information received in Russia and abroad: Thesis … for PhD in Geology and Mineralogy: 04.00.07 / Lega Elena Arkad'evna]. Moskow, 1998. 124 p. (rus)
10. Boldyrev G.G., Idrisov I.H., Kondrat'ev R.A. Novye rossijskie tehnologii polevyh i laboratornyh ispytanij gruntov [New Russian technologies for in-situ and laboratory tests of soils]. *Inzhenernye izyskanija.* 2015. No. 8. Pp. 48–57. (rus)
11. Boldyrev G.G. Seminar-soveshhanie po polevym i laboratornym metodam ispytanij gruntov [Workshop-conference on in-situ and laboratory methods of soil testing]. *Geotehnika*. 2014. No. 3. Pp. 4–9. (rus)
12. Boldyrev G.G., Aref'ev D.V., Gordeev A.V. Opredelenie deformacionnyh harakteristik gruntov razlichnymi laboratornymi metodami [Determination of soil deformation characteristics by laboratory methods]. *Inzhenernye izyskanija.* 2010. No. 8. Pp. 16–23. (rus)
13. Boldyrev G.G., Mel'nikov A.V., Novichkov G.A. Chast' 3. Interpretacija rezul'tatov polevyh ispytanij s cel'ju opredelenija deformacionnyh harakteristik gruntov [Part 3. Interpretation of in-situ test results for determining soil deformation characteristics]. *Inzhenernye izyskanija*. 2014. No. 5-6. Pp. 86–97. (rus)
14. Boldyrev G.G., Mel'nikov A.V., Novichkov G.A. Chast' 4. Interpretacija rezul'tatov laboratornyh ispytanij s cel'ju opredelenija deformacionnyh harakteristik gruntov [Part 4. Interpretation of in-situ test results for determining soil deformation characteristics]. *Inzhenernye izyskanija.* 2014. No. 5-6. Pp. 98–105. (rus)
15. Förster W. (Hrsg.). *Bodenmechanik. TeubnerVerlag.* Stuttgart, 1998. 385 p.
16. Möller G. (Hrsg.). *Geotechnik: Bodenmechanik, 3 Auflage.* Ernst &Sohn. Berlin, 2016. 584 p.
17. Schmidt H. (Hrsg.). *Grundlagen der Geotechnik: Geotechniknach Eurocode.* Springer Vieweg. Wiesbaden, 2013. 716 p.
18. Witt K. J. (Hrsg.). *Grundbau-Taschenbuch: Teil 1: GeotechnischeGrundlagen.* Ernst &Sohn. Berlin, 2008. 815 p.
19. Khafizov R.M.Analysis of methods for determining the deformation characteristics of cohesive soil. *Soil Mechanics and Foundation Engineering.* 2016. Pp. 1–5.
20. Boldyrev G.G., Novichkov G.A. O vlijanii metoda opredelenija modulja deformacii na ego znachenie [Effect of method for determination modulus of deformation on its value]. *Geotehnika.* 2010. No. 3. Pp. 36–43. (rus)
21. Kashirskij V.I. Sravnitel'nyj analiz deformacionnyh harakteristik gruntov, poluchaemyh laboratornymi i polevymi metodami [Comparative analysis of soil deformation characteristics obtained by laboratory and in-situ tests]. *Geotehnika.* 2014. No. 05/06. Pp. 32–46. (rus)
22. Mirsayapov I.T., Koroleva I.V. Strength and deformability of clay soil under different triaxial load regimes that consider crack formation. *Soil Mechanics and Foundation Engineering.* 2016. Vol. 53. No.1. Pp. 5–11.
23. Trufanov A.N., Gabsalyamov G.U., Klimov V.Y., Sheveleva N.V. Determination of the deformation properties of clayey soils in the vend deposits in Saint Petersburg. *Soil Mechanics and Foundation Engineering.* 2013. Vol. 50.

9. Лега Е.А. Исследование качества лабораторной инженерно-геологической информации, получаемой в России и за рубежом: дис. … канд. геол.-мин. наук: 04.00.07 / Лега Елена Аркадьевна. М., 1998. 124 с.
10. Болдырев Г.Г., Идрисов И.Х., Кондратьев Р.А. Новые российские технологии полевых и лабораторных испытаний грунтов // Инженерные изыскания. 2015. № 8. С. 48–57.
11. Болдырев Г.Г. Семинар-совещание по полевым и лабораторным методам испытаний грунтов // Геотехника. 2014. № 3. С. 4–9.
12. Болдырев Г.Г., Арефьев Д.В., Гордеев А.В. Определение деформационных характеристик грунтов различными лабораторными методами // Инженерные изыскания. 2010. № 8. С. 16–23.
13. Болдырев Г.Г., Мельников А.В., Новичков Г.А. Часть 3. Интерпретация результатов полевых испытаний с целью определения деформационных характеристик грунтов // Инженерные изыскания. 2014. № 5-6. С. 86–97.
14. Болдырев Г.Г., Мельников А.В., Новичков Г.А. Часть 4. Интерпретация результатов лабораторных испытаний с целью определения деформационных характеристик грунтов // Инженерные изыскания. 2014. № 5-6. С. 98–105.
15. Förster W. (Hrsg.). Bodenmechanik. TeubnerVerlag. Stuttgart, 1998. 385 p.
16. Möller G. (Hrsg.). Geotechnik: Bodenmechanik, 3 Auflage. Ernst &Sohn. Berlin, 2016. 584 p.
17. Schmidt H. (Hrsg.). Grundlagen der Geotechnik: Geotechniknach Eurocode. Springer Vieweg. Wiesbaden, 2013. 716 p.
18. Witt K. J. (Hrsg.). Grundbau-Taschenbuch: Teil 1: GeotechnischeGrundlagen. Ernst &Sohn. Berlin, 2008. 815 p.
19. Khafizov R.M. Analysis of methods for determining the deformation characteristics of cohesive soil // Soil Mechanicsand Foundation Engineering. 2016. Pp. 1–5.
20. Болдырев Г.Г., Новичков Г.А. О влиянии метода определения модуля деформации на его значение // Геотехника. 2010. № 3. С. 36–43.
21. Каширский В.И. Сравнительный анализ деформационных характеристик грунтов, получаемых лабораторными и полевыми методами // Геотехника. 2014. № 05/06. С. 32–46.
22. Mirsayapov I.T., Koroleva I.V. Strength and deformability of clay soil under different triaxial load regimes that consider crack formation // Soil Mechanics and Foundation Engineering. 2016. Vol. 53. № 1. Pp. 5–11.
23. Trufanov A.N., Gabsalyamov G.U., Klimov V.Y., Sheveleva N.V. Determination of the deformation properties of clayey soils in the vend deposits in Saint Petersburg // Soil Mechanics and Foundation Engineering. 2013. Vol. 50. № 2. Pp. 50–55.
24. Pilyagin A.V. Determinations of the overall compression modulus of soils from test data // Soil Mechanics and Foundation Engineering. 2013. Vol. 50. № 2. Pp. 76–81.
25. СП 22.13330.2011 Основания зданий и сооружений. Актуализированная редакция СНиП 2.02.01-83*. М.: Минрегион России, 2011.
26. СП 22.13330.2016 Основания зданий и сооружений. Актуализированная редакция СНиП 2.02.01-83*. М.: Минстрой России, 2017.
27. DIN 4019: 2015-05 Baugrund. Setzungsberechnungen.
28. DIN 18135: 2012-04. Baugrund. UntersuchungvonBodernproben. EindimensionalerKompressionsversuch.
29. ГОСТ 12248-2010 Грунты. Методы лабораторного определения характеристик прочности и

No. 2. Pp. 50–55.

24. Pilyagin A.V. Determinations of the overall compression modulus of soils from test data. *Soil Mechanics and Foundation Engineering.* 2013. Vol. 50. No. 2. Pp. 76–81.
25. *SP 22.13330.2016 Osnovanija zdanij i sooruzhenij*. Aktualizirovannaja redakcija SNiP 2.02.01-83* [Russian Building Regulations 22.13330.2016 Foundations of buildings and structures. Updated edition of Building Codes (SNiP) 2.02.01-83*]. Moscow: Minstroj Rossii. 2017. (rus)
26. *SP 22.13330.2011 Osnovanija zdanij i sooruzhenij.* Aktualizirovannaja redakcija SNiP 2.02.01-83* [Russian Building Regulations 22.13330.2016 Foundations of buildings and structures. Updated edition of Building Codes (SNiP) 2.02.01-83*]. Moscow: Minstroj Rossii. 2011. (rus)
27. *DIN 4019: 2015-05 Baugrund. Setzungsberechnungen.*
28. *DIN 18135: 2012-04 Baugrund. Untersuchung von Bodernproben. Eindimensionaler Kompressionsversuch.*
29. *GOST 12248-2010. Grunty. Metody laboratornogo opredelenija harakteristik prochnosti i deformiruemosti.* [Russian State Standard GOST 12248-2010. Soils. Methods for laboratory determination of strength and deformability characteristics] Moscow: MNTKS. 2011. (rus)
30. Kalugina Ju.A. Sravnenie raschetnyh modulej deformacii, poluchennyh po dannym kompressionnyh ispytanij, soglasno nacional'nym standartam Rossii i Germanii [Comparison of design moduli of deformation obtained by compression tests after National Building Codes of Russia and Germany]. *Sbornik materialov mezhdunarodnoj nauchno-prakticheskoj konferencii: v treh tomah. Federal'noe gosudarstvennoe bjudzhetnoe obrazovatel'noe uchrezhdenie vysshego obrazovanija «Tjumenskij industrial'nyj universitet».* Tjumen': TIU, 2016. Pp. 74–81. (rus)
31. Pronozin Ja.A., Kalugina Ju.A. Sravnenie vlijanija harakteristik konsistencii na klassifikaciju pylevato-glinistyh gruntov soglasno rossijskih i germanskih normativnyh standartov [Comparison of the influence of consistency characteristics on classification of silty-clay soils in accordance with Russian and German regulatory standards]. *Sovremennye problemy nauki i obrazovanija.* 2015. No. 1-1. Pp. 332. (rus)
32. *GOST 5180-2015 Grunty. Metody laboratornogo opredelenija fizicheskih harakteristik* [Russian State Standard GOST 5180-2015 Soils. Methods for laboratory physical characteristics] Moscow: MGS, 2015. (rus)
33. *DIN EN ISO 17892-1:2015-03 Geotechnische Erkundung und Untersuchung - Laborversuche an Bodenproben. Teil 1: Bestimmung des Wassergehalts (ISO 17892-1:2014)*; Deutsche Fassung EN ISO 17892-1:2014.
34. *DIN EN ISO 17892-2:2015-03 Geotechnische Erkundung und Untersuchung - Laborversuche an Bodenproben. Teil 2: Bestimmung der Dichte des Bodens (ISO 17892-2:2014);* Deutsche Fassung EN ISO 17892-2:2014.
35. *DIN 18122-1: 1997-07 Baugrund, Untersuchung von Bodenproben - Zustandsgrenzen (Konsistenzgrenzen). Teil 1: Bestimmung der Fließ- und Ausrollgrenze.*
36. *GOST 25100-2011. Grunty. Klassifikacija* [Russian State Standard GOST 25100-2011. Soils. Classification] Moskow: MNTKS, 2011. (rus)
37. *DIN 18196: 2011-05 Erd- und Grundbau. Bodenklassifikationfürbautechnische Zwecke.*
38. *DIN EN ISO 14688-1: 2013-12 Geotechnische Erkundung und Untersuchung. Benennung, Beschreibung und Klassifizierung von Boden. Teil 1: Benennung und Beschreibung (ISO 14688-1: 2002 + Amd 1: 2013);* Deutsche Fassung EN ISO 14688-1: 2002 + A1: 2013.
39. *DIN EN ISO 14688-2: 2013-12 Geotechnische Erkundung und Untersuchung. Benennung, Beschreibung und Klassifizierung von Boden. Teil 1: GrundlagenfürBodenklassifizierung (ISO 14688-2: 2004 + Amd 1: 2013*); Deutsche Fassung EN ISO 14688-2: 2004 +

деформируемости. М.: МНТКС, 2011.

30. Калугина Ю.А. Сравнение расчетных модулей деформации, полученных по данным компрессионных испытаний, согласно национальным стандартам России и Германии. // Сборник материалов международной научно-практической конференции: в трех томах. Федеральное государственное бюджетное образовательное учреждение высшего образования «Тюменский индустриальный университет». Тюмень: ТИУ, 2016. С. 74–81.
31. Пронозин Я.А., Калугина Ю.А. Сравнение влияния характеристик консистенции на классификацию пылевато-глинистых грунтов согласно российских и германских нормативных стандартов // Современные проблемы науки и образования. 2015. № 1-1. С. 332.
32. ГОСТ 5180-2015 Грунты. Методы лабораторного определения физических характеристик. М.: МГС, 2015.
33. DINENISO 17892-1:2015-03 Geotechnische Erkundung und Untersuchung – Laborversuche an Bodenproben. Teil 1: Bestimmung des Wassergehalts (ISO 17892-1:2014); Deutsche Fassung ENISO 17892-1:2014.
34. DIN EN ISO 17892-2:2015-03 Geotechnische Erkundung und Untersuchung – Laborversuche an Bodenproben. Teil 2: Bestimmung der Dichte des Bodens (ISO 17892-2:2014); Deutsche Fassung EN ISO 17892-2:2014.
35. DIN 18122-1: 1997-07 Baugrund, Untersuchung von Bodenproben - Zustandsgrenzen (Konsistenzgrenzen).Teil 1: Bestimmung der Fließ- und Ausrollgrenze.
36. ГОСТ 25100-2011 Грунты. Классификация. М.: МНТКС, 2011.
37. DIN 18196: 2011-05 Erd- und Grundbau. Bodenklassifikationfürbautechnische Zwecke.
38. DINENISO 14688-1: 2013-12 Geotechnische Erkundung und Untersuchung. Benennung, Beschreibung und Klassifizierung von Boden. Teil 1: Benennung und Beschreibung (ISO 14688-1: 2002 + Amd 1: 2013); Deutsche Fassung EN ISO 14688-1: 2002 + A1: 2013.
39. 3DIN EN ISO 14688-2: 2013-12 Geotechnische Erkundung und Untersuchung. Benennung, Beschreibung und Klassifizierung von Boden. Teil 1: Grundlagenfür Bodenklassifizierung (ISO 14688-2: 2004 + Amd 1: 2013); Deutsche Fassung EN ISO 14688-2: 2004 + A1: 2013.
40. Рекомендации по определению значений модуля деформации грунтов по результатам компрессионных испытаний с использованием региональных корректировочных коэффициентов. Региональные нормативы градостроительного проектирования Томской обл. Томск: 2007. 21 с.

A1: 2013.

40. *Rekomendacii po opredeleniju znachenij modulja deformacii gruntov po rezul'tatam kompressionnyh ispytanij s ispol'zovaniem regional'nyh korrektirovochnyh kojefficientov. Regional'nye normativy gradostroitel'nogo proektirovanija Tomskoj obl.* [Recommendations for determining the values of soil deformation modulus by results of compression tests using regional adjustment coefficients. Regional Standards of Urban Planning of Tomsk Region]. Tomsk: 2007. 21 p. (rus)

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Approximated methods of estimation of the reliability of framed railway structures of railway bridges

Приближенные методы оценки надежности балочных пролетных строений железнодорожных мостов

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Abstract. The development of methods for rapid assessment of the reliability of span structures of beam railroad bridges is relevant in connection with the trend towards increasing loads from the reversing freight rolling stock and the speeds of movement on the main transport with a mixed cargo-and-passenger turnover. This problem is especially urgent for the creation and implementation of special technical conditions (STC) for the design of bridges on high-speed lines (BCM), as in Europe and in America, the process of improving the regulatory framework is actively being implemented and innovative developments in this field. Normative documents of the STC take into account only two opposite tendencies. The first is the reduction of equivalent loads from specialized high-speed rolling stock, the second is the increase in dynamic coefficients with an increase in speeds of up to 350 ... 400 km/h. At the same time, the documents being developed require the provision of a relatively heavy load from a special technological rolling stock when planning work on the maintenance of the railway track, as well as during the elimination of the consequences of accidents and other emergencies. The class of this load corresponds to C11, which is 78.5 % of the load of class C14, which is calculated for bridges on public railways. The authors suggests a method for estimating the reliability of a limited pass on beam span

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structures of a railway transport, the load from which is 10–20 % higher than the design load. A solution describing the probability of failure-free operation in the absence of a sudden failure at the level of 0.97-0.98 is described. Such a solution is relevant for railways with normal traffic conditions with the possibility of providing high-speed rail traffic. This technical method for rapid assessment of the reliability of span structures of beam railroad bridges be used as a basis for harmonizing National Standard and for the further evolution of the codes for HSR. Bridge authorities are therefore interested in agreed methods to assess the safety, reliability, durability of existing bridges.

Аннотация. Разработка методов экспресс оценки надежности пролетных строений балочных железнодорожных мостов актуально в связи с тенденцией к возрастанию нагрузок от обращающего грузового подвижного состава и скоростей движения на магистральном транспорте со смешанным грузопассажирским оборотом. Эта проблема особо актуальна при создании и внедрении специальных технических условий (СТУ) проектирования мостов на высокоскоростных магистралях (ВСМ), как и в Европе, так и в Америке идет активно процесс совершенствования нормативной базы и являются инновационными разработками в данной области. Нормативные документы СТУ учитывают только две противоположные тенденции. Первая – снижение эквивалентных нагрузок от специализированного высокоскоростного подвижного состава, вторая – увеличение динамических коэффициентов при увеличении скоростей движения до 350…400 км/ч. Одновременно разрабатываемые документы требуют обеспечения пропуска относительно тяжелой нагрузки от специального технологического подвижного состава при плановых работах по содержанию железнодорожного пути, а также при ликвидации последствий аварий и иных чрезвычайных ситуациях. Класс этой нагрузки соответствует С11, что составляет 78,5 % от нагрузки класса С14, являющегося расчетным для мостов на железных дорогах общего пользования. Авторы предлагают методику оценки надежности ограниченного пропуска по балочным пролетным строениям железнодорожного транспорта, нагрузка от которого на 10–20% превышает проектную расчетную нагрузку. Описано решение позволяющее оценить вероятность безотказной работы при отсутствии внезапного отказа на уровне 0,97–0,98. Такое решение актуально для железных дорог с обычным режимом движения с возможностью обеспечения проезда высокоскоростного состава. Представленный технический метод экспресс оценки надежности пролетных строений балочных железнодорожных мостов может использоваться в качестве основного инструмента для разработки национального стандарта и для дальнейшего развития нормативной базы для ВСМ. Управляющие компании заинтересованы в утверждении специализированных методов по оценке безопасности, надежности, прочности существующих мостов.

Introduction

A large number of eminent scientists, mechanics and engineers were involved in the study of reliability and the development of methods for its evaluation of various building systems. Considering the fact that this article deals with a certain element of the bridge structure – the beam span of the railway bridge, the following scientists and engineers deserve special attention: A.R. Rzhanicyn [1], K. Kapur and L. Lamberson [2], V.N. Smirnov [3, 5–8], V.V. Kondratov [4], V.V. Bolotin [9–11], J.G. Panovko [12] and many others.

Performance of the railway transportation network depends on the reliability of railway bridges. Developments in this field in the US sponsored by the U.S. Department of Transportation Research and Innovative Technology Administration [13].

Current high-speed railway (HSR) bridge design adopts the design concept which increases the stiffness of existing bridge introducing impact factor at the static load condition. However, given the extended total length and high speed of HSR system which has significant effect on resonance phenomenon, the in-depth evaluation of its dynamic performance shall be implemented. The optimal design of HSR bridges based on expected life-cycle cost (LCC) was very critical in securing the economic efficiency and the research has been underway in various ways [14].

The optimal policy has to be chosen based on minimum expected total LCC criterion including its effect on structural reliability and the expected costs associated with failure [15–18].

The results from the reliability analysis for the fatigue limit state are presented for various time periods from 10 to 100Â years and three cases of operating conditions [19–21].

As reported in Wisniewski et al. 2012, several years of research at National, European and International levels, including several European Projects, as well as practical implementations of these concepts on specific projects have demonstrated the benefits of incorporating advanced assessment and load rating in bridge assessment codes. However, the proposed advanced assessment methods for bridges as presented in the previous chapters are not yet included in most of the codes and recommendations or national or international regulations, where a standard basic assessment is normally applied. However, several countries have already included in their codes the possibility of using to some extent the proposed methods [22–27].

At the present moment, none is fully implementing all possible choices. It also becomes evident that USA and Canada are the countries where more of the proposed advanced methods are considered.

The European Convention for Constructional Steelwork (ECCS) has in its Technical Committee 6 – Fatigue (that also laid the basis for EN 1993 – Eurocode 3 – Part 1-9 – Fatigue) agreed to support the preparation of such European technical “Recommendations for the estimation of remaining Assessment of Existing Steel Structures; Remaining Fatigue Life First edition 2008 iv fatigue life”, that could be used as a basis for harmonising National procedures and for the further evolution of the Eurocodes [28].

This article is devoted to solving the problem related to the assessment of the reliability of beam span structures of railway bridges. The urgency of this problem stems from the need to improve the regulatory framework for the design of railway bridges, designed for high-speed traffic and mixed cargo and passenger traffic, and the development of special technical conditions (STC).

As the analysis of the existing regulatory framework shows, it is necessary to ensure that a relatively heavy load passes from the special technological rolling stock when planning work on the maintenance of the railway track, as well as during the liquidation of the consequences of accidents and other emergencies. The class of this load corresponds to C11, which is 78.5 % of the load of class C14, which is calculated for bridges on public railways.

In practical terms, the author proposes a technique for express estimation of the reliability of span structures, taking into account the skipping load exceeding the estimated design.

Allowable rail car loads are also expected to be increased by 10–20 % over the next few years. Knowledge of the current loadings to which railway bridges are subjected is imperative for accurate bridge evaluation.

Task description

Represented in foreign norms and rules (Eurocode, AAHSTO) methods for assessing the reliability of beam span structures of railway bridges include absolutely different algorithms for estimating and deducing the final result, which makes this express method developed and presented by the author in this article as original and separate as possible.

The live loads and dynamic amplification factors in the design codes are given for the design of new structures and can therefore be very conservative in some circumstances leading to structures failing their assessments. Consequently, it is often beneficial to use site-specific live loads and dynamic amplification factors when assessing existing railway bridges. In fact, this has been recognized worldwide.

The basis of the supposed approximate method for determining the reliability parameter and the probability of failure-free operation of the span structure according to the first limiting state (providing the carrying capacity) is the determination of the value of the mobile load intensities q (kN/m), corresponding to the limits of the normative and design fail-safe regions for given real geometric characteristics of the principal beams of span structure.

With this rapid assessment, it is considered that the refusal is of a sudden nature.

The failure group and the failure tree are not analyzed.

The intensity values indicated above are determined from the equation of the first limiting state:

$$2mkR^{\text{H}}A = n_1 p\Omega_p + n_2 q_{*}(1+\mu)\Omega_q, \qquad (1)$$

where m – operating ratio;

k – material resistance factor;

n_1 and n_2 – respectively, the reliability coefficients for the constant and temporary loads;

R^H – the standard resistance of the material, the values of these quantities are regulated by the relevant documents (SNiP);

p and *q* – intensity of constant and temporary loads;

Ω_p and Ω_q – the area of the lines of influence of the bending moment loaded respectively by the constant and temporary loads.

From the equation of the limiting state (1), the values of the intensity of the movable load q_* corresponding to the boundary of the normative region of fail-safe operation:

$$q_* = \frac{2mkR^{\mathrm{H}}A - n_1 p^{\mathrm{H}}\Omega_p}{n_2(1+\mu)\Omega_q}. \tag{2}$$

The boundary of the calculated region of fail-safe operation corresponds to the value of the intensity of the movable load q_{**} obtained from equation (2) with $n_1 = n_2 = 1$, $m = 1$ и $k = 1$.

$$q_{**} = \frac{2R^{\mathrm{H}}A - p^{\mathrm{H}}\Omega_p}{(1+\mu)\Omega_q}. \tag{3}$$

Safety factor at pass of normative loading $q_* = q^{\mathrm{H}}$ [1]:

$$\gamma = \frac{q_{**} - q_*}{\sqrt{\hat{q}_{**} + \hat{q}_*}} = \frac{\beta - 1}{\frac{1}{q_*}\sqrt{\hat{q}_{**} + \hat{q}_*}} = \frac{\beta - 1}{\sqrt{W_R^2\beta^2 + W_q^2}}, \tag{4}$$

where $\beta = \frac{q_{**}}{q_*}$, $\hat{q}_{**}$ and $\hat{q}_*$ – variance of random variables,

W_R^2 and W_q^2 – coefficient of variation of the standard resistance and the normative load.

The probability of failure-free operation of the span structure when passing the normative load q is given by:

$$P = \frac{1}{2} + \Phi\left(\frac{\beta - 1}{\sqrt{W_R^2\beta^2 + W_q^2}}\right), \tag{5}$$

where $\Phi\left(\frac{\beta - 1}{\sqrt{W_R^2\beta^2 + W_q^2}}\right)$ – the value of the Laplace function.

If the load exceeds the normative one, according to SNiP, equation (5) takes the form:

$$P = \frac{1}{2} + \Phi\left(\frac{\beta' - 1}{\sqrt{W_R^2\beta^2 + W_q^2}}\right), \tag{6}$$

where $\beta' = \dfrac{q_{**}}{q}$.

From the point of view of mathematical statistics, the mathematical expectation is shifted to the right and together with it the whole curve of the density of the load distribution without changing the shape of the curve.

We will estimate the value of the reliability index when passing over the span structure of the load, which is 15% higher than the standard load, with the values of the parameters $\beta = 1.1 \div 1.5$, $W_R = 0.05$ and $W_q = 0.0833$.

The results of calculating the reliability index as a function of the change in the calculated parameter β are shown in Figures 1 and 2, respectively.

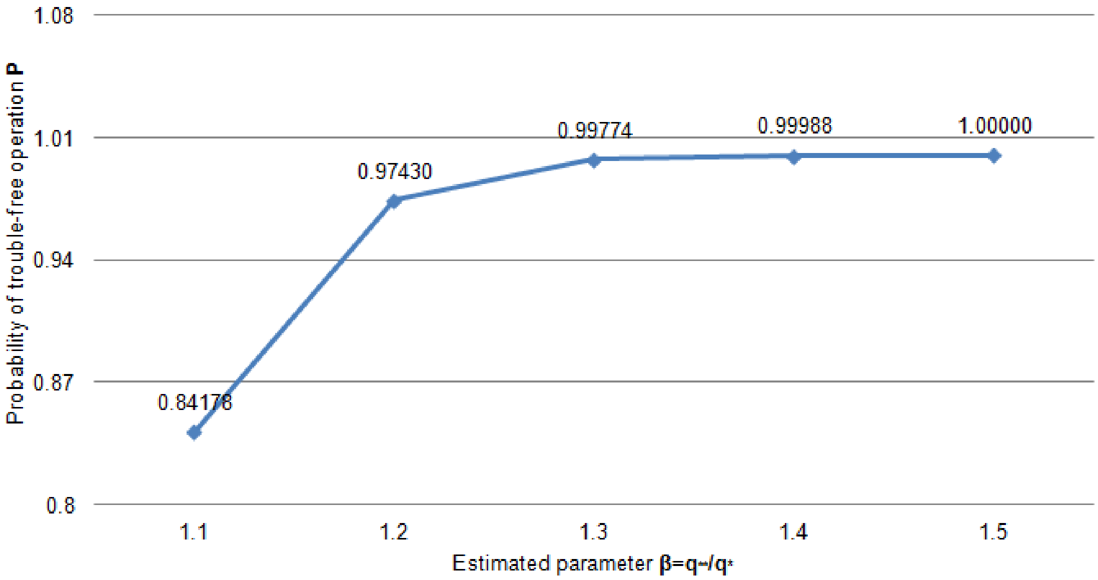

Figure 1. The values of the parameter P for the movement of the normative load

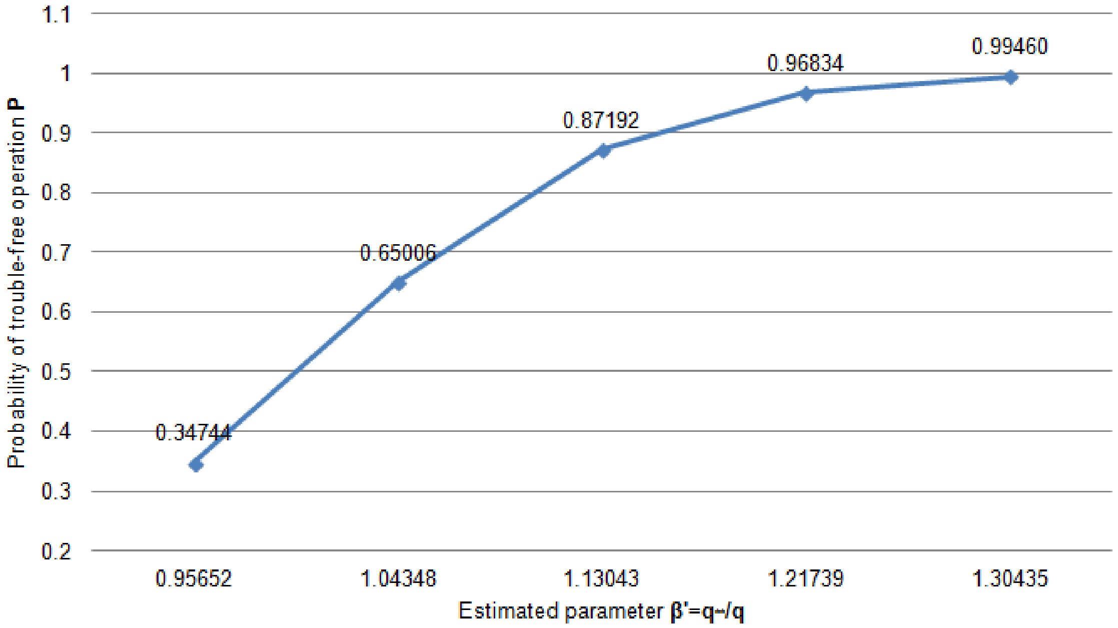

Figure 2. The values of the parameter P when moving the normative load, increased by 15%

Analyzing the graphs presented in Figures 1 and 2, it can be concluded that with an increase in the design load, there is a clear decrease in the reliability index, which is approximately 0.2 %.

Influence of the laws of distribution of extreme values of maxima for load and minima for strength

Equations 5 and 6 can be used when making the distribution laws of generalized loads and strengths different from the normal distribution laws.

For example, when accepting as distribution laws the Gumbel law (the law of distribution of extreme values) - the maximums for the load:

$$F(Q) = \exp\left[-\exp\left(-\frac{x-a_Q}{b_Q}\right)\right], \tag{7}$$

and minima for strength:

$$F(R) = 1-\exp\left[-\exp\left(\frac{x-a_Q}{b_Q}\right)\right], \tag{8}$$

using the first members of the Edgeworth series [2].

We obtain an analog of equation (1) for the probability of failure-free operation:

$$P = \frac{1}{2} + q\left(-\frac{a_R - a_Q + \Gamma'(1)(b_R + b_Q)}{\frac{\pi}{\sqrt{6}}\sqrt{b_R{}^2 + b_Q{}^2}}\right), \tag{9}$$

where, $\Gamma'(1) = -0.57721$ – first derivative of the gamma function $\Gamma'(x)$ with respect to x at the point $x=1$.

The analogues of equations (5) and (6) are:

$$P^{\text{H}} = \frac{1}{2} + \Phi\left(\frac{\beta - 1 + \Gamma'(1)\left(\frac{b_R + b_Q}{q_*}\right)}{\frac{\pi}{q_*}\sqrt{\frac{b_R{}^2 + b_Q{}^2}{6}}}\right), \tag{10}$$

$$P = \frac{1}{2} + \Phi\left(\frac{\beta' - 1 + \Gamma'(1)\left(\frac{b_R + b_Q}{q}\right)}{\frac{\pi}{q}\sqrt{\frac{b_R{}^2 + b_Q{}^2}{6}}}\right). \tag{11}$$

Application of the method for estimating the reliability of span structures intended for the movement of high-speed trains

When assessing the reliability of the span of bridges on highways designed exclusively for high-speed traffic, one should take into account the difference in the loading scheme of the influence lines by the time load in comparison with the loading scheme for Russian Construction Norms and Regulations SNiP 2.05.03-84 *.

In accordance with the "Instructions for the design of bridges for the main section of the Leningrad-Moscow high-speed passenger highway Center-South" (1990), a trainload was introduced from a specialized passenger electric vehicle with a design speed of up to 350 km/h in the form of a concentrated cargo Pi 180 kN and 170 kN.

Equivalent load is calculated by the formula:

$$q_{equal} = \frac{\sum P_i \, y_i}{\Omega}, \qquad (12)$$

where, y_i – ordinate of the influence line under the load P_i;

Ω – the area of the influence line (for the length of the influence line λ = 50…100 м it was 33.89 kN/m and 29.36 kN / m, respectively, instead of 138.3 kN / m and 137 kN / m according to Russian Construction Norms and Regulations SNiP 2.05.03-84*).

The value of the dynamic coefficient $1+\mu$ increased and became equal to $1+\mu = 1+\frac{24}{30+\lambda}$ instead of $1+\mu = 1+\frac{18}{30+\lambda}$ in Russian Construction Norms and Regulations SNiP.

he load reliability factor is n_2 = 1.15 regardless of the length of the load.

The value of the intensity of the mobile load corresponding to the boundary of the normative fail-safe area can be determined from expression (2), taking into account the changed parameters.

Similarly, the expression (3) is used to determine the value of q **.

The value of the coefficient β determined by formula (4) differs little from that obtained for passenger-and-freight railways, since it does not depend on the dynamic coefficient very much on the value of the numerators of expressions (2) and (3).

The coefficient of load reliability for a span structure at λ = 50 m, calculated according to the "Instructions" and Russian Construction Norms and Regulations SNiP 2.05.03-84 * practically coincide.

In accordance with the STC for the design, construction and operation of the high-speed Moscow-Kazan-Yekaterinburg railroad developed at the MRIT in 2013, equivalent loads from high-speed rolling stock have increased for span structures of *l* = 50 m and 100 m, respectively 3 % and 1.7 %.

V.V. Kondratov on the basis of theoretical and experimental studies, a new approach to the designation of a dynamic coefficient for high-speed motion was proposed [3]:

$$1+\mu = 1+\alpha\, K + \frac{3}{20+\ell}, \qquad (13)$$

where, $\alpha = \frac{v}{2 f \ell}$, *K*=0.8

f – basic frequency of free oscillations of the span structure, Hz;

ℓ – calculated span, m;

v – travel speed, m/sec.

For span structures ℓ = 50…100 m, the values of the dynamic coefficient are 1.3 ... 1.2 instead of the minimum value adopted for Russian Construction Norms and Regulations SNiP 2.05.03-84 * when calculating for strength 1.15..

According to the requirements of the normative documents for the construction of bridges at the bridge, these bridges must provide access to technological structures and transport units, with loading characteristics not exceeding the normative load of C11 (for the span structures $\ell = 50...100$ m is 108.66 kN/m and 107.64 kN/m, respectively 3.2 times the value of the standard load for high-speed rolling stock) at speeds of up to 120 km/h [4].

Consequently, in most cases it is the admission of a special load that is calculated to ensure the guaranteed load-bearing capacity of span structures.

Results and Discussion

Taking into account the structural reserves of strength, reflected in the values of the geometric characteristics of section A, and taking into account the reliability coefficients adopted in Russian Construction Norms and Regulations SNiP for materials and loads under extreme conditions, in a limited number of implementations, the load on beam girder bridges is allowed to exceed the design load by 20 ... 25% if the probability of failure-free operation of span structures (no sudden failures) is not lower than 0.95.

The AASHTO LRFR code (AASHTO LRFR, 2003) for Load and Resistance Factor Rating (LRFR) uses similar reliability based assessment principles and the load rating process to those of CAN/CSA-S6-06 [29, 30].

If, as a limiting state, the plastic hinge is formed in the main beam and the moment of resistance W is replaced by the plastic moment of resistance W_{pl}, then $A = \frac{W_{pl} h}{2}$, where, *h* – height of the main beam, the probability of failure-free operation when the above load is skipped increases to a value of 0.97 ... 0.98.

Due to the greater static stability of the load from the high-speed rolling stock, the probability of failure-free operation for the span structures of bridges on the HSR at a 10% excess of the standard load from the high-speed rolling stock is 0.99.

If the load from the special rolling stock is missed in case the load of the normative value for the C11 class is exceeded by 27 %, which corresponds to the skipping of the load of the C14 class, the reliability index decreases by 5.8 %.

Conclusions

1. The technique of express evaluation of beam span structures of railway bridges located on the railway network is developed, both with the usual driving mode and with high-speed.
2. The developed methodology allows to justify the possibility of skipping the load exceeding the design estimate for planned maintenance of the railway track, as well as for the elimination of the consequences of accidents and other emergencies with an acceptable degree of reliability.
3. This technique is based on special technical conditions (STU) and complements and is consistent with the foreign technical documentation related to the design of railway bridges on high-speed highways.

This technical method for rapid assessment of the reliability of span structures of beam railroad bridges be used as a basis for harmonizing National Standard and for the further evolution of the codes. Bridge authorities are therefore interested in agreed methods to assess the safety, reliability, durability of existing bridges.

The need to increase the lifetime of beam girder bridges, which have the highest percentage of distribution worldwide, and require a continuous assessment of operational reliability, determines the relevance of the calculations.

The application of the theoretical provisions outlined in the work makes it possible to provide the methodological support for the integrated system for assessing the suitability for the operation of beam girder bridge bridges with significant operating times and with increased operational loads, both in load and speed, which corresponds to the recent trend in the operation of the railway transport.

References

1. Rzhanicyn A.R. *Teorija rascheta stroitel'nyh konstrukcij na nadezhnost'* [Theory of calculating building structures for reliability]. Moscow: Strojizdat, 1978. 239 p. (rus)
2. Kapur K., Lamberson L. *Nadezhnost' i proektirovanie sistem* [Reliability and design of systems]. Moscow: Mir, 1980. 604 p. (rus)
3. Bogdanov G.I., Smirnov V.N. Iz istorii razrabotki proekta special'nyh tehnicheskih uslovij dlja proektirovanija mostov na VSM-2 Moskva-Kazan' [From the history of the development of the project of special technical conditions for the design of bridges at the Moscow-Kazan-HSR-2]. *Novye tehnologii v mostostroenii: mosty na vysokoskorostnyh magistraljah* [New technologies in bridge building: bridges on high-speed rail]. SPb: Peterburgskij gosudarstvennyj universitet putej soobshhenija, 2013. Pp. 57–67. (rus)
4. Kondratov V.V. Rezul'taty jeksperimental'nyh i teoreticheskih issledovanij dinamicheskogo vozdejstvija vysokoskorostnyh poezdov na proletnye stroenija mostov [The results of experimental and theoretical studies of the dynamic effect of high-speed trains on span structures of bridges] *Novye tehnologii v mostostroenii: mosty na vysokoskorostnyh magistraljah* [New technologies in bridge building: bridges on high-speed rail]. SPb: Peterburgskij gosudarstvennyj universitet putej soobshhenija, 2013. Pp. 13–23. (rus)
5. Benin A.V., D'jachenko L.K., Smirnov V.N. Osobennosti proektirovanija i stroitel'stva mostov vysokoskorostnoj zheleznodorozhnoj magistrali «Moskva-Kazan'» [Features of designing and building bridges of the high-speed railway "Moscow-Kazan"]. *Izvestija peterburgskogo universiteta putej soobshhenija.* 2015. No. 4(45). Pp. 15–20. (rus)
6. Smirnov V.N. R*aschet mostov vysokoskorostnyh zheleznodorozhnyh magistralej na prodol'nye sily* [Calculation of bridges of high-speed railways for longitudinal forces]. SPb: Peterburgskij gosudarstvennyj universitet putej soobshhenija, 2013. 64 p. (rus)
7. Smirnov V.N., Pal'ceva A.I. Proektirovanie mostov dlja vysokoskorostnyh zheleznodorozhnyh magistralej [Designing bridges for high-speed railways]. *Promyshlennoe i grazhdanskoe stroitel'stvo.* 2011. No. 5. Pp. 30–31. (rus)
8. Smirnov V.N. Osobennosti vysokoskorostnogo dvizhenija zheleznodorozhnyh jekspressov po mostam [Features of high-speed movement of railway express trains by bridges]. *Novye tehnologii v mostostroenii (ot proshlogo k budushhemu)* [New technologies in bridge building (from the past to the future)]. SPb: Peterburgskij gosudarstvennyj universitet putej soobshhenija, 2015. Pp. 50–57. (rus)
9. Bolotin V.V. *Primenenie metodov teorii verojatnostej i teorii nadezhnosti v raschetah sooruzhenij* [The application of methods of probability theory and reliability theory in the calculations of structures]. Moscow: Strojizdat, 1971. 255 p. (rus)
10. Bolotin V.V. *Statisticheskie metody v stroitel'noj mehanike. 2-e izd.* [Statistical methods in construction mechanics. 2nd ed.]. Moscow: Strojizdat, 1965. 279 p. (rus)
11. Bolotin V.V. *Dinamicheskaja ustojchivost' uprugih system* [Dynamic stability of elastic systems]. Moscow: Gosudarstvennoe izdatel'stvo tehniko-teoreticheskoj literatury, 1956. 595 p. (rus)
12. Panovko J.G. *Mehanika deformiruemogo tverdogo tela: sovremennye koncepcii, oshibki i paradoksy* [Mechanics of a deformable solid: modern concepts, errors and paradoxes]. Moscow: Nauka, 1985. 288 p. (rus)
13. Rakoczy A.M. *Development of System Reliability Models for Railway Bridges.* 2012. 212 p.
14. Rakoczy A.M., Nowak A.S. Development of System Reliability Models for Railway Bridges. *International Journal*

Литература

1. Ржаницын А.Р. Теория расчета строительных конструкций на надежность. М: Стройиздат, 1978. 239 с.
2. Капур К., Ламберсон Л. Надежность и проектирование систем. М: Мир, 1980. 604 с.
3. Богданов Г.И., Смирнов В.Н. Из истории разработки проекта специальных технических условий для проектирования мостов на ВСМ-2 Москва-Казань // Новые технологии в мостостроении: мосты на высокоскоростных магистралях. СПб: Петербургский государственный университет путей сообщения, 2013. С. 57–67.
4. Кондратов В.В. Результаты экспериментальных и теоретических исследований динамического воздействия высокоскоростных поездов на пролетные строения мостов // Новые технологии в мостостроении: мосты на высокоскоростных магистралях. СПб: Петербургский государственный университет путей сообщения, 2013. С. 13–23.
5. Бенин А.В., Дьяченко Л.К., Смирнов В.Н. Особенности проектирования и строительства мостов высокоскоростной железнодорожной магистрали «Москва-Казань» // Известия петербургского университета путей сообщения. 2015. № 4(45). С. 15–20.
6. Смирнов В.Н. Расчет мостов высокоскоростных железнодорожных магистралей на продольные силы. СПб: Петербургский государственный университет путей сообщения, 2013. 64 с.
7. Смирнов В.Н., Пальцева А.И. Проектирование мостов для высокоскоростных железнодорожных магистралей // Промышленное и гражданское строительство. 2011. № 5. С. 30–31.
8. Смирнов В.Н. Особенности высокоскоростного движения железнодорожных экспрессов по мостам // Новые технологии в мостостроении (от прошлого к будущему). СПб: Петербургский государственный университет путей сообщения, 2015. С. 50–57.
9. Болотин В.В. Применение методов теории вероятностей и теории надежности в расчетах сооружений. Москва: Стройиздат, 1971. 255 с.
10. Болотин В.В. Статистические методы в строительной механике. 2-е изд., перераб. и доп. М.: Стройиздат, 1965. 279 с.
11. Болотин В.В. Динамическая устойчивость упругих систем. М: Государственное издательство технико-теоретической литературы, 1956. 595 с.
12. Пановко Я.Г. Механика деформируемого твердого тела: современные концепции, ошибки и парадоксы. М: Наука, 1985. 288 с.
13. Rakoczy A.M. Development of System Reliability Models for Railway Bridges, 2012. 212 p.
14. Rakoczy A.M., Nowak A.S. Development of System Reliability Models for Railway Bridges // International Journal of Steel Structures. 2017. Vol. 17. Pp. 339–349
15. AASHTO LRFR, 2003. Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges. USA: American Association of State Highway and Transportation Officials.
16. AASHTO MBE, 2008. MBE-1-M, Manual for Bridge Evaluation. 1st ed. Washington, DC: AASHTO.
17. Alampalli S., Ettouney M. Bridge Load Testing: Cost-Benefit Evaluation. Orlando, Florida, USA, ASCE, Structures Congress 2010.
18. Baker J., Schubert M., Faber M. On the assessment of Robustness // Structural Safety. 2008. Vol. 30(1). Pp. 253–267.

of Steel Structures. 2017. Vol. 17. Pp. 339–349.

15. *AASHTO LRFR, 2003.* Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges. USA: American Association of State Highway and Transportation Officials.
16. *AASHTO MBE, 2008.* MBE-1-M, Manual for Bridge Evaluation. 1st ed. Washington, DC: AASHTO.
17. Alampalli S., Ettouney M. *Bridge Load Testing: Cost-Benefit Evaluation.* Orlando, Florida, USA, ASCE, Structures Congress. 2010.
18. Baker J., Schubert M., Faber M. On the assessment of Robustness. *Structural Safety.* 2008. Vol. 30(1). Pp. 253–267.
19. Rakoczy A.M., Nowak A.S., Dick S. Fatigue reliability model for steel railway bridges. *Structure and Infrastructure Engineering.* 2016. Vol. 12. Pp. 1602–1613.
20. Beck J., Au S. Bayesian updating of structural models and reliability using Markov Chain Monte Carlo Simulation. *Journal of Engineering Mechanics.* 2002. Vol. 128(4). Pp. 380–391.
21. Bocchini P., Saydam D., Frangopol D. Efficient, accurate, and simple Markov chain model for the life-cycle analysis of bridge groups. *Structural Safety*. 2013. Vol. 40. Pp. 51–64.
22. Wisniewski D., Casas J., Ghosn M. Codes for safety assessment of existing bridges. Current state and further development. *Structural Engineering International*. 2012. Vol. 22(4). Pp. 552–561.
23. Brühwiler E. Extending fatigue life of structures beyond 100 years using monitored data. *Proceedings of IABSE Workshop on Safety, Failures and Robustness of Large Structures.* Helsinki (Finland), 2013.
24. Casas J. Reliability-based assessment of masonry arch bridges. *Construction and Building Materials*. 2011. Vol. 25(4). Pp. 1621–1631.
25. Casas J., Cavaco E., Neves L. Quantifying redundancy and robustness of Structures. *Proceedings of IABSE Workshop on Safety, Failures and Robustness of Large Structures.* Helsinki (Finland), 2013.
26. Catbas F., Susoy M., Frangopol D. Structural health monitoring and reliability estimation: Long span truss bridge application with environmental monitoring data. *Engineering Structures.* 2008. Vol. 30(9). Pp. 2347–2359.
27. Cavaco E.S., Casas J.R., Neves L., Huespe A. Robustness of corroded reinforced concrete structures – a structural performance approach. *Structure and Infrastructure Engineering.* 2013. Vol. 9(1). Pp. 42–58.
28. Kühn B., Lukić M., Nussbaumer A., Günther H.-P., Helmerich R., Herion S., Kolstein M.H., Walbridge S., Androic B., Dijkstra O., Bucak Ö. *Assessment of Existing Steel Structures: Recommendations for Estimation of Remaining Fatigue Life. First Edition.* 2008. 108 p.
29. *CAN/CSA-S6.1-06, 2006.* Commentary on CAN/CSA-S6-06, Canadian Highway Bridge Design Code, Canadian Standards Association. Canada: Canadian Standards Association.
30. *CAN/CSA-S6-06, 2006.* Canadian Highway Bridge Design Code. Canada: Canadian Standards Association.
31. Vatin N.I., Rybakov V.A Raschet metallokonstrukcij: sed'maja stepen' svobody [alculation of metal structures: the seventh degree of freedom]. *Strojprofil'.* 2007. No. 2. Pp. 60–63. (rus)

19. Rakoczy A.M., Nowak A.S., Dick S. Fatigue reliability model for steel railway bridges // Structure and Infrastructure Engineering. 2016. Vol. 12. Pp. 1602–1613.
20. Beck J., Au, S. Bayesian updating of structural models and reliability using Markov Chain Monte Carlo Simulation // Journal of Engineering Mechanics. 2002. Vol. 128(4). Pp. 380–391.
21. Bocchini P., Saydam D., Frangopol D. Efficient, accurate, and simple Markov chain model for the life-cycle analysis of bridge groups // Structural Safety. 2013. Vol. 40. Pp. 51–64.
22. Wisniewski D., Casas J., Ghosn M. Codes for safety assessment of existing bridges. Current state and further development // Structural Engineering International. 2012. Vol. 22(4). Pp. 552–561.
23. Brühwiler E. Extending fatigue life of structures beyond 100 years using monitored data // Proceedings of IABSE Workshop on Safety, Failures and Robustness of Large Structures. Helsinki (Finland), 2013.
24. Casas J. Reliability-based assessment of masonry arch bridges // Construction and Building Materials. 2011. Vol. 25(4). Pp. 1621–1631.
25. Casas J., Cavaco E., Neves L. Quantifying redundancy and robustness of Structures // Proceedings of IABSE Workshop on Safety, Failures and Robustness of Large Structures. Helsinki (Finland), 2013.
26. Catbas F., Susoy M., Frangopol D. Structural health monitoring and reliability estimation: Long span truss bridge application with environmental monitoring data // Engineering Structures. 2008. Vol. 30(9). Pp. 2347–2359.
27. Cavaco E.S. Casas J.R., Neves L., Huespe A. Robustness of corroded reinforced concrete structures – a structural performance approach // Structure and Infrastructure Engineering. 2013. Vol. 9(1). Pp. 42–58.
28. Kühn B., Lukić M., Nussbaumer A., Günther H.-P., Helmerich R., Herion S., Kolstein M.H., Walbridge S., Androic B., Dijkstra O., Bucak Ö. Assessment of existing steel structures: recommendations for estimation of remaining fatigue life. First Edition. 2008. 108 p.
29. CAN/CSA-S6.1-06, 2006. Commentary on CAN/CSA-S6-06, Canadian Highway Bridge Design Code, Canadian Standards Association. Canada: Canadian Standards Association.
30. CAN/CSA-S6-06, 2006. Canadian Highway Bridge Design Code. Canada: Canadian Standards Association.
31. Ватин Н.И., Рыбаков В.А. Расчет металлоконструкций: седьмая степень свободы // Стройпрофиль. 2007. № 2. С. 60–63.

Indeykin A.V., Chizhov S.V., Shestakova E.B., Antonyuk A.A., Kulagin N.I., Smirnov V.N., Karpov V.V., Golitsynsky D.M. Approximated methods of estimation of the reliability of framed railway structures of railway bridges. *Magazine of Civil Engineering*. 2017. No. 7. Pp. 150–160. doi: 10.18720/MCE.75.15.

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Expanding cements hardening within the limited deformations conditions

Гидратация и структурообразование при твердении расширяющихся цементов в условиях ограниченных деформаций

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Key words: hydration product; amorphous phase; calcium sulfoaluminate; x-ray diffraction; Portlandite; ettringite

Ключевые слова: продукты гидратации; аморфная фаза; сульфоалюминат кальция; рентгеновская дифракция; портландит; эттрингит

Abstract. The features of expanding cement strength and structure formation at different constrained deformation conditions were studied (without constraint of deformations, uniaxial constraint of deformations, biaxial constraint of deformations, and triaxial constraint of deformations). The study was conducted for three doses of 5 %, 15 % and 25 % of expander on the calcium sulfoaluminate basis in Portland cement of CEM 42.5 mark. For experimental data interpretation in the analysis of structure and phase composition, the combination of methods of x-ray diffraction, differential scanning calorimetry and scanning electron microscopy were used. The constraint of expansion deformations has a significant impact on the volume and nature of the expanding cement pore structure, and leads to the formation in the volume of cement stones of black agglomerates, which represent the mixture of portlandite and ettringite crystals, uniformly distributed in the amorphous CSH-phase. It has been established that by increasing the expander amount and the degree of deformation constraint increases the number of black agglomerates and an increase in compressive strength.

Аннотация. Изучались особенности набора прочности и структурообразования расширяющихся цементов в различных условиях ограничения деформаций (без ограничения деформаций, одноосное ограничение деформаций, двухосное ограничение деформаций, трехосное ограничение деформаций). Исследования проведены для трех дозировок 5 %, 15 % и 25 % расширяющейся добавки на основе сульфоалюмината кальция в портландцемент марки CEM 42,5. Для интерпретации экспериментальных данных при анализе структуры и фазового состава использовалась комбинация методов рентгеновской дифракции, дифференциальной сканирующей калориметрии и сканирующей электронной микроскопии. Ограничение деформаций расширения оказывает существенное влияние на объём и характер поровой структуры расширяющегося цемента, а так же приводит к формированию в объеме затвердевшего цементного камня чёрных агломератов представляющих собой смесь кристаллов портландита и эттрингита, равномерно распределённых в аморфной CSH-фазе. Установлено, что при увеличении количества расширяющей добавки и степени ограничения деформаций возрастает число черных агломератов и наблюдается рост прочности при сжатии.

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Introduction

One of the areas of effective use of expanding cements and concretes based on them is butt joints sealing of concrete and reinforced concrete structures [1, 2]. Controlled expansion allows ensuring the functional characteristics uniformity of structures [3–5]. For these purposes, high workability mixtures are mainly used [6], which evenly fill the butt joints volume and the hardening at the flat or volumetric constraint deformation conditions expansions [7–8]. The energy, released during the expanding cement system hardening, is spent for prestressing of the butt joint concrete that provides the hardening concrete structure compaction and creates reinforcement stress when butt joint reinforcing. The uniformity of sectional construction functional characteristics with butt joints sealing using expanding concretes is determined not only by the concrete composition and expanding cement properties [9–12], but also by the expansion deformation constraint conditions.

There are publications of volumetric expansion magnitude studies [13–16], which describe the structure and properties formation mechanisms, depending on the expansion deformation magnitude, expanding cement composition and hardening conditions in the absence of constraint of expansion deformations, while for sulfoaluminate systems the main criterion for expansion is the crystallization pressure, caused by ettringite intense crystallization [17, 18]. However, the structure and properties formation mechanisms of expanding cements at the constrained deformation conditions have been studied to date only in part of the reinforcement post-stress by the expanding concretes [19].

Materials and method

In order to study the influence of constrained deformation conditions on the structure and properties of hardened cement stone and concrete, on its basis the comprehensive studies of expanding cement were made, prepared by homogenizing of the base portland cement of CEM 42.5 mark with low content of $C3A$= 3.2% and expander on the calcium sulfoaluminate and calcium sulfate dihydrate basis in stechiometric ratio, required for the ettringite formation. The expanding cement is prepared by intergrinding of Portland cement with expander components in the amount of 5 %, 15 % and 25 % by Portland cement mass [20].

The study of expanding cement properties at the different constrained deformation conditions were conducted in accordance with method [16], through the preparation of cement-sand mortar in the ratio of the expanding cement: sand = 1 : 1 with water-cement ratio of 0.30.

To determine the linear expansion the beam samples with dimensions 40x40x160 mm were made. Measurement frequency was 1, 2, 7, 14, 21 and 28 days. Linear expansion of each beam sample (%), was calculated according to the following formula:

$$\Delta l = \left[(n_2 - n_1) / l \right] \times 100, \tag{1}$$

where n1 – a reading taken during the measurement of a beam sample that was released from the mold after 1 day from the start of cement mixing, mm;

l – length of the standard sample, l =160 mm.

As a linear expansion of tensile cement the average results of three measurements of beam samples was assumed.

The result was rounded to 0.01 %.

The constraint of deformations was carried out using the dynamometric rings for biaxial constraint of deformations and with the use of dynamometric conductors for uniaxial constraint of deformations (Fig.1). The expansion pressure of the beam sample in the ring (MPa) was calculated according to the following formula:

$$\sigma = 2\Delta l, \tag{2}$$

where Δl – deformation value of the ring in the direction of the sample axis, mm.

The cement expansion pressure was calculated as an average result of three measurements of the beam samples molded from a single batch. Calculations were carried out with accuracy of 0.01 MPa.

Figure 1. The influence of expansion deformation constraint degree on the change in volume of the expanding concrete of B 35 grade (2 ÷ 4).
1 – fine-grained concrete without expanders at normal hardening; 2 – fine-grained concrete with 15% of expander, the hardening without the deformation constraint (expansion is 185 mm/m); 3 – fine-grained concrete with 15% of expander, uniaxial expansion constraint (expansion pressure is 3.9 N/mm2); 4 – fine-grained concrete with 15% of expander, biaxial expansion constraint (expansion pressure is 15.3 N/mm2)

The study of the structure and properties of expanding cement at the triaxial expansion constraint conditions was conducted according to the method [21], the cement-water paste of normal density for the manufacture of samples was used. For studies of the influence of triaxial deformation constraint on the structure and properties of expanding cements and concrete, the measuring chamber was used. The overall view of the measuring chamber is shown in (Fig. 2). The camera consists of a massive metal vessel with the screw cap. The cap is equipped with a liquid pressure gauge and valve for air drain from the chamber. The samples of cylindrical shape of diameter 50...70 mm from the cement stones or cement-sand mortar are prepared for studies, the ends of which are closed with plastic washers of appropriate diameter, which eliminates the contact of the sample with outdoor environment.

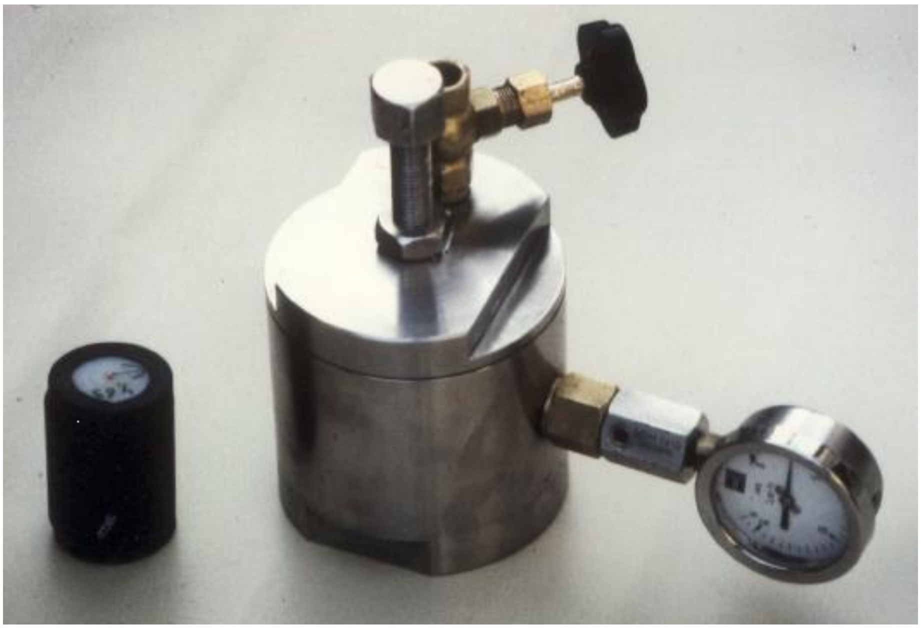

Figure 2. Measuring chamber for the structure and properties of expanding cement study that hardens at the isotropic deformation constraint conditions

After hardening at normal conditions for 24 hours, the samples are released from the formwork and placed in a heat-shrinking tube of the appropriate diameter, which, after short-time heating with hot air, is securely pre-stress the sample all around except for its contact with the outdoor environment. In this case, the expandable tube can deform without preventing the expansion process. The prepared sample is placed in a measuring vessel filled with water, went trough a preliminary treatment for air elimination. The vessel is tightly closed with screw cap. In the vessel there is a pre-stress of 0.01 MPa, which is taken as the initial reference. The measuring chamber is in the room with a constant temperature of 20 ^{0}C to complete stabilization of the expansion process. Indicators of the pressure gauge are recorded every hour in an automatic mode with the use of special recording device.

Studies of the composition of hydration products were carried out using quantitative X-ray diffraction (XRD) and differential scanning calorimetry (DSC) methods. X-ray phase analysis was carried out on a powder X-ray diffractometer ARL X'TRA (Switzerland) equipped with a narrow-focus tube 2200 W (Cu anodes) and an energy-dispersive solid-state detector with a Peltier cooler. The registration of the diffractograms was carried out in a step-by-step mode (with a step of 0.02° 2Θ), in the angle range 2Θ°=4-70°. For qualitative phase analysis, the database ICDD PDF-2 was used. The analysis was carried out on interplanar distances in manual mode using the Hanavalt method and in semi-automatic mode using the software Oxford Crystallographica Search Match. Non-standard quantitative X-ray phase analysis by the Rietveld method was carried out using the software Siroquant 3 Sietronics Pty Ltd. For all phases, the following parameters were specified: scale factor, zero offset of the instrument counter, background parameters (the Chebyshev polynomial of 5th degree), unit cell parameters. Also, the profile parameters varied during the adjustment - the profile function Pearson VII (U, V, W in the Kalotti dependence) was used.

To study the thermal behavior of the samples, the apparatus for carrying out thermogravimetric analysis (TGA) and differential scanning calorimetry (DSC) SETARAM LABSYS TGA / DSC / DTA (France) was used. The obtained samples were abraded in an agate mortar with an agate pestle and then sieved through a sieve with a 008 cell. The abrasion process was repeated until the entire sample passed through a sieve. Then we weighed a sample of each sample with a mass of 50 ± 2 mg and weighed on an analytical scale with an accuracy of 0.0001 g. The sample was then placed in a corundum crucible for analysis. Measurements of the thermal behavior of the samples were carried out in a temperature range of 50 °C to 1000 °C with a stabel heat rate of 10 °C/min in the atmosphere of air.

The microstructure of the samples was studied on a FEI Quanta 200 scanning electron microscope with an Apollo 40 elemental analysis attachment. Samples were fixed to the sample holders with carbon scotch tape. Studies of the microstructure were carried out in the regime of deep vacuum.

Results

Figure 3 presents the results of a study of the compressive strength of fine-grained concrete, depending on the conditions of limited expansion deformations.

It was found that with an increase in the degree of deformations restriction, an increase in the compressive strength is observed. In the case of triaxial and biaxial constraints, an increase in strength is observed with an increase in the amount of the expanding admixture in fine-grained concrete. With uniaxial deformation constraint, the drop in strength is observed when the content of the expanding additive in the cement is 15 %. During the hardening process of the fine-grained concrete with expanding admixture without deformation restrictions, the compressive strength decreases even when the content of the expanding admixture in cement is 5 %.

It has been established that with increase of the deformation constraint degree there is the increase in strength under compression. In the case of biaxial deformation constraint, the strength increase is observed with the expander amount increase in the fine-grained concrete composition. The strength growth under compression for uniaxial deformation constraint is terminated when the expander content in the fine-grained concrete is 2.5 % by the total mass. While fine-grained concrete hardening with the expander without deformation constraint, it significantly reduces the strength with the expander content increase more than 0.5 % by the total mass. That confirms the results of fine-grained concrete strength properties studies on the expanding cements basis [22–26], where a sharp decrease in strength with calcium sulfoaluminate content increase in the expanding cement composition was observed [16, 27, 28].

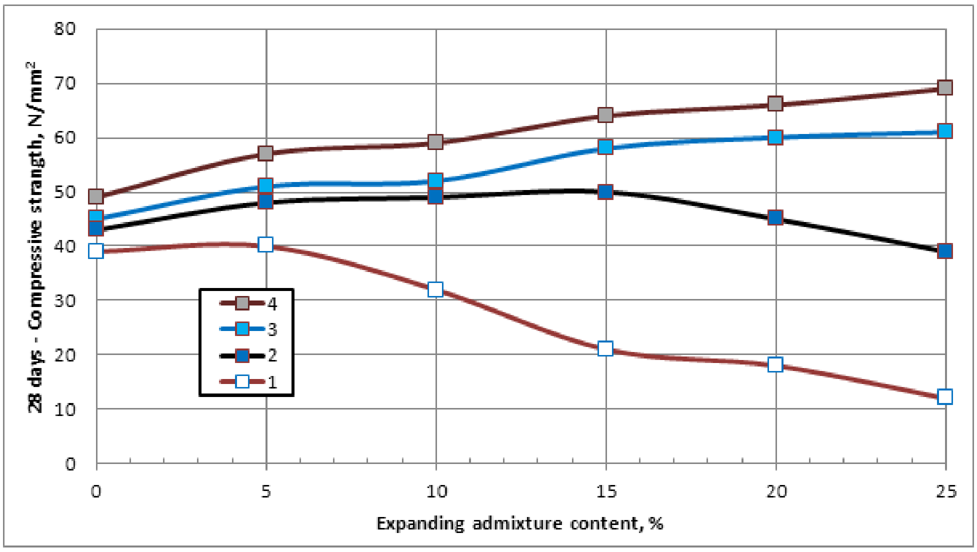

Figure 3. The influence of expansion deformation constraint conditions on the strength under the concrete compression: 1 – without deformation constraint; 2 – uniaxial deformation constraint; 3 – biaxial deformation constraint; 4 – triaxial deformation constraint

The analysis of the study results showed that the expansion deformation constraint has a significant impact on the volume and nature of the expanding cement pore structure while hardening. It also has been established that the extent of this influence increases in proportion to expansion potential capacity increase.

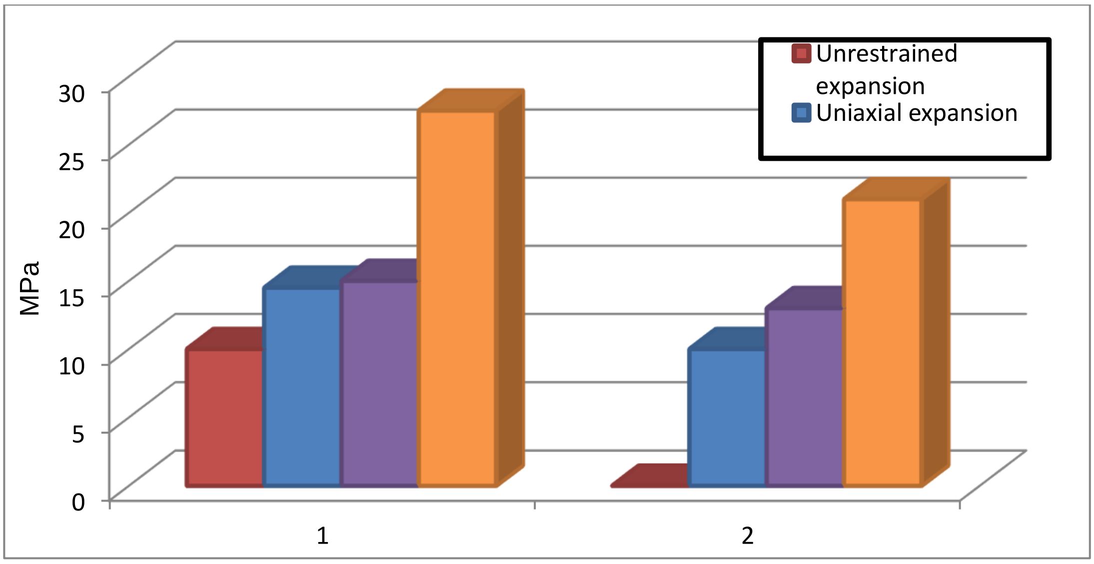

Figure 4. The influence of constraint degree of the expansion process on the strength under compression, porosity and expansion pressure: 1 – strength under compression; 2 – pressure under stable expansion

As the expansion potential capacity, the expansion value during sample hardening without deformation constraint at normal temperature-humidity conditions has been established (temperature – 20 ^{0}C, humidity – 50%) [21, 29]. During the introduction in the base cement composition of 5 % of expander, the deformation constraint degree has not substantially influence on the cement stone strength and pore structure. Compared to the base Portland cement strength under compression, the strength under compression increases by 6-8% regardless of the constrained deformation conditions.

During the introduction in the cement composition of 15% of expander, the expansion potential capacity increases by 12 mm/m and in this case, the sample strength under compression, hardening under the uniaxial and biaxial expansion constraint increases by 7 % and 10 % respectively (Fig. 4). At the same time, the significant differences in volume and nature of the pore structure were not established. While expanding cement hardening, which contains in its composition 25% of expander, the expansion potential capacity increases by 185 mm/m. In this case, while uniaxial expansion deformation constraint the self-destruction of concrete structure at the sample centerline is observed that do not have expansion constraint (Fig. 1). While biaxial deformation constraint, a fairly dense and solid structure of cement stone is formed (Fig. 3) and deformation in “free” direction does not exceed 5% of the expansion potential capacity value.

Of special scientific and practical interest are the study results of expanding cement structure formation processes under triaxial expansion deformation, when trough the entire volume of hardened cement stone the anomalous inclusions of the black color are fairly formed, which optically resembles the filler grains with a size of 0.1 to 5.0 mm. (Fig. 5)

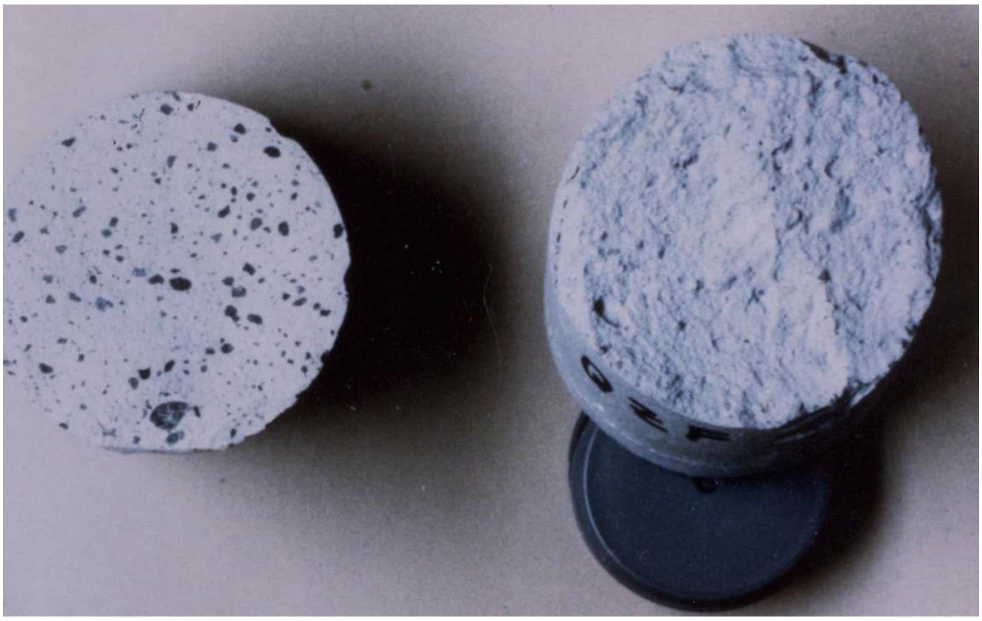

Figure 5. The formation of black aggregates in the cement stone volume, hardening at isotropic deformation constraint conditions (left sample) and the absence of aggregates during hardening without deformation constraint (right sample) at an expansion potential capacity of 185 mm/m

The studies performed using optical polarizing microscope and electronic scanning microscope, did not allow identifying a clear border zone between the cement stone matrix, which has a gray color and black agglomerates. The agglomerate microstructure is extremely dense, continuous, and substantially free from micro cracks and pores. In this case the studies of hydration products, performed using DSC and XRF analysis has allowed establishing that for the cement stone matrix the predominant content of hydrosilicate phase and hydrosulfoaluminate of monosulfate form is typical.

Table 1. The results of XRF, DSC and ISA of hydration products after the statistical processing

Hydration products	**Sample 1**	**Sample 2**	**Sample 3**	**Sample 4**
Ettringite	36.7 %	62.3 %	57.7 %	55.8 %
Monosulfate	22 %	small amount	small amount	small amount
Portlandite	4 %	5.8 %	5.4 %	4.9 %
Calcite	6.4 %	9.0 %	10.2 %	8.9 %

Note: sample 1 – grey part or matrix; samples 2 and 4 – black agglomerates; sample 3 –transition zone between the “matrix” and black agglomerates.

In the study of the black agglomerate phase composition, a preferential content of ettringite was established, as well as the high content of portlandite and calcite, the number of which is more than 1.5 times higher than their content in the cement matrix (Table 1).

Thus, the above crystallohydrates are located in a dense environment of CSH-phase. Using X-ray microstructural sensing of different zones of black agglomerates, the presence of only AFt-phase was established, while in the section zone of black agglomerates and the gray matrix, a preferential content of AFm-phase was established. This can be explained by increased water demand of crystallizing AFt - phase, at which point in adjacent regions, containing sulfates and calcium, reactive water and the sulfate component is sufficient only for the AFm-phase formation. In the main volume of the black agglomerates there is a mixture of crystals of portlandite and ettringite, uniformly distributed in the CSH-phase. In this area there are ordinary ettringite crystals of in the form of needle-like prisms, and its morphological structure resembles the form of fused crystals (Figs 6 and 7).

Figure 6. Mixed crystal of hydration products (ettringite, portlandite, hydrosilicate) in the black aggregate volume: zoom x10000

Figure 7. Transition zone between the black agglomerate and gray matrix, zoom x10 000: ettringite, portlandite, silicate hydrate, calcite

Discussion

The number and density distribution of black agglomerates in the cement matrix volume increases in proportion to the pressure magnitude, developing in the process of expanding cement hardening in the complete expansion deformation constraint. The emergence of black agglomerates was identified in the expansion pressure exceeding 0.4 MPa, The process of their formation is probably associated with the reaction system equilibrium state disturbance at the isotropic expansion constrained deformation conditions and change of component chemical potential, involved in the development of hydration reactions. It has a significant impact on changing the crystallohydrate habit with preferential formation of microcrystalline forms, as well as high content of CSH-phase in the black agglomerate volume. In accordance with the Taylor's theoretical views [30], the morphology of the forming crystallohydrates is primarily determined not only by the ion ratio in the reaction medium, but the temperature and humidity and adiabatic conditions, under which the hydration processes are developing.

The distribution of individual black agglomerates by the volume of hardening cement stone may be a result of sulfoaluminate expander mixing with the base Portland cement while preparing a dry mixture and cement-water paste and during the introduction of water of mixing. In this case the calcium aluminates function as centers of crystallization, and the growth rate and the size of the producing crystallohydrates in the aquatic environment are determined by diffusion processes at the interface of the reaction phases.

Optical anomalies, such as established in the study of structure formation processes of expanding cements at the full expansion constrained deformation conditions, exist in cement silos and filters on the cement plants, when while their cleaning the layers of hardened cement stone with separate inclusions are removed, which are of the dark gravel form [31].

Conclusions

Experimentally determined anomalous phenomena during expanding cement hardening on the calcium sulfoaluminate basis at the expansion constrained deformation conditions are determined by the combination of two processes – structure strengthening due to hydration sulphoaluminates and silicates of calcium and the ettringite formation, thus the constraint of the deformation on the one hand increases the degree of self-stress, and on the other hand reduces the role of structure plasticity of cement stone newgrowths while hardening, which allows in wide ranges modifying the concrete properties on the expanding cement basis and ensuring the uniformity of the structure functional characteristics. Further theoretical and experimental studies of expanding cement hardening on the calcium sulfoaluminate basis at the expansion constrained deformation conditions opens up new possibilities for obtaining concrete products and constructions with the structure and properties, commensurate with ceramics, but without the use of cost demanding processes, only by chemical energy, released during the hydration of sulfoaluminate systems.

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References

1. Odum-Ewuakyea B., Attoh-Okineb N. Sealing system selection for jointed concrete pavements – A review. *Construction and Building Materials*. 2006. Vol. 20. No. 8. Pp. 591–602.
2. Akimov L.I., Ilenko N., Mizharev R., Cherkashin A., Vatin N.I., Chumadova L.I. Composite concrete modifier cm 02-10 and its impact on the strength characteristics of concrete. *MATEC Web of Conferences.* 2016. 01022.
3. Vella J.P., Vollum R. L., Jackson A. Flexural behaviour of headed bar connections between precast concrete panels *Construction and Building Materials*. 2017. Vol. 154. Pp. 236–250.
4. Syed A., Sonparote R.S. Analysis of Prestressed Precast Concrete Pavement. *Materials Today: Proceedings.* 2017. Vol. 4. No. 9. Pp. 9713–9717.
5. Vella J.P., Vollum R. L., Jackson A. Investigation of headed bar joints between precast concrete panels. *Engineering Structures.* 2017. Vol. 138. Pp. 351–366.
6. Barabanshchikov Yu.G., Belyaeva S.V., Arkhipov I.E., Antonova M.V., Shkolnikova A.A., Lebedeva K.S. Influence of superplasticizers on the concrete mix properties. *Magazine of Civil Engineering*. 2017. Vol. 74. No. 6. Pp. 140–146.

Литература

1. Odum-Ewuakyea B., Attoh-Okineb N. Sealing system selection for jointed concrete pavements – A review // Construction and Building Materials. 2006. Vol. 20. No. 8. Pp. 591–602.
2. Akimov L.I., Ilenko N., Mizharev R., Cherkashin A., Vatin N.I., Chumadova L.I. Composite concrete modifier cm 02-10 and its impact on the strength characteristics of concrete // MATEC Web of Conferences. 2016. 01022.
3. Vella J.P., Vollum R. L., Jackson A. Flexural behaviour of headed bar connections between precast concrete panels // Construction and Building Materials. 2017. Vol. 154. Pp. 236–250.
4. Syed A., Sonparote R.S. Analysis of Prestressed Precast Concrete Pavement // Materials Today: Proceedings. 2017. Vol. 4. Issue 9. Pp. 9713-9717
5. Vella J.P., Vollum R. L., Jackson A. Investigation of headed bar joints between precast concrete panels // Engineering Structures. 2017. Vol. 138. Pp. 351–366.
6. Барабанщиков Ю.Г., Беляева С.В., Архипов И.Е., Антонова М.В., Школьникова А.А., Лебедева К.С. Влияние суперпластификаторов на свойства бетонной смеси // Инженерно-строительный журнал. 2017. № 6(74). С. 140–146.

7. Brykov A.S., Vasil'ev A.S., Mokeev M.V. Hydration of portland cement in the presence of aluminum-containing setting accelerators. *Russian Journal of Applied Chemistry.* 2013. Vol. 86. No. 6. Pp. 793–801.
8. Lee H.X.D., Wong H.S., Buenfeld N.R. Self-sealing of cracks in concrete using superabsorbent polymers. *Cement and Concrete Research.* 2016. Vol. 79. Pp. 194–208.
9. Bastos da Costa E., Rodríguez E.D., Bernal S.A., Provis J.L., Gobbo L.A., Kirchheim A.P. Production and hydration of calcium sulfoaluminate-belite cements derived from aluminium anodising sludge. *Construction and Building Materials.* 2016. Vol. 122. Pp. 373–383.
10. Chaunsali P., Mondal P. Physico-chemical interaction between mineral admixtures and OPC–calcium sulfoaluminate (CSA) cements and its influence on early-age expansion. *Cement and Concrete Research.* 2016 Vol. 80. Pp. 10–20.
11. Zajac M., Skocek J., Bullerjahn F., Mohsen Ben Haha. Effect of retarders on the early hydration of calcium-sulpho-aluminate (CSA) type cements. *Cement and Concrete Research.* 2016. Vol. 84. Pp. 62–75.
12. Chen I.A., Hargis C.W., Juenger M.C.G. Understanding expansion in calcium sulfoaluminate–belite cements. *Cement and Concrete Research.* 2012. Vol. 42. No. 1. Pp. 51–60.
13. Trauchessec R., Mechling J.-M., Lecomte A., Roux A., Le Rolland B. Hydration of ordinary Portland cement and calcium sulfoaluminate cement blends. *Cement and Concrete Composites.* 2015. Vol. 56. Pp. 106–114.
14. Ioannou S., Reig L., Paine K., Quillin K. Properties of a ternary calcium sulfoaluminate–calcium sulfate–fly ash cement. *Cement and Concrete Research.* 2014. Vol. 56. Pp. 75–83.
15. Lorenzo Valenti G., Marroccoli M., Lucia Pace M., Telesca A. Discussion of the paper “Understanding expansion in calcium sulfoaluminate–belite cements” by I.A. Chen et al. *Cement and Concrete Research.* 2012. Vol. 42. No. 11. Pp. 1555–1559.
16. Hu C., Hou D., Li Z. Micro-mechanical properties of calcium sulfoaluminate cement and the correlation with microstructures. *Cement and Concrete Composites.* 2017. Vol. 80. Pp. 10–16.
17. Winnefeld F., Lothenbach B. Hydration of calcium sulfoaluminate cements — Experimental findings and thermodynamic modelling. *Cement and Concrete Research.* 2010. Vol. 40. No. 8. Pp. 1239–1247.
18. Martin L.H.J., Winnefeld F., Tschopp E., Müller C.J., Lothenbach B. Influence of fly ash on the hydration of calcium sulfoaluminate cement. *Cement and Concrete Research.* 2017. Vol. 95. Pp. 152–163.
19. García-Maté M., Santacruz I., De la Torre A.G., León-Reina L., Aranda M.A.G., Rheological and hydration characterization of calcium sulfoaluminate cement pastes. *Cement and Concrete Composites.* 2012. Vol. 34. No. 5. Pp. 684–691.
20. Martin L.H.J., Winnefeld F., Müller C.J., Lothenbach B. Contribution of limestone to the hydration of calcium sulfoaluminate cement. *Cement and Concrete Composites.* 2015. Vol. 62. Pp. 204–211.
21. Chartschenko I., Rudert, V., Wihler, H-D. Einflusse auf den Hydratationsprozess und die Eigenschaften von Quellzement Teil. *Zement-Kalk-Gips.* 1996. Vol. 49. Heft 8. Pp. 21–30.
22. Telesca A., Marroccoli M., Pace M.L.,Tomasulo M., Valenti G.L., Monteiro P.J.M. A hydration study of various calcium sulfoaluminate cements. *Cement and Concrete Composites.* 2014. Vol. 53. Pp. 224–232.
23. Charschenko I., Stark, J. Control of Structure formation in

7. Brykov A.S., Vasil'ev A.S., Mokeev M.V. Hydration of portland cement in the presence of aluminum-containing setting accelerators // Russian Journal of Applied Chemistry. 2013. Vol. 86. № 6. Pp. 793–801.
8. Lee H.X.D., Wong H.S., Buenfeld N.R. Self-sealing of cracks in concrete using superabsorbent polymers // Cement and Concrete Research. 2016. Vol. 79. Pp. 194-208
9. Bastos da Costa E., Rodríguez E.D., Bernal S.A., Provis J.L., Gobbo L.A., Kirchheim A.P. Production and hydration of calcium sulfoaluminate-belite cements derived from aluminium anodising sludge // Construction and Building Materials. 2016. Vol. 122. Pp. 373–383.
10. Chaunsali P., Mondal P. Physico-chemical interaction between mineral admixtures and OPC–calcium sulfoaluminate (CSA) cements and its influence on early-age expansion // Cement and Concrete Research. 2016. Vol. 80. Pp. 10–20.
11. Zajac M., Skocek J., Bullerjahn F., Mohsen Ben Haha Effect of retarders on the early hydration of calcium-sulpho-aluminate (CSA) type cements // Cement and Concrete Research. 2016. Vol. 84. Pp. 62–75.
12. Chen I.A., Hargis C.W., Juenger M.C.G. Understanding expansion in calcium sulfoaluminate–belite cements // Cement and Concrete Research. 2012. Vol. 42. № 1. Pp. 51–60.
13. Trauchessec R., Mechling J.-M., Lecomte A., Roux A., Le Rolland B. Hydration of ordinary Portland cement and calcium sulfoaluminate cement blends // Cement and Concrete Composites. 2015. Vol. 56. Pp. 106–114.
14. Ioannou S., Reig L., Paine K., Quillin K. Properties of a ternary calcium sulfoaluminate–calcium sulfate–fly ash cement // Cement and Concrete Research. 2014. Vol. 56. Pp. 75–83.
15. Lorenzo Valenti G., Marroccoli M., Lucia Pace M., Telesca A. Discussion of the paper “Understanding expansion in calcium sulfoaluminate–belite cements” by I.A. Chen et al. // Cement and Concrete Research. 2012. Vol.42. № 11. Pp. 1555–1559.
16. Hu C., Hou D., Li Z. Micro-mechanical properties of calcium sulfoaluminate cement and the correlation with microstructures // Cement and Concrete Composites. 2017. Vol. 80. Pp. 10–16.
17. Winnefeld F., Lothenbach B. Hydration of calcium sulfoaluminate cements — Experimental findings and thermodynamic modelling // Cement and Concrete Research. 2010. Vol. 40. № 8. Pp. 1239–1247.
18. Martin L.H.J., Winnefeld F., Tschopp E., Müller C.J., Lothenbach B. Influence of fly ash on the hydration of calcium sulfoaluminate cement // Cement and Concrete Research. 2017. Vol. 95. Pp. 152–163.
19. García-Maté M., Santacruz I., De la Torre A. G., León-Reina L., Aranda M. A.G., Rheological and hydration characterization of calcium sulfoaluminate cement pastes // Cement and Concrete Composites. 2012. Vol. 34. № 5. Pp. 684–691.
20. Martin L.H.J., Winnefeld F., Müller C.J., Lothenbach B. Contribution of limestone to the hydration of calcium sulfoaluminate cement // Cement and Concrete Composites. 2015. Vol. 62. Pp. 204–211.
21. Chartschenko I., Rudert, V., Wihler, H-D. Einflusse auf den Hydratationsprozess und die Eigenschaften von Quellzement Teil // Zement-Kalk-Gips. 1996. Vol. 49 . Heft 8. Pp. 21–30.
22. Telesca A., Marroccoli M., Pace M.L.,Tomasulo M., Valenti G.L., Monteiro P.J.M. A hydration study of various calcium sulfoaluminate cements // Cement and Concrete Composites. 2014. Vol. 53. Pp. 224–232.
23. Charschenko I., Stark J. Control of Structure formation in

expansive cements and in concretes containing these cements. *Second CANMET/ACI Intern. Symposium of Advances in Concrete Technology.* 1999. Pp. 62–71.

24. Fedorov N.F. Synthesis and properties of special cements. *VI International Congress on Cement Chemistry (book).* 1976. Vol. II.
25. Hargis C.W., Kirchheim A.P., Monteiro P.J.M., Gartner E.M. Early age hydration of calcium sulfoaluminate in the presence of gypsum and varying amounts of calcium hydroxide. *Cement and Concrete Research.* 2013. Vol. 48. Pp. 105–115.
26. Tang S.W., He Z. , Cai X.H., Cai R.J., Chen E. Volume and surface fractal dimensions of pore structure by NAD and LT-DSC in calcium sulfoaluminate cement pastes. *Construction and Building Materials.* 2017. Vol. 143. Pp. 395–418.
27. Zhang J., Guan X., Li H., Liu X. Performance and hydration study of ultra-fine sulfoaluminate cement-based double liquid grouting material. *Construction and Building Materials.* 2017. Vol. 132. Pp. 262–270.
28. Ghorpade P.A., Ha M-G., Choi W-H., Park J-Y. Combined calcium sulfoaluminate and ordinary portland cement/Fe(II) system for enhanced dechlorination of trichloroethylene. *Chemical Engineering Journal.* 2013. Vol. 231. Pp. 326–333.
29. Tang S.W., Zhu H.G., Li Z.J., Chen E., Shao H.Y. Hydration stage identification and phase transformation of calcium sulfoaluminate cement at early age. *Construction and Building Materials.* 2015. Vol. 75. Pp. 11–18.
30. Chartschenko I., Stark J. Steuerung des Strukturbildungsprozesses bei Quellzementen sowie Betonen auf deren Basis. *Wiss. Zeitschr. Hochsch. Arch. BauwesenWeimar.* 1993. Vol. 39/3. Pp. 163–171.
31. Royak S.M., Royak G.S. Spetsial'nyye tsementy [*Special cements].* Moscow: Stroyizdat. 1993. Pp. 364–363. (rus)

expansive cements and in concretes containing these cements // Second CANMET/ACI Intern. Symposium of Advances in Concrete Technology. 1999. Pp. 62–71.

24. Fedorov N.F. Synthesis and properties of special cements // VI International Congress on Cement Chemistry (book) 1976. Vol. II.
25. Hargis C.W., Kirchheim A.P., Monteiro P.J.M., Gartner E.M. Early age hydration of calcium sulfoaluminate in the presence of gypsum and varying amounts of calcium hydroxide // Cement and Concrete Research. 2013. Vol. 48. Pp. 105–115.
26. Tang S.W., He Z. , Cai X.H., Cai R.J., Chen E. Volume and surface fractal dimensions of pore structure by NAD and LT-DSC in calcium sulfoaluminate cement pastes // Construction and Building Materials. 2017. Vol.143. Pp. 395–418.
27. Zhang J., Guan X., Li H., Liu X. Performance and hydration study of ultra-fine sulfoaluminate cement-based double liquid grouting material // Construction and Building Materials. 2017. Vol. 132. Pp. 262–270.
28. Ghorpade P.A., Ha M-G., Choi W-H., Park J-Y. Combined calcium sulfoaluminate and ordinary portland cement/Fe(II) system for enhanced dechlorination of trichloroethylene // Chemical Engineering Journal. 2013. Vol. 231. Pp. 326–333.
29. Tang S.W., Zhu H.G., Li Z.J., Chen E., Shao H.Y. Hydration stage identification and phase transformation of calcium sulfoaluminate cement at early age // Construction and Building Materials. 2015. Vol. 75. Pp. 11–18.
30. Chartschenko I., Stark J. Steuerung des Strukturbildungsprozesses bei Quellzementen sowie Betonen auf deren Basis // Wiss. Zeitschr. Hochsch. Arch. BauwesenWeimar. 1993. Vol. 39/3. Pp. 163–171.
31. Рояк С.М., Рояк Г.С. Специальные цементы. М.: Стройиздат. 1993. 410 с.

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Kharchenko I.Ya., Pustovgar A.P., Pashkevich S.A., Eremin A.V., Ivanova I.S., Bazhenov Yu.M., Kharchenko A.I. Expanding cements hardening within the limited deformations conditions. *Magazine of Civil Engineering.* 2017. No. 7. Pp. 161–170. doi: 10.18720/MCE.75.16.

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The interaction between the kaolinite or bentonite clay and plasticizing surface-active agents

Взаимодействие каолиновой и бентонитовой глин с пластифицирующими поверхностно-активными веществами

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Key words: clay soil; kaolinite; montmorillonite; polycarboxylate superplasticizer; infrared spectroscopy; optimum moisture; maximum density; plastic index; ultimate compressive strength

Ключевые слова: глинистый грунт; каолинит; монтмориллонит; поликарбоксилатный суперпластификатор; ИК-спектроскопия; оптимальная влажность; максимальная плотность; число пластичности; предел прочности при сжатии

Abstract. The influence of a number of superplasticizers of various chemical bases on the physical and technical properties of clay soils has been studied. Model soils are considered as clay soils: kaolin clay with kaolinite mineral content up to 95 %, bentonite clay with montmorillonite content up to 70%. The greatest increase in the physical and technical properties of clay soils is achieved when they are modified by a polycarboxylate superplasticizer, which is due to adsorption of polymer molecules on clay minerals even with a negative charge of chips and basal planes. This is explained by the ability of side chains of the polycarboxylate ester, which has a similar composition with polyethylene glycols, to be adsorbed on the aluminosilicate layers of clay minerals by the formation of hydrogen bonds. The interaction of kaolinite and montmorillonite included in kaolin and bentonite clay with a polycarboxylate superplasticizer was studied by infrared spectroscopy. It was found that the additive under study belongs to a type of polycarboxylate superplasticizers modified with organo-silanes. Introduction to kaolin and bentonite clays leads to chemisorption with clay minerals and the formation of organomineral bonds, which leads to an increase in the strength characteristics of clays.

Аннотация. Проведены исследования влияния ряда суперпластификаторов различной химической основы на физико-технические свойства глинистых грунтов. В качестве глинистых грунтов рассмотрены модельные грунты: каолиновая глина с содержанием минерала каолинита до 95 % и бентонитовая глина с содержанием монтмориллонита до 70 %. Наибольшее повышение физико-технических свойств глинистых грунтов достигаются при модификации их поликарбоксилатным суперпластификатором, что обусловлено адсорбцией молекул полимера на глинистых минералах даже при отрицательном заряде сколов и базальных плоскостей. Это объясняется способностью боковых цепей эфира поликарбоксилата, имеющий аналогичный состав с полиэтиленгликолями, адсорбироваться на алюмосиликатных слоях глинистых минералов посредством образования водородных связей. Методом ИК-спектроскопии изучено взаимодействие каолинита и монтмориллонита, входящих в каолиновую и бентонитовую глины с поликарбоксилатным суперпластификатором. Установлено, что исследуемая добавка относится к типу поликарбоксилатных суперпластификаторов, модифицированных органо-силанами. Введение в каолиновую и бентонитовую глины, приводит к хемосорбции с глинистыми минералами и образованию органоминеральных связей, что приводит к повышению прочностных характеристик глин.

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Introduction

Clay soils have a wide range of mineral composition and consist of different types of clay and non-clay minerals [1, 2]. They are the product of weathering of feldspathic and other silicate rocks, composed mainly of montmorillonite, kaolinite and hydromica with an admixture of mainly relict minerals – quartz, mica, secondary calcite, opal and others. Kaolinite and montmorillonite are the most common clay minerals including in soils. Kaolinite – is a relatively stable mineral contained relatively in large quantities in many clay soils (Al_2O_3•$2SiO_2$•$2H_2O$). The montmorillonite ($2Al_2O_3$•$2SiO_2$•$4H_2O$) is the second most occurring clay mineral. It is formed under alkaline conditions in marine sediments and in the weathering crust. It belongs to a subclass of phyllosilicates, has the ability to a strong bulking [3]. Usually clay soils are subdivided on mineral composition into monomineral (kaolinite, montmorillonite, illite, etc.) and polymineral consisting of various clay and relict minerals [4, 5].

It is known that intromission of different surface-active agents into clay soils changes its structure and properties [6]. This is because of the presented in the soils finely dispersed and colloidal particles leads to significant increase the total surface of the partition between solid phase and liquid medium and hence to increase the free surface energy [7]. This allows to increase the physical and chemical activity of the particles, to increase the adsorption capacity of the surface, that allows the surface-active agents, concentrated on interfaces, to form the finest adsorbed layers, dramatically altering the molecular nature and properties of the soil surface [8].

Previous studies [9] have shown that the polycarboxylate superplasticizers compared with plasticizing additives, other have the greatest effect on increasing of physical and technical properties of soil-cement in accordance with Russian State Standard GOST 23558-94. It is also determined by a number of positive effects of polycarboxylate superplasticizers on the physical properties of clay soils with different mineral composition [10]. However, the effect of plasticizing surface-active agents with various chemical bases on the physical and strength properties of clay soils has not been sufficiently investigated.

In this way, the aim of the work was to determine the most effective plasticizing surface-active agents for kaolinite and bentonite clay on the bases results of experimental studies.

To achieve this aim, the following tasks are formulated:

1) to study the influence the plasticizing surface-active agents with various chemical bases on properties of kaolinite and bentonite clays by methods of laboratory studies of the construction and technical properties of soils;

2) to investigate the interactions between considering clays with the polycarboxylate superplasticizer by the infrared spectroscopy.

Materials and methods of research

The following clay soils were used for the research:

1) kaolinite clay (KC) with the plasticity index 18.07 and content of sandy particles 6.02 %, pH of aqueous extract 8.3, content of mineral kaolinite up to 95 %;

2) bentonite clay (BC) with the plasticity index 22.32 and content of sandy particles 8.00 %, pH of the aqueous extract 8.3, the mineral content of montmorillonite up to 70 %.

The following additives as plasticizing surface-active agents are used.

1) Sulfonated melamineformaldehyde compound (SMF) – superplasticizer Melment F10, which is a white powder. It consists of sulfonated powdered polycondensated products based on melamine, obtained by spray drying. Producer BASF Constraction Polymers, Germany.

2) Sulfonated naphthaleneformaldehyde compound (SNF) – superplasticizer SP-1 (TU 6-36-0204229-625-90 with changes 1, 2), which is a brown powder. It consists of a mixture of oligomers and polymers of different relative molecular weight obtained by condensation of naphthalene sulfonic acids with formaldehyde and technical lignosulfonates and neutralized with sodium hydroxide. The main component (the active substance) is the unreacted salt of β-naphthalenesulfonic acid and sodium sulfate. Producer company "Polyplast", Russia.

3) Lignosulfonate technical (LST) – plasticizer LST (TU 2455-028-00279580-2004), which is a powdery powder from light yellow to brown color. It is a natural water-soluble sulfonic derivative of lignin, formed during the sulfite process of wood delignification. Producer company "Solikamskbumprom", Russia.

4) Polycarboxylate (PCE) – polycarboxylate superplasticizer Pantarhit PC 160 Plv is a light gray powder obtained by spray drying from a polymer solution. The active substance is polyacrylic acid. Producer is BASF Constraction Polymers, Germany. Pantarhit PC 160 Plv was selected in comparison and research as the most effective polycarboxylate superplasticizer based on the results [9].

Dosages of plasticizing additives are 0.05 %, 0.5 %, 1 %, 2 % of the clay soil weight.

Building and technical indicators are accepted following: physical properties – the plasticity index, the optimum moisture content and maximum density, the strength properties – the ultimate compressive strength. The plasticity index (I_p) is calculated in accordance with Russian State Standard GOST 5180-2015 as the difference between the creep stress (W_c) and the rolling-out limit (W_r). Optimum moisture content (W_{opt}) and maximum density (ρ_{max}) are determined in accordance with Russian State Standard GOST 22733-2016 on the standard compaction measuring device of the SoyuzdorNII. To determine the ultimate compressive strength (R_{str}), samples were prepared with the maximum density, which were stored for 7 days under air conditions, and then dried to constant weight in a drying oven at a temperature of 105 °C and tested on a press.

The registration of infra-red spectrum samples was made on IR Fourier-spectrophotometer by Perkin-Elmer firm, the Spectrum 65 model, using a Miracle ATR (ZnSe crystal) attachment in the 4000-650 cm^{-1} area, usually at 20 scans. The background spectrum was recorded and subtracted automatically. The powder, after grinding clay soil and additives, was pressed against the crystal by a special clamp, which is part of the set-top box. After registration, automatic correction and spectrum conservation were automatically performed.

Before registration of the infra-red spectrum, the clay soils were dried at a temperature of 105 °C to a constant mass. Samples for the study were prepared as follows. The surface-active agents were previously diluted in a ceramic bowl with distilled water. Additives were added in amount of 20 % of the clay soil weight. Next, a pre-dried clay soil was added for the paste formation. In this case, the clay soil paste was in the state of creep stress and stored for 7 days in normal-moisture conditions. After that, the modified clay soil was dried, as well as unmodified. The dried clay soil samples were crushed in a vibratory mill for 5 minutes to particles of micron size.

Results and Discussion

Figures 1 and 2 show the research results of the influence of surface-active agents on the construction and technical properties of kaolinite and bentonite clays. It was established that the parameters of standard compaction, creep stress and rolling-out limit, the plasticity index and ultimate compressive strength are changed during intromission of plasticizing surface-active agents into clay composition.

Introduction the SMF from 0.05 % to 2 % of the clay mass weight decreased the optimum moisture content for KC on 0.11–8.22 %, for BC on 0.08–5.47 %. The maximum density increased in KC on 4.73 %, in BC on 3.29 %. The creep stress and the rolling-out limit decreased in KC on 0.26-9.52 % and 0.12-5.43 %, in BC on 2.13–4.33 % and respectively 0.08–4.33 %. The plasticity index decreased for KC on 0.50–16.93 %, for BC on 0.05–6.10 %. The ultimate compressive strength was increased in KC on 2.46–24.18 %, and in BC on 1.38–56.55 %.

Adding the SNF from 0.05 % to 2 % of the clay mass weight, the optimum moisture decreased for KC on 0.11–9.07 %, for BC on 0.11–5.66 %. The maximum density increased in KC on 5.41 %, in BC on 3.95 %. The creep stress and the rolling-out limit decreased in KC on 0.75-10.72 % and 0.31–5.49 %, in BC on 2.17–7.29 % and respectively on 0.11–4.64 %. The plasticity index decreased for KC on 1.55–20.20 %, for BC on 0.09–6.19 %. The compressive strength was increased in KC on 4.10–30.33 %, in BC it is 1.38–58.62 %.

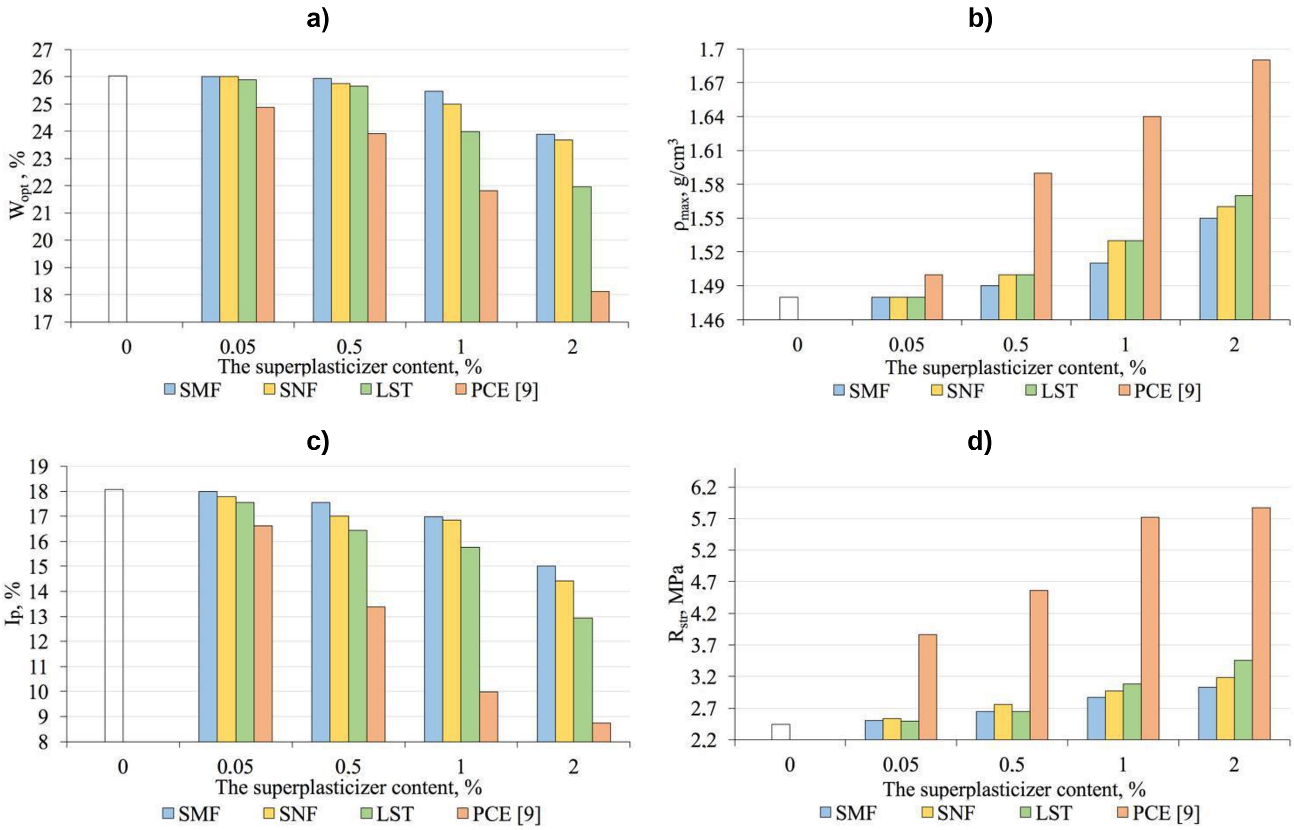

Figure 1. Dependences physical and strength properties of kaolinite clay of the dosage of plasticizing surface-agents: a) the optimum moisture; b) the maximum density; c) the plastic index; d) the ultimate compressive strength

Introduction the LST from 0.05 % to 2 % of the clay mass weight led to a decrease of the optimum moisture content of KC on 0.54–15.67 %, BC on 0.23–6.47 %. The maximum density is increased in KC on 6.08%, in BC on 5.26 %. The creep stress and the rolling-out limit decreased in KC on 1.24–13.89 % and 2.82–28.39 %, in BC on 2.21–7.52 % and 0.19–4.67 %, respectively. The plasticity index decreased for KC on 2.82–28.39 %, for BC on 0.09–6.66 %. The compressive strength limit was increased in KC on 2.05–41.80 %, in BC it increased on 1.38–74.48%.

The use of PCE, that was studied in [9], in an amount of 0.05 % to 2 % of the clay mass weight ensured a reduction in the optimum moisture content of the KC on 4.46–30.35 %, BC on 1.07–18.36 %. The maximum density increased in KC on 1.35–14.19 %, in BC on 1.32–11.84 %. The creep stress and the rolling-out limit decreased in the KC on 3.76–23.25 % and 1.40–51.63, in BC on 3.13–21.97 % and 0.46–19.54 %, respectively. The plasticity index decreased for KC on 8.02-51.63 %, for BC on 1.88–35.37 %. The compressive strength limit increased in KC on 58.20–140.57 %, in BC on 2.76–115.86 %.

According to the data presented in Figures 1 and 2, it can be seen that among the studied plasticizers of various chemical bases the most effective plasticizing surface-active agent was polycarboxylate superplasticizer Pantarhit PC 160 Plv (PCE). It should be noted that in [9] other polycarboxylate superplasticizers (Odolit-K, Giperlit, Melflux 2641 F), as well as Pantarhit PC 160 Plv, were more effective than SMF, SNF and LST.

PCE can be adsorbed on clay minerals even with a negative charge of chips and basal planes. This is due to the ability of the side chains of PCE, that have a similar composition with polyethylene glycols, to be adsorbed on the aluminosilicate layers of clay minerals through hydrogen bonds [11–17], which probably explains their high efficiency.

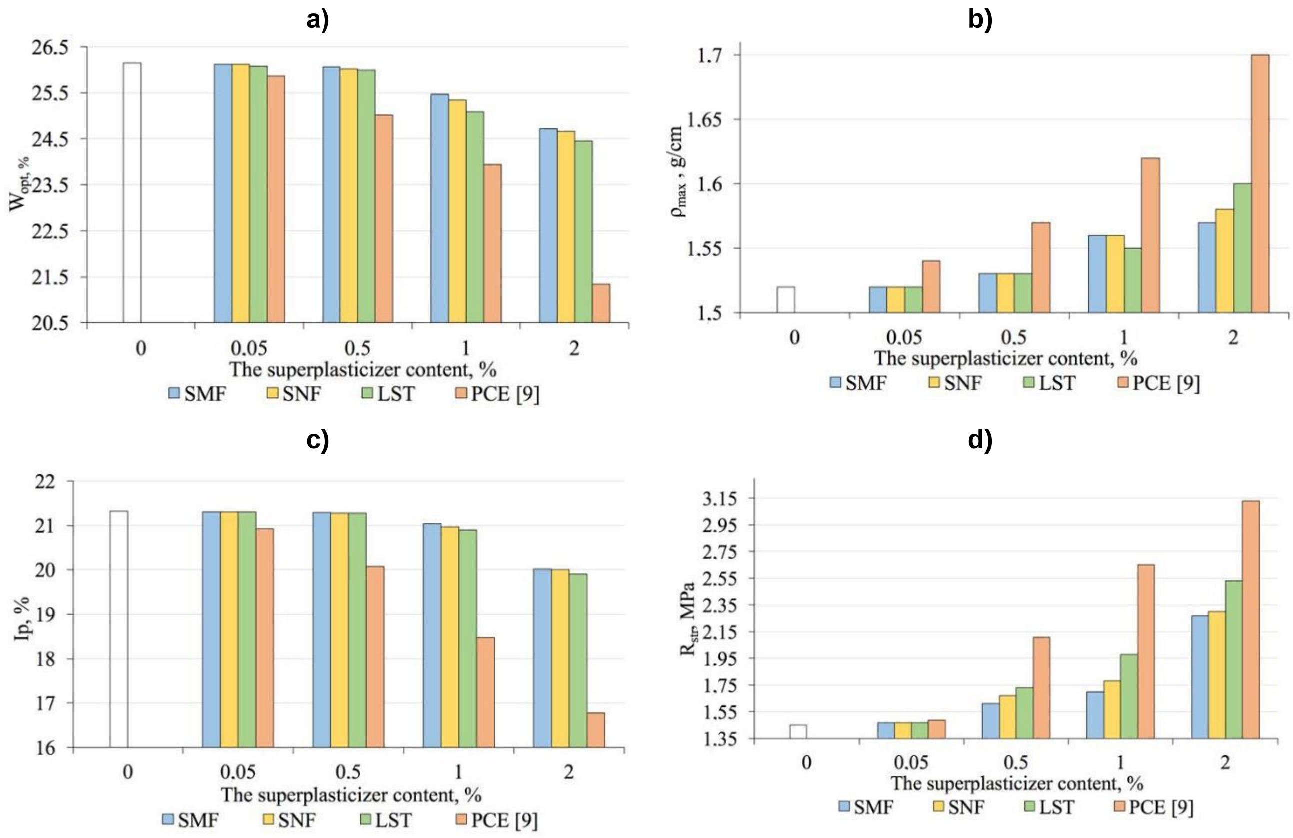

Figure 2. Dependences physical and strength properties of bentonite clay of the dosage of plasticizing surface-agents: a) the optimum moisture; b) the maximum density; c) the plastic index; d) the ultimate compressive strength

According to E. Tombacz, M. Szckcres, J.L. Suter, P.V. Coveney, L. Lei, J. Plank [18–20], injecting the PCE in the clay soil increases the basal reflection thickness of montmorillonite from 1.23 to 1.72–1.77 nm, but in kaolinite and muscovite practically does not change. Montmorillonite sorbs about 230 mg/g of PCE, and kaolinite and muscovite sorb about 20 mg/g, which probably explains the more effective effect of PCE on KC, in comparison with BC.

LST also showed greater efficiency in comparison with SMF and SNF. Perhaps this is due to the fact that the molecule of LST is spheroidal and consists of disc-shaped aggregates up to 200 nm in size, where hydrated groups are localized in the internal structure of associates [21–24]. Such molecules configuration limits the water mobility, but at the same time, its lyophobicity increases. Such a structure, in spite of the fact that the mechanism of interaction with particles surface is mainly based on the forces of electrostatic repulsion, can lead to a significant decrease in the interaction between individual clay particles with adsorbed on their surface surface-active agents.

As is known, a continuous spatial grid with a coagulation structure acquiring plastic-viscous properties is formed in the clay-water system. The increase of the thickness of the water shells has a plasticizing effect on the clay, while reducing the strength of the structure and its ability to permanent elastic deformation [8]. Adsorption of surface-active agents on clay minerals in the clay-water system allows to reduce the surface energy at the interface, which contributes to the increase on density due to the decrease the water demand (optimum moisture), to the decrease in creep stress and the rolling-out limit [8, 25], and ultimately to the increase of the strength of the soil.

PCE showed the greatest efficiency among the considered plasticizers, by allowing to reduce the optimum moisture and plastic index, to increase the maximum density and compressive strength limit of kaolinite and bentonite clay. The positive influence of PCE on both the physical and mechanical characteristics of the cement stone [26] and on the physical and strength properties of clay soils explains the strength and frost resistance increase of soil cements modified with such additives.

At the next stage, the interaction of kaolinite and bentonite clays with a polycarboxylate superplasticizer was studied by methods of infrared spectroscopy.

It can be seen from Figure 3 that each of the spectra is characterized by the presence of intense absorption bands in the area of 1000–1200 cm-1. These peaks are due to valence vibrations of Si-O-Si bonds [27]. In our case, they include a doublet 1024-999 cm-1 in the spectrum of KC and an intense peak of 1097 cm-1 with a shoulder of 1060 cm-1 in the PCE spectrum. Probably, the investigated additive PCE (Pantarhit PC 160 Plv) belongs to the group of polycarboxylate superplasticizers modified with organosilanes [28-30], as shown in Figure 4.

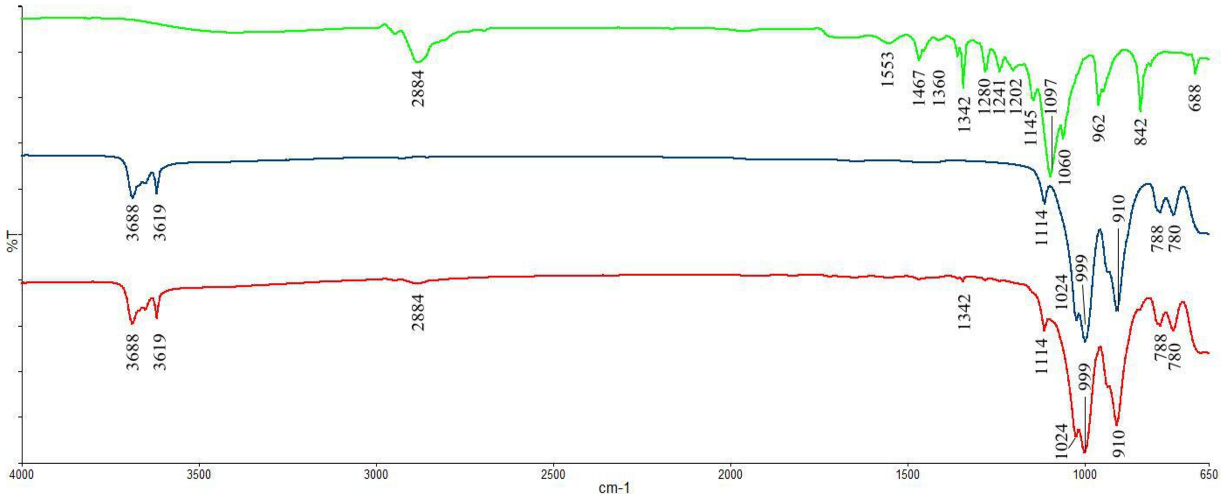

Figure 3. Infrared spectroscopy in the study of kaolinite clay and polycarboxylate: —— – PCE, —— – KC, —— – KC + PCE system

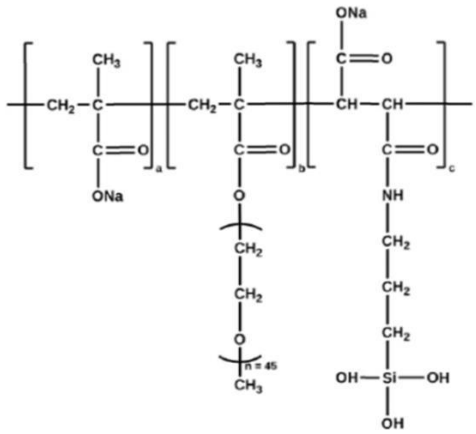

Figure 4. The chemical formula of a polycarboxylate superplasticizer modified by organosilanes, according to J. Plank, E. Sakai, C.W. Miao, C. Yu, J.X. Hong

Peaks 3688, 3673, 3619 cm^{-1} on the KC and KC + PCE curves correspond to the stretching vibrations of the -OH gibbsite layer of the kaolinite mineral in the KC. With the injection of the PCE additive leads to intense band of 1097 cm^{-1} disappears in the spectrum of the KC + PCE system, although the remaining peaks corresponding to the PCE spectrum are conserved. In particular, the bands 2884 cm^{-1} (stretching vibrations of CH_2 and CH_3 groups), 1467 cm^{-1} and 1342 cm^{-1} (deformation vibrations of the same groups) are retained. This indicates that the chemical interaction between kaolinite and PCE proceeds, and this interaction leads to the conversion of the Si-O-Si bonds of the PCE additive to other structures, apparently chemically related to kaolinite. The aliphatic portion of the PCE containing the CH_2 and CH_3 groups does not appear to undergo any changes.

Similar results were obtained with adding PCE into BC.

Oscillations of Si-O-Si bonds in the BC spectrum correspond to peaks of 1002, 912, 873 cm^{-1}. When PCE was injected into BC, results similar to KC have been obtained. The corresponding infrared spectroscopy is shown in Figure 5. Intensive spectra of 1097 and 1060 cm^{-1}, corresponding to stretching vibrations of Si-O-Si bonds in PCE, practically disappear in the spectra of the modified BC. The presence of aliphatic CH_2 and CH_3 groups will remain. In this case, the modifiers chemically bind to the mineral by Si-O bonds, which leads to their conversion. Interactions on hydroxyl groups as a modifier and a mineral are not observed.

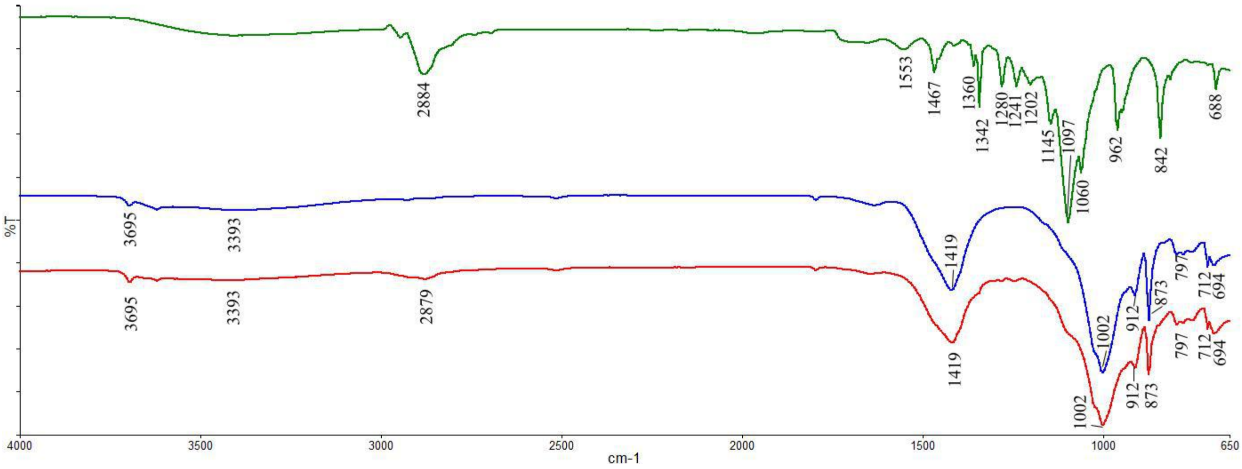

Figure 5. Infrared spectroscopy in the study of bentonite clay BC and polycarboxylate PCE: —— – PCE, —— – BC, —— – BC + PCE system

It should be noted that according to W. Fan, F. Stoffelbach, J. Rieger, L. Regnaud, A. Vichot, B. Bresson, N. Lequeux, J. Witt, J. Plank [28, 30] organosilane modified polycarboxylates with -Si-OH bonds adding into cement systems can form chemical bonds with cement neoplasms of the type C-S-H-PCE. That is, these polycarboxylates in the first stage are adsorbed on the surface of the flocculum of cement, and then their chemisorption occurs on the surface of the hydrate particles. Also in the works of E. Sakai, D. Atarashi, M. Daimon, S. Ng, J. Plank [31, 32] noted the possibility of the formation of such bonds in clay minerals.

Conclusions

The influence of plasticizing surface-active agents with various chemical bases on the creep stress and the rolling-out limit, optimum moisture and maximum density, compressive strength of kaolinite and bentonite clay is considered. It was determined that the most effective among the studied surfactants was polycarboxylate superplasticizer. Polycarboxylate showed maximum efficiency in kaolinite clay.

The interaction of kaolinite and montmorillonite included in kaolinite and bentonite clay with a polycarboxylate superplasticizer (Pantarhit PC 160 Plv additive) was studied by infrared spectroscopy.

Characteristic absorption bands corresponding to vibrations of Si-O-Si functional bonds are established.

It has been shown that the introduction of polycarboxylates modified with organosilanes into kaolinite and bentonite clay leads to the disappearance of absorption bands corresponding to Si-O bonds in the surface-active agents. This fact, apparently, indicates the chemical interaction of the modifiers used with clay minerals, leading to the conversion of Si-O groups. There are no interactions (chemical or physical, like hydrogen bonding) relatively to OH groups of clay minerals and modifiers.

References

1. Bell F.G. Lime stabilization of clay minerals and soils. *Engineering Geology.* 1996. No. 42. Pp. 223–237.
2. Grim R.Ye. *Mineralogiya glin* [Mineralogy of clays]. M.: Izdatelstvo inostrannoy literatury, 1959. 452 p. (rus)
3. Minke G. *Glinobeton i yego primeneniye* [Clay concrete and its application]. Kaliningrad: FGUIPP «Yantarnyy skaz», 2004. 232 p. (rus)
4. Shelikhov N.S., Rakhimov R.Z. *Stroitelnyye materialy iz mestnogo karbonatnogo syrya* [Building materials from local carbonate raw materials]. Kazan: Izdatelstvo KGASU, 2015. 324 p. (rus)
5. Gayfullin A.R., Rakhimov R.Z., Rakhimova N.R. The influence of clay additives in portland cement on the compressive strength of the cement stone. *Magazine of*

Литература

1. Bell F.G. Lime stabilization of clay minerals and soils // Engineering Geology. 1996. № 42. Pp. 223–237.
2. Грим Р.Е. Минералогия глин. М.: Издательство иностранной литературы, 1959. 452 с.
3. Минке Г. Глинобетон и его применение. Калининград: ФГУИПП «Янтарный сказ», 2004. 232 с.
4. Шелихов Н.С., Рахимов Р.З. Строительные материалы из местного карбонатного сырья. Казань: Издательство КГАСУ, 2015. 324 с.
5. Гайфуллин А.Р., Рахимов Р.З., Рахимова Н.Р. Влияние добавок глинитов в портландцемент на прочность при сжатии цементного камня // Инженерно-строительный журнал. 2015. № 7. С. 66–73.
6. Ребиндер П.А. Физико-химическая механика, как основа

Civil Engineering. 2015. No. 7. Pp. 66–73. (rus)
6. Rebinder P.A. Fiziko-khimicheskaya mekhanika, kak osnova zakrepleniya gruntov v dorozhnom stroitelstve i proizvodstve stroitelnykh materialov na osnove gruntov. [Physico-chemical mechanics, as the basis for fixing soils in road construction and production of building materials based on soils]. *Trudy soveshchaniya po teoreticheskim osnovam tekhnicheskoy melioratsii gruntov* [Proceedings of the meeting on the theoretical foundations of technical soil reclamation]. M.: Izd-vo MGU, 1961. Pp. 4–12. (rus)
7. Mogilevich V.M., Shcherbakova R.P., Tyumentseva O.V. *Dorozhnyye odezhdy iz tsementogrunta* [Pavement from soil-cement]. M.: Transport, 1972. 215 p. (rus)
8. Rebinder P.A. *Poverkhnostnyye yavleniya v dispersnykh sistemakh. Kolloidnaya khimiya. Izbrannyye trudy* [Surface phenomena in disperse systems. Colloid chemistry. Selected works.]. M.: Nauka, 1978. 368 p. (rus)
9. Bulanov P.E., Vdovin Ye.A., Mavliyev L.F. Vliyaniye plastifitsiruyushchikh dobavok na fiziko-mekhanicheskiye svoystva tsementogrunta dorozhnogo naznacheniya [Influence of plasticizers on physical and mechanical properties of soil-cement for road purpose]. *Izvestiya KGASU.* 2015. No. 1(31). Pp. 160–164. (rus)
10. Bulanov P.E., Mavliyev L.F., Vdovin E.A., Zakharov V.V., Gimazov A.R. Issledovaniye vliyaniya polikarboksilatnykh superplastifikatorov na fizicheskiye i prochnostnyye svoystva glinistykh gruntov razlichnogo mineralnogo sostava [The investigation of the effect of polycarboxylate superplasticizer on the physical and strength properties of clay soils with different mineral composition]. *Izvestiya KazGASU.* 2016. No. 4(38). Pp. 469–476. (rus)
11. Jeknavorian A.A., Jardine L., Ou C.C., Koyata H., Folliard K.J. Interaction of Superplasticizers with Clay-Bearing Aggregates. *Special Publication ACI/7th CANMET Conference on Superplasticizers and Oher Chemical Admixtures.* Berlin (Germany), 2003. Pp. 1293–1316.
12. Sakai E., Atarashi D., Daimon M. Interaction Between Superplasticizers and Clay Minerals. *Special Publication ACI/6th CANMET Conference on Superplasticizers and Other Chemical Admixtures.* Xi'an (China), 2006. Pp. 1560–1566.
13. Atarashi D., Sakai E., Obinata R., Daimon M. Interactions between superplasticizers and clay minerals. *Cement Science and Concrete Technology.* 2004. No. 58. Pp. 387–392.
14. Atarashi D., Sakai E., Obinata R., Daimon M. Influence of clay minerals on fluidity of CaCO3 suspension containing comb-type polymer. *Cement Science and Concrete Technology.* 2003. No. 57. Pp. 386–391.
15. Liu S., Mo X., Zhang C., Sun D., Mu C. Swelling inhibition by polyglycols in montmorillonite dispersions. *Journal of Dispersion Science and Technology.* 2004. No. 25. Pp. 63–66.
16. Amarasinghe P.M., Katti K.S., Katti D.R. Nature of Organic Fluid-Montmorillonitu Interactions: an FTIR spectroscopic study. *Colloid and Interface Science.* 2009. No. 337. Pp. 97–105.
17. Burchill S., Hall P.L., Harrison R., Hayes M.H.B., Langford J.I., Livingston W.R., Smedley R.J., Ross D.K., Tuck J.J. Smectite-polymer interactions in aqueous systems. *Clay Minerals.* 1983. No. 18. Pp. 373–397.
18. 1Tombacz E., Szckcres M. Colloidal behavior of aqueous montmorillonite suspensions: the specific role of pH in the presence of indifferent electrolytes. *Applied Clay Science.* 2004. No. 27. Pp. 75–94.
19. Suter J.L., Coveney P.V. Computer simulation study of the materials properties of intercalated and exfoliated poly(ethylene) glycol clay nanocomposites. *Soft Matter.* 2009. No. 5(11). Pp. 2239–2251.
20. Lei L., Plank J. A Study on the impact of different clay minerals on the dispersing force of conventional and modified vinyl ether based polycarboxylate superplasticizers. *Cement and Concrete Research.* 2014. No. 6. Pp. 1–10.

закрепления грунтов в дорожном строительстве и производстве строительных материалов на основе грунтов // Труды совещания по теоретическим основам технической мелиорации грунтов. М.: Изд-во МГУ, 1961. С. 4–12.
7. Могилевич В.М., Щербакова Р.П., Тюменцева О.В. Дорожные одежды из цементогрунта. М.: Транспорт, 1972. 215 с.
8. Ребиндер П.А. Поверхностные явления в дисперсных системах. Коллоидная химия. Избранные труды. М.: Наука, 1978. 368 с.
9. Буланов П.Е., Вдовин Е.А., Мавлиев Л.Ф. Влияние пластифицирующих добавок на физико-механические свойства цементогрунта дорожного назначения // Известия КГАСУ. 2015. № 1(31). С. 160–164.
10. Буланов П.Е., Мавлиев Л.Ф., Вдовин Е.А., Захаров В.В., Гимазов А.Р. Исследование влияния поликарбоксилатных суперпластификаторов на физические и прочностные свойства глинистых грунтов различного минерального состава // Известия КазГАСУ. 2016. № 4(38). С. 469–476.
11. Jeknavorian A.A., Jardine L., Ou C.C., Koyata H., Folliard K.J. Interaction of superplasticizers with clay-bearing aggregates // Special Publication ACI/7th CANMET Conference on Superplasticizers and Oher Chemical Admixtures. Berlin (Germany), 2003. Pp. 1293–1316.
12. Sakai E., Atarashi D., Daimon M. Interaction between superplasticizers and clay minerals // Special Publication ACI/6th CANMET Conference on Superplasticizers and Other Chemical Admixtures. Xi'an (China), 2006. Pp. 1560–1566.
13. Atarashi D., Sakai E., Obinata R., Daimon M. Interactions between superplasticizers and clay minerals // Cement Science and Concrete Technology. 2004. № 58. Pp. 387-392.
14. Atarashi D., Sakai E., Obinata R., Daimon M. Influence of Clay Minerals on Fluidity of CaCO3 Suspension Containing Comb-Type Polymer // Cement Science and Concrete Technology. 2003. № 57. Pp. 386–391.
15. Liu S., Mo X., Zhang C., Sun D., Mu C. Swelling inhibition by polyglycols in montmorillonite dispersions // Journal of Dispersion Science and Technology. 2004. № 25. Pp. 63–66.
16. Amarasinghe P.M., Katti K.S., Katti D.R. Nature of organic fluid-montmorillonity interactions: an FTIR spectroscopic study // Colloid and Interface Science. 2009. № 337. Pp. 97–105.
17. Burchill S., Hall P.L., Harrison R., Hayes M.H.B., Langford J.I., Livingston W.R., Smedley R.J., Ross D.K., Tuck J.J. Smectite-polymer interactions in aqueous systems // Clay Minerals. 1983. № 18. Pp. 373–397.
18. Tombacz E., Szckcres M. Colloidal behavior of aqueous montmorillonite suspensions: the specific role of ph in the presence of indifferent electrolytes // Applied Clay Science. 2004. № 27. Pp. 75–94.
19. Suter J.L., Coveney P.V. Computer simulation study of the materials properties of intercalated and exfoliated poly(ethylene) glycol clay nanocomposites // Soft Matter. 2009. № 5(11). Pp. 2239–2251.
20. Lei L., Plank J. A Study on the impact of different clay minerals on the dispersing force of conventional and modified vinyl ether based polycarboxylate superplasticizers // Cement and Concrete Research. 2014. № 6. Pp. 1–10.
21. Батраков В.Г. Модифицированные бетоны. Теория и практика. М.: Высшая школа, 1998. 768 с.
22. Юхневский П.И. О механизме пластификации цементных композиций добавками // Строительная наука и техника: научно-технический журнал. 2010. № 1(2). С. 64–69.
23. Болатбаев К.Н., Луговицкая Т.Н., Колосов А.В. Идентификация и физико-химические свойства лигносульфонатов в растворах // Ползуновский вестник. 2009. № 3. С. 308–312.

Bulanov P.E., Mavliev L.F., Vdovin E.A., Yagund E.M. The interaction between the kaolinite or bentonite clay and plasticizing surface-active agents. *Magazine of Civil Engineering*. 2017. No. 7. Pp. 171–179. doi: 10.18720/MCE.75.17.

21. Batrakov, V.G. *Modifitsirovannyye betony. Teoriya i praktika* [Modified concrete. Theory and practice]. M.: Vysshaya shkola, 1998. 768 p. (rus)
22. Yukhnevskiy, P.I. O mekhanizme plastifikatsii tsementnykh kompozitsiy dobavkami [On the mechanism of plasticization of cement compositions with additives]. *Stroitelnaya nauka i tekhnika: nauchno-tekhnicheskiy zhurnal.* 2010. № 1(2). Pp. 64–69. (rus)
23. Bolatbayev K.N., Lugovitskaya T.N., Kolosov A.B. Identifikatsiya i fiziko-khimicheskiye svoystva lignosulfonatov v rastvorakh [Identification and physico-chemical properties of lignosulfonates in solutions]. *Polzunovskiy vestnik.* 2009. № 3. Pp. 308–312. (rus)
24. Plank J., Sachgenhauser B. Experimental determination of the effective anionic charge density of polycarboxylate superplasticizers in cement pore solution. *Cement and Concrete Research.* 2009. No. 39(1). Pp. 1–5.
25. Ovcharenko F.D. *Gidrofilnost glin i glinistykh mineralov* [Hydrophilicity of clays and clay minerals]. M.: Izdatelstvo Akademii nauk USSR, 1961. 292 p. (rus)
26. Kalashnikov V.I., Volodin V.M., Moroz M.N., Yerofeyeva I.V., Petukhov A.V. Super- i giperplastifikatory. Mikrokremnezemy. Betony novogo pokoleniya s nizkim udelnym raskhodom tsementa na yedinitsu prochnosti [Super- and hyperplasticizers. Microsilica. Concrete of new generation with low specific consumption of cement per unit of strength]. *Molodoy uchenyy.* 2014. No. 19. Pp. 207–210. (rus)
27. Kuptsov A.Kh., Zhizhin G.N.. *Furye-KR i Furye-IK spektry polimerov.* M.: Fizmatlit, 2001. 308 s. (rus)
28. Plank J., Sakai E., Miao C.W., Yu C., Hong J.X. Chemical admixtures – chemistry, applications and their impact on concrete microstructure and durability. *Cement and Concrete Research.* 2015. No. 78. Pp. 81–99.
29. Fan W., Stoffelbach F., Rieger J., Regnaud L., Vichot A., Bresson B., Lequeux N. A new class of organosilane-modified polycarboxylate superplasticizers with low sulfate sensitivity. *Cement and Concrete Research.* 2012. No. 42. Pp. 166–172.
30. Witt J., Plank J. A Novel Type of PCE Possessing Silyl Functionalities. *Special Publication ACI/10th CANMET Conference on Superplasticizers and Other Chemical Admixtures.* Prague, 2012. Pp. 57–70.
31. Sakai E., Atarashi D., Daimon M. Interaction Between superplasticizers and Clay Minerals. *Special Publication ACI/6th CANMET Conference on Superplasticizers and Other Chemical Admixtures.* Xi'an (China), 2006. Pp. 1560–1566.
32. Ng S., Plank J. Study on the interaction of Na-montmorillonite clays with polycarboxylate based superplastirizers. *Special publication ACI/10th CANMET conference on superplasticizers and other chemical admixtures.* Prague, 2012. P.407-421

24. Plank J., Sachgenhauser B. Experimental determination of the effective anionic charge density of polycarboxylate superplasticizers in cement pore solution // Cement and Concrete Research. 2009. № 39(1). Pp. 1–5.
25. Овчаренко Ф.Д. Гидрофильность глин и глинистых минералов. М.: Издательство Академии наук УССР, 1961. 292 с.
26. Калашников В.И., Володин В.М., Мороз М.Н., Ерофеева И.В., Петухов А.В. Супер- и гиперпластификаторы. Микрокремнеземы. Бетоны нового поколения с низким удельным расходом цемента на единицу прочности // Молодой ученый. 2014. № 19. С. 207–210.
27. Купцов А.Х., Жижин Г.Н.. Фурье-КР и Фурье-ИК спектры полимеров. М.: Физматлит, 2001. 308 с.
28. Plank J., Sakai E., Miao C.W., Yu C., Hong J.X. Chemical admixtures – chemistry, applications and their impact on concrete microstructure and durability // Cement and Concrete Research. 2015. № 78. Pp. 81–99.
29. Fan W., Stoffelbach F., Rieger J., Regnaud L., Vichot A., Bresson B., Lequeux N. A new class of organosilane-modified polycarboxylate superplasticizers with low sulfate sensitivity // Cement and Concrete Research. 2012. № 42. Pp. 166–172.
30. Witt J., Plank J. A Novel Type of PCE Possessing Silyl Functionalities // Special Publication ACI/10th CANMET Conference on Superplasticizers and Other Chemical Admixtures. Prague, 2012. Pp. 57-70.
31. Sakai E., Atarashi D., Daimon M. Interaction between superplasticizers and clay minerals // Special Publication ACI/6th CANMET Conference on Superplasticizers and Other Chemical Admixtures. Xi'an (China), 2006. Pp. 1560–1566.
32. Ng S., Plank J. Study on the interaction of Na-montmorillonite clays with polycarboxylate based superplastirizers // Special publication ACI/10th CANMET conference on superplasticizers and other chemical admixtures. Prague, 2012. Pp. 407–421.

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Инженерно-строительный институт
Центр дополнительных профессиональных программ

ПОЛИТЕХ
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Приглашает специалистов проектных и строительных организаций, не имеющих базового профильного высшего образования на курсы профессиональной переподготовки (от 500 часов) по направлению «Строительство» по программам:

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***
- ***Управление инвестиционно-строительными проектами. Выполнение функций технического заказчика***

П-02 «Экономика и управление в строительстве»

Программа включает учебные разделы:

- ***Основы строительного дела***
- ***Инженерное оборудование зданий и сооружений***
- ***Технология и контроль качества строительства***
- ***Управление инвестиционно-строительными проектами. Выполнение функций технического заказчика и генерального подрядчика***
- ***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, дающий право на ведение профессиональной деятельности