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e-ISSN 2789-7338

Technobius

A peer-reviewed open-access journal

Technobius, LLP *Volume 3, No. 2, 2023*

Technobius

Volume 3, No. 2, 2023

A peer-reviewed open-access journal registered by the Ministry of Information and Social Development of the Republic of Kazakhstan, Certificate № KZ00VPY00039799 dated 7.09.2021

ISSN (Online): 2789-7338
Thematic Directions: Construction and Materials Science
Publisher: Technobius, LLP
Address: 17A Momyshuly street, office 22, 010000, Astana, Republic of Kazakhstan

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Website: https://technobius.kz/
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CONTENTS

Technobius, 2023, 3(2), 0036, DOI: https://doi.org/10.54355/tbus/3.2.2023.0036

Technobius

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e-ISSN

2789-7338

Article

Evaluation of tensile strength characteristics of geosynthetic materials designed to ensure embankment stability

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Abstract. This article highlights the significance of geogrids and geosynthetic materials in addressing geotechnical engineering challenges and provides a foundation for further research and advancements in this field. The article explores the role of geogrids and geosynthetic materials in modern geotechnical engineering. Geogrids are three-dimensional structures made of polymer materials with apertures or cells filled with soil or other materials. They are extensively utilized for soil reinforcement, erosion control, surface stability, and ensuring the durability of various geotechnical structures. Geosynthetic materials, in turn, are artificial materials produced from polymers and are used for soil filtration, separation, protection, and reinforcement. They find wide application in various geotechnical systems and constructions, including drainage systems, hydrological barriers, road construction, and airports. The article also describes the Strain-control method for testing geosynthetic materials, allowing for result adjustments relative to specimen dimensions. The research underscores the significance of geogrids and geosynthetic materials in contemporary engineering practice and provides a foundation for further investigations and developments in the field of geotechnics.
Keywords: geogrids, geosynthetic materials, strength testing, tensile testing, embankment.

1. Introduction

Geogrids and geosynthetic materials are vital components in modern geotechnical engineering. They play a significant role in addressing various engineering challenges related to soil reinforcement, erosion protection, and land mass control.

Currently, the use of geosynthetic materials for soil improvement methods has gained significant popularity in geotechnical construction practices. These materials are employed for various methods of soil reinforcement, as local soil is one of the most cost-effective and readily available materials on construction sites.

Soil foundation reinforcement is widely applied in strengthening building foundations, transportation infrastructure, and the construction of various storage facilities.

This study provides experimental and theoretical research materials in the field of slope and embankment protection technology using geosynthetic materials. These materials enable more efficient design and construction of earth structures.

Reinforcing foundations with geosynthetic materials for deformable soil masses is one method of improving the strength and deformation properties of soils. However, the behavior of reinforced foundations under soil deformation is currently not well understood. Therefore, the research and development of stability methods for such foundations are of utmost importance [1].

Geogrids are three-dimensional structures made from polymer materials that feature openings or cells filled with soil or other materials. They are extensively used for soil reinforcement and stabilization, erosion prevention and control, the creation of stable surfaces, and ensuring the longevity of various geotechnical structures, such as roads, slopes, embankments, earth dams, and more.

Monograph [2] considers the reinforcement of the foundation, which leads to an increase in the ultimate load and a decrease in the die settlement. Reinforcement prevents the development of shear zones in depth - we can see the shear strain values dropping at the intersection of geogrids. Localization of shear strains in the form of strips is less evident. The character of deformation in general is similar to a natural base with a slightly wider and buried to the level of the lower layer of the geogrid stamp. The limit of linear proportionality of the pressure-settlement graph for the reinforced base also increases. This is due to the large distribution capacity of the reinforced base due to the lateral compression of the soil in the reinforcement zone

Geosynthetic materials, in turn, are artificial materials produced from polymers or polymer combinations that possess specific geotechnical properties. They serve various functions, including filtration, separation, protection, and soil reinforcement. Geosynthetics are employed in diverse geotechnical constructions and systems, such as drainage systems, soil and hydrological barriers, as well as in the construction of roads and airports [3].

Research objectives:

1) Testing five types of geogrids to determine the optimal choice:

– Conducting tensile strength tests on five types of geogrids.

– Performing tests on geogrids using standardized methods to evaluate their strength under various conditions.

2) Analyzing the results of geogrid testing and selecting the most effective and durable geogrid. Establishing optimal usage parameters for the selected geogrid based on the obtained data, such as element spacing, stress on the geogrid, and other factors.

3) Selecting an equivalent geogrid for model testing based on tasks 1 and 2:

– Defining the requirements for model testing, including soil parameters, loads, and operating conditions.

– Utilizing the test results from tasks 1 and 2 to select an equivalent geogrid that corresponds to the primary geogrid but can be utilized in more controlled model testing conditions.

– Conducting model tests using the chosen equivalent geogrid and comparing their results with the data from the main research.

– Analyzing the results of model tests and the draw conclusions to confirm the effectiveness of the selected equivalent geogrid.

Methods

The tests were conducted using the Strain-Control method, which is based on maintaining a specified level of deformation in the specimen throughout the test. In the case of specimen elongation, the deformation is controlled and maintained at a constant level, while the tensile forces required to maintain this deformation are measured. In this method, we applied tensile forces that were adjusted relative to the actual length of the specimens [4].

For conducting Strain-Control tests, specimens of geosynthetic materials are subjected to elongation at a specified deformation rate, while the tensile forces are simultaneously measured. To adjust the tensile forces relative to the actual length of the specimens, relevant formulas or calculation methods are used, which take into account the specimen length and its geometric parameters.

Thus, when using the Strain-Control method to compare the strength of geosynthetic materials, the tensile forces can be adjusted considering the actual length of the specimens, enabling a more accurate comparison and analysis of their strength properties [5].

The tests were performed on specimens with a width close to the width of the clamping device (clamp jaws), which was 16 cm. Since the distance between the geogrid ribs and the aperture of each specimen is individual, the widths of the specimens varied. Therefore, when comparing the strength indicators of the specimens, it is necessary to adjust the tensile forces relative to the actual length of the specimens. The results of the actual tensile forces, obtained from the Strain-Control tests (at specified displacements), are presented in Figure 1 [6].

The tests were conducted for 5 different types of geogrids, as presented in Table 1.

Table 1 – Geosynthetic materials

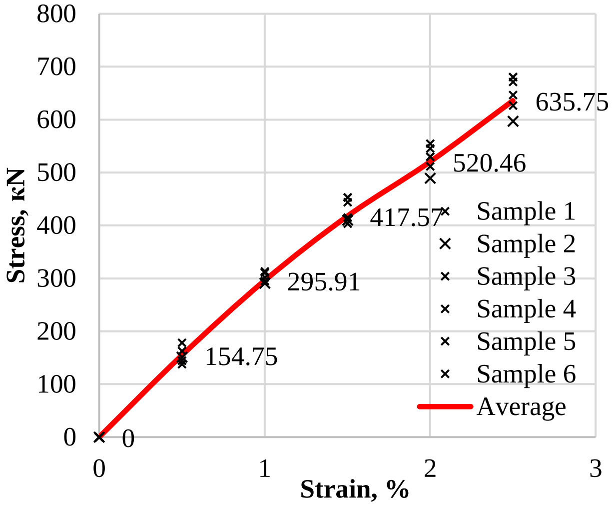
No. of sample	Appearance	Characteristic
Type 1		Polyester geogrids - reinforcing material used in the upper layers of pavements during construction, repair and reconstruction of roads and railways, airfields, bridges and overpasses, reinforcement of weak bases, as well as in other geotechnical constructions.
Type 2		SD geogrid is a material made of polypropylene. In order to obtain high strength characteristics with low creep, the mesh is stretched in two directions during the production process Geogrid is specially designed to increase the ability of structures to bear high dynamic and static loads, including construction on weak soils.
Type 3		The composite bonded in this way evenly distributes loads over large areas, thus dramatically increasing the bearing capacity of weak subgrade soils. This property lies in the ability of the material to absorb various tensile forces even with minimal deformation and settlement of the foundation.
Type 4		Geogrid is specially designed for reinforcement of bearing pavement bases, as well as for construction on weak soils and for use in structures that support high dynamic and static loads.

Type 5		Polymer hexagonal geogrid "RGK" is a flat triaxially oriented polypropylene lattice with a triangular cell, used for reinforcement of large-fraction bearing layers of pavement, as well as for use in structures subject to increased static and dynamic loads and in the base of the earth bed on weak (subsidence) soils of the existing base [7].

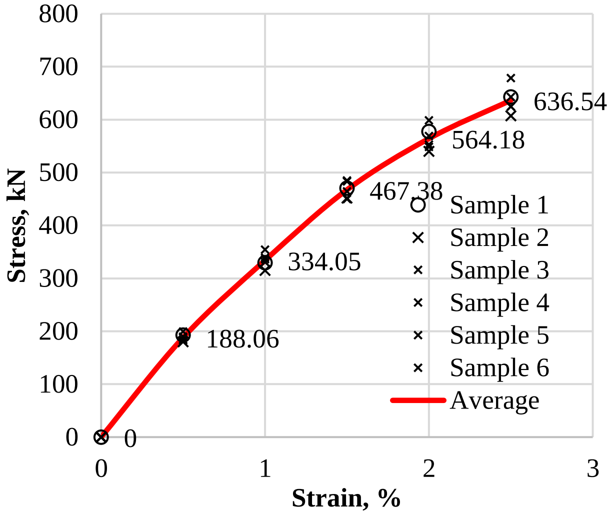
The tests were performed on a tensile press. The results for all 5 samples are shown as follows.

3. Results and Discussion

Figure 1 depicts the results of the tensile forces tests conducted on specimens of the compared types of geogrids (Type 1 – Type 5). Each graph shows the results of all six individual tests for each type of geogrid. Figure 2 shows the mean value of the tests of all samples.



Type 1



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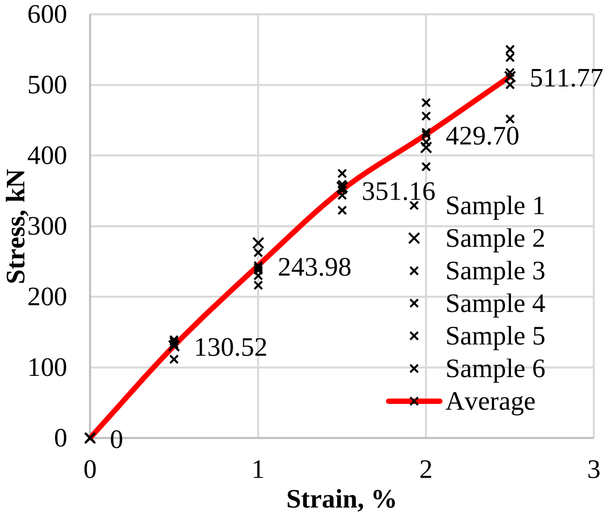

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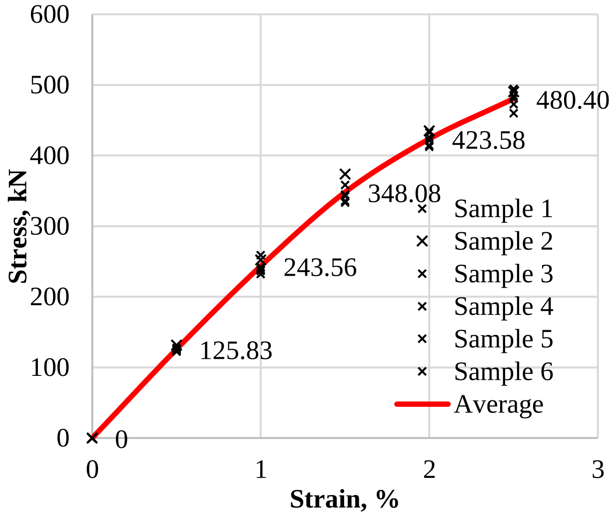
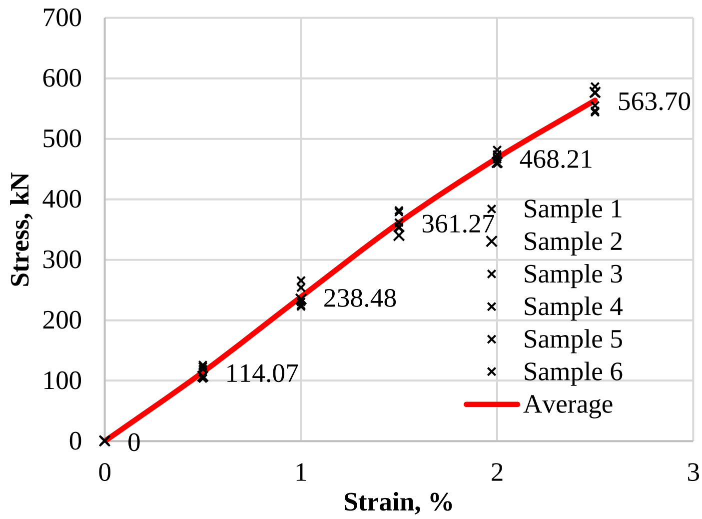


Type 4



Type 5

Figure 1 – Graph of geogrid tensile strength

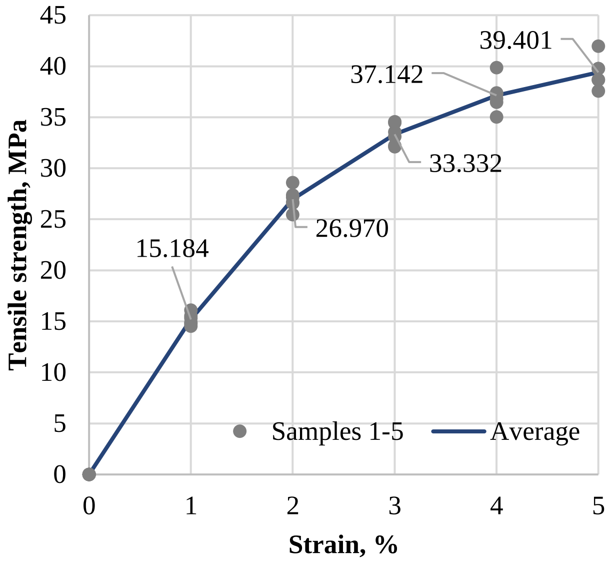
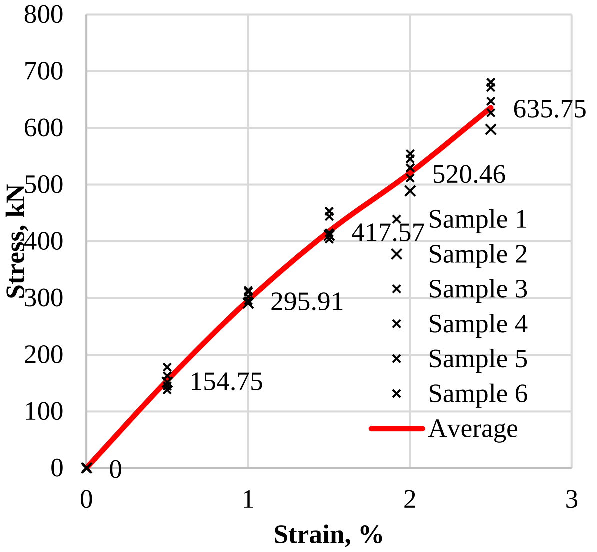


Figure 2 – Tensile test results without correction average of all specimens.

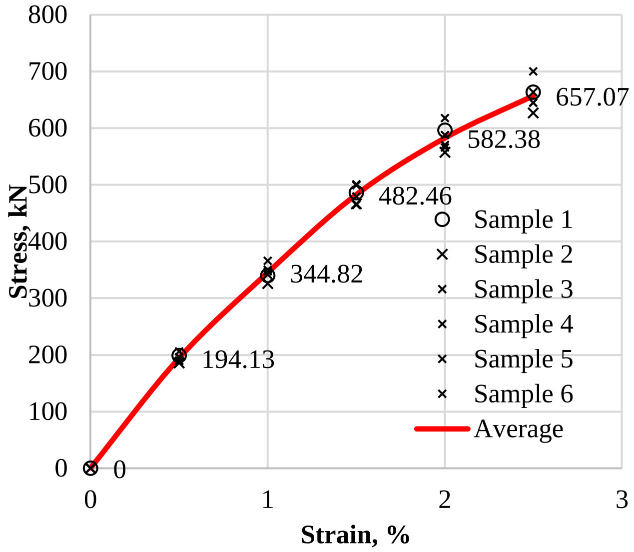
The correction factors for each sample type are presented in Table 2. The correction was made relative to the dimensions of Sample 1. The results, considering the corrections based on the specific length of the specimens, are presented in Figure 3. According to the test results, the highest tensile resistance values were observed for Sample 2. Therefore, for the selection of geometric dimensions for the equivalent geogrid, further tests to determine the tensile strength of a single rib will only be performed for Sample 2.

Table 2 – Correction indicators

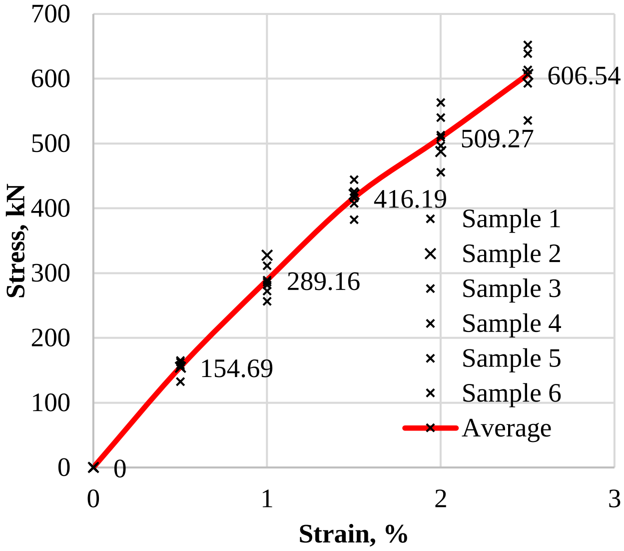
Indicators	Type 1	Type 2	Type 3	Type 4	Type 5
Sample width, cm	12.8	12.4	10.8	13.6	13.5
Correction, $\frac{\text{Sample width1}}{\text{Sample width } N}$	1.00	1.03	1.19	0.94	0.95



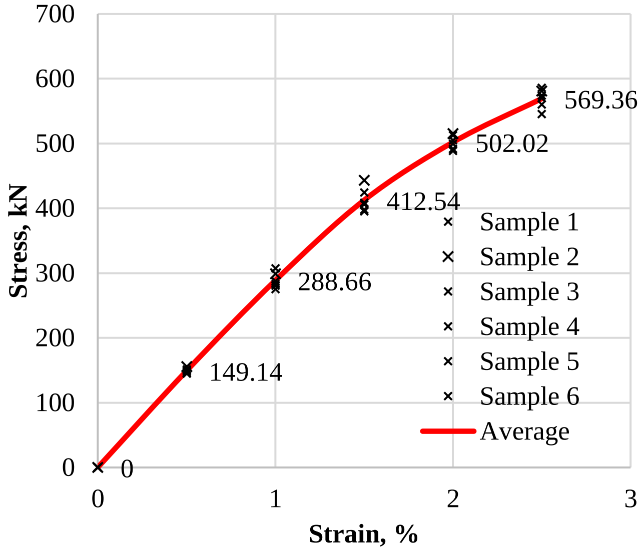
Type 1



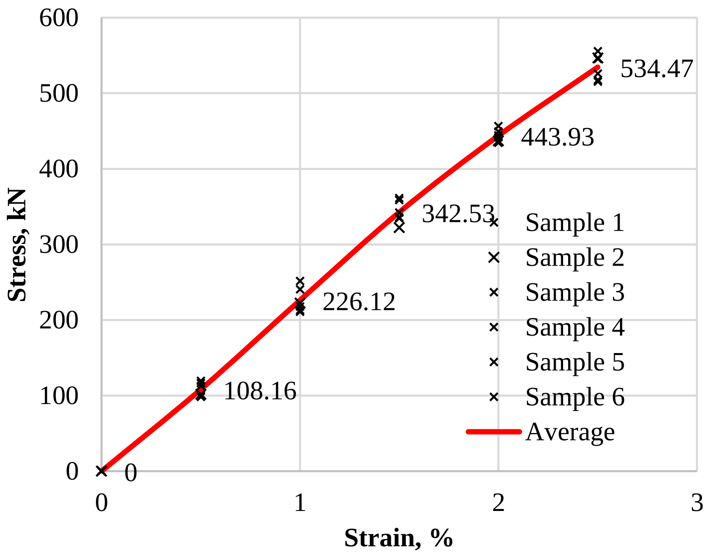
Type 2



Type 3



Type 4



Type 5

Figure 3 – Graph of geogrid tensile strength

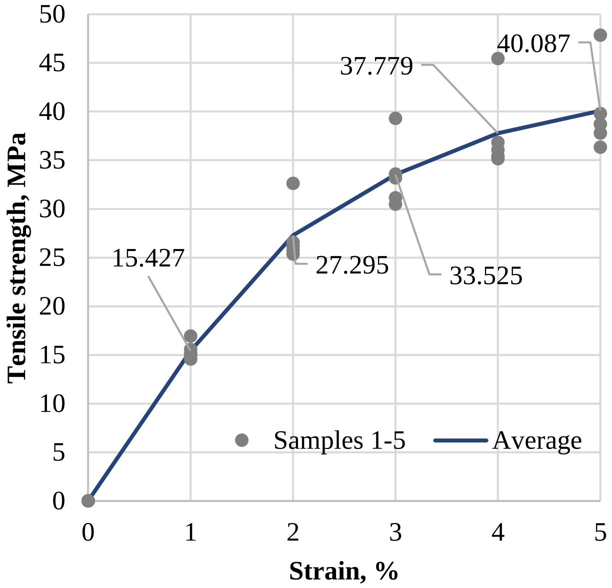


Figure 4 – Tensile test results with corrected average of all specimens.

Ultimately, Type 2 was selected as the most optimal choice due to its highest strength and performance relative to its geometric dimensions and specific cross-sectional area, as presented in Figure 4.

Research conducted by other authors in the past provides a context for our work and reinforces the overall significance of employing geogrids and geosynthetic materials in contemporary geotechnical engineering. This reaffirms the relevance and importance of our study, contributing to the broader comprehension of geotechnical and geotechnical engineering fields.

However, it should be noted that some discrepancies with prior research may arise due to differences in testing methodologies, experimental conditions, or characteristics of the utilized geosynthetic materials. These present opportunities for further research to refine and expand upon our findings [8].

In conclusion, the discussion of the results in comparison with analogous studies substantiates the significance and timeliness of our research and identifies prospects for further investigations in the realm of geotechnical engineering.

4. Conclusions

Based on the conducted research, the following conclusions can be drawn:

– By using the Strain-Control method to compare the strength of geosynthetic materials, the adjustment of tensile forces relative to the actual length of the specimens allowed for a more accurate comparison and analysis of their strength properties.

– The test results indicate a significant influence of geosynthetic materials on soil reinforcement, erosion prevention, and the durability of geotechnical structures. The effectiveness of geogrids and geosynthetic materials in solving engineering problems has been demonstrated.

– These findings can be utilized in further research and development in the field of geotechnical engineering to create more stable and reliable engineering structures.

– According to the test results, the highest tensile resistance was observed in Sample 2. Therefore, for further trough testing and Plaxis simulations, Sample 2 was chosen due to its higher resistance to tension.

– These findings provide valuable insights for the field of geotechnical engineering and can contribute to the development of more robust and reliable engineering constructions.

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Aigerim Yenkebayeva – visualization, drafting, editing.

Received: 18.06.2023
Revised: 24.06.2023
Accepted: 24.06.2023
Published: 30.06.2023

Technobius, 2023, 3(2), 0037, DOI: https://doi.org/10.54355/tbus/3.2.2023.0037

Technobius

https://technobius.kz/

e-ISSN

2789-7338

Article

Methodology for determining the extent of soil compaction deformation zones beneath foundations

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Abstract. The article is dedicated to a methodology for determining the actual dimensions of the zone of long-term compressed soils under existing foundations from their previous loading. Although construction norms provide conditional criteria for this, they do not always accurately reflect the real distribution of soil deformations at the depth of the foundation. The article presents a methodology for determining the actual dimensions of the compacted soil zone under the foundations. This methodology is based on both empirical and theoretical approaches, taking into account both the strength and deformation characteristics of the soil. The application of this methodology allows obtaining more accurate results and more reliably determining the dimensions of the compacted zone, which can be useful for conducting additional research for object reconstruction or determining the permissible distance between closely located foundations. During experimental and theoretical studies, regularities in the changes of soil compaction at the depth of the foundation and its deformations over time under constant loading were identified. The developed methodology enables a more precise determination of the actual dimensions of the compacted zone, which is of great significance for ensuring the safety and reliability of building structures.
Keywords: consolidated zone, soil, foundations, deformation, settlement.

1. Introduction

The formation of the consolidated zone of soils beneath foundations depends on several factors, such as soil type, foundation load, depth of placement, and foundation laying method. Practical methods based on the theory of linearly deformable bodies do not accurately predict the actual dimensions of the active foundation zone since they are designed for calculating assumptions about idealized soil conditions. Consequently, these methods often yield inflated values for the parameters of the consolidated foundation zone. Thus, there is a need to seek a new approach to assess the actual dimensions of the deformable zone beneath building and structure foundations. It is well-known that when calculating foundation settlements, the main task is to determine the ground displacements resulting from the action of vertical stress components σ_z. However, in reality, the foundation settlement occurs due to the volumetric compression of the soil within a limited zone under the combined effect of all stress components. This is confirmed by the results of numerous experiments – the actual dimensions of the soil compaction zone beneath foundations significantly exceed the boundaries of the isobar $\sigma_z = \sigma_{str}$ (where σ_{str} is the compressive strength of the soil) for both plane and spatial problems [1].

In the context of designing buildings within existing urban developments, including additions or extensions with consideration for potential future inundation of foundations, it is crucial to determine the actual dimensions of the long-term soil consolidation zone beneath existing foundations from their previous loading. The deformation of the foundation within a confined zone results primarily from structural consolidation deformations, leading to settlement of the foundation. These deformations cause significant changes in the inherent compressibility, strength, and other properties of the natural soils. Numerous studies have been conducted by researchers on the formation of the deformation and consolidation zone beneath loaded foundations and real buildings and structures. The results of these experimental studies have shown that the settlement of the foundation is caused by deformation within a confined zone, where structural consolidation deformations predominantly develop. As a consequence of these deformations, there is a substantial alteration in the original characteristics of the soils, including their compressibility, strength, and other properties associated with the natural soil compression process over time. During the compression of clayey soils, consolidation deformations predominantly occur, leading to changes in the soil structure. This is manifested by a reduction in porosity, more densely packed solid particles, and an increase in the contact area between them. Consequently, soil strengthening occurs through increased interlocking forces between the solid particles, primarily driven by structural and textural changes [2–4]. Various methods, such as optical, X-ray diffraction, and electron microscopy techniques, are employed to investigate changes in the texture of soils during their deformation. These methods enable the determination of the orientation of structural elements, such as particles and aggregates, before and after compaction or shearing. The utilization of such methods allows for a more detailed examination of the internal soil structure and understanding of the mechanisms that lead to its strengthening. This is crucial for comprehending the soil properties and developing appropriate models and methods to predict its behavior under loading and deformation. The analysis of soil texture using different methods during its deformation plays a significant role in understanding the mechanisms of soil compaction and consolidation [5].

The post-genetic changes in the properties of soils subjected to compaction beneath building foundations cannot be fully explained solely by the process of soil compaction. In the soil mass of urbanized areas, more complex phenomena occur, which involve not only soil compaction but also soil strengthening with alterations in the initial strength properties. Prolonged soil compaction can induce additional processes, such as water dissipation and an increase in interlocking forces between solid soil particles. As a result of these processes, structural and textural changes in the soil take place, leading to its strengthening. Soil strengthening due to prolonged compaction can be accompanied by modifications in its strength properties, including an increase in the coefficient of internal friction and cohesion, elevation of the angle of internal friction, and a reduction in compressional deformation [6]. The accurate assessment of the actual dimensions of the deformable zone beneath building and structure foundations holds significant importance from both scientific and practical perspectives in the field of construction. These parameters play a crucial role in solving numerous tasks related to engineering surveys and design. It is particularly important to consider these parameters when refining methods for calculating foundation settlements, taking into account local changes in soil properties within the active foundation zone during the building's service life [7].

The calculation of foundation settlements should take into account the factor of soil compaction, especially when dealing with extensions of existing structures, completion of unfinished preserved objects, and considering the impact of potential foundation inundation in operational buildings. Determining the optimal thickness of the reinforced load-bearing soil layer during the construction of artificial foundations, defining the required depth of dewatering, and establishing the maximum allowable distance between neighboring foundations also necessitate a reliable assessment of the dimensions of the deformable foundation zone. Additionally, evaluating the sizes of settlement funnels that may occur around constructed objects during the stabilization of foundation soil deformations is crucial in the design of vertical planning for developed areas [8].

During the survey of foundations for reconstructed structures, it is also essential to consider the actual dimensions of the deformable zone to determine the optimal scheme for the horizontal and vertical positioning of sampling points. This includes the use of trenches, boreholes, and conducting field tests on soils, such as plate load tests, static and dynamic probing, and other methods. Hence, the accurate determination of the actual dimensions of the deformable foundation zone plays a crucial role in solving various tasks in the field of engineering surveys and design. It ensures the reliability and safety of construction and helps develop effective solutions for specific construction challenges [9]. The investigation of existing theoretical and experimental methods for determining the dimensions of the consolidated zone beneath foundations is currently insufficiently developed. The existing analytical methods only yield approximate results [10]. There is currently no methodology for assessing similar parameters in the case of unconnected soils that lack cohesion. This creates difficulties in solving important practical tasks related to engineering surveys and the design of buildings and structures, especially when constructing on urbanized territories subject to various post-construction anthropogenic impacts. In light of the aforementioned, the aim of this research is to develop a methodology for determining the dimensions of the soil compaction deformation zone beneath foundations, taking into account both the strength and deformation characteristics of the soils.

2. Methods

To determine the dimensions of the soil compaction zone beneath strip footings, the method of isobars of maximum principal normal stresses is commonly employed, assuming that these isobars have a circular shape [11]. The calculations are based on the following condition:

$$\sigma_1 = \sigma_{str} \quad (1)$$

The extension of the foundation's compaction zone beyond the loaded area with a width of b is determined by the following equations:

$$l_a = (B_a - b)/2; \quad l_a/b = 0{,}5\left(\frac{B_a}{b} - 1\right) \quad (2)$$

Based on the literature review and statistical analysis of field test results conducted in various soil conditions and carried out according to a standardized procedure using large square-shaped stamp plates (with a base area of at least 0.5 m²), correlation dependencies have been obtained. These correlations include $B_a/b = f(H_a/b)$; $l_a/b = f(H_a/b)$ and $H_a/b = f(\sigma_{str})$. However, for construction practice, the most significant interest lies in the methodology for determining the dimensions of the deformable foundation zone, taking into account the deformational properties (E_o, m_o) and the strength properties (c, φ) the soils, obtained through geotechnical investigations. The analysis of field test results, during which measurements of the foundation's compaction zone dimensions were combined with the determination of soil deformation moduli, has allowed us to establish a highly significant correlation relationship $H_a/b = f(E_o)$ with a correlation coefficient of $\eta = 0{,}9$ (Figure 1).

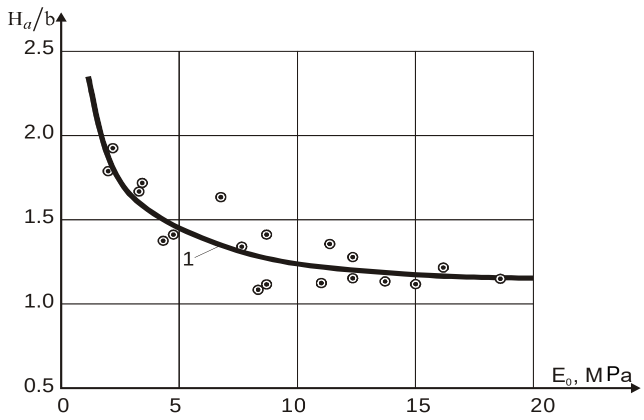


Figure 1 – The dependence of the relative depth of the foundation's compaction deformation zone for square footings on the value of the soil's deformation modulus: ⦿ – The experimental data points (correlation field); 1 – The curve obtained from the empirical equation

Based on the obtained regularities, it appears feasible to determine the dimensions of the soil compaction zone beneath square footings using the following correlation equations:

$$B_a/b = 0{,}14 + 1{,}13H_a/b, \quad (3)$$

$$l_a/b = -0,43 + 0,565H_a/b, \quad (4)$$
$$H_a/b = 0,596 + 0,046/\sigma_{str}, \quad (5)$$
$$H_a/b = 1,05 + 1,95/E_o, \quad (6)$$

Where: 0.046 is a coefficient with the dimension of MPa^{-1}; E_o is the soil's modulus of deformation in MPa; σ_{str} is the compressive strength of the soil's structural bonds in MPa.

The proposed equations are applicable to foundations with a circular footprint as well, provided that the relationship $b = D/1.13$ is introduced, where D represents the diameter of the circular footing and 1.13 is the transition coefficient. The value of σ_{str} in Eq. (5) is recommended to be determined using Eqs. (7) and (8), which are derived from the condition of natural soil structure failure during compaction.

For cohesive soils, the exponent can be determined using the following equation:

$$\sigma_{str} = \frac{\xi_0 - \xi_a}{\xi_a} \cdot \sigma_{nat}, \quad (7)$$

For cohesive soils possessing cohesion, according to the equation:

$$\sigma_{стр} = \frac{1}{\xi_a}\left[(\xi_0 - \xi_a) \cdot \sigma_{быт} + 2C\sqrt{\xi_a}\right]. \quad (8)$$

Where: ξ_0 is a coefficient of lateral earth pressure for soil in a state of rest is determined by the equation:

$$\xi_0 = \frac{\mu}{1-\mu}, \quad (9)$$

where μ – is the coefficient of lateral earth pressure. The maximum principal normal stress at the considered point M is equal to the natural pressure of the soil σ_{nat}:

$$\sigma_1 = \sigma_1^g = \gamma \cdot h_1;\ \sigma_1^l = \sigma_1^g = \sigma_{nat} \quad (10)$$

Where: γ - specific gravity of soil; h_1 – thickness of the soil strata; ξ_a is a coefficient determined depending on the angle of internal friction, determined by the equation:

$$\xi_a = tg^2(45^o - \frac{\phi}{2}) = 1 \quad (11)$$

These dependencies can also be applied for approximate estimation of the size of the actual soil compaction zone under strip foundations, if correction factors are introduced into Eqs. (5) and (6), which take into account the difference of the stress state for the cases of spatial and planar problems.

For this purpose, it is proposed to use the transition coefficient:

$$K_\sigma^c = F_\sigma^{fl} / F_\sigma^{sp}, \quad (12)$$

Where: F_σ^{fl} and F_σ^{sp} are the areas of influence coefficient curves K_z (curves 1 and 2 in Figure 2) corresponding to the flat and spatial problems used to determine the distribution of normal stresses ($\sigma_z = \sigma_1$), occurring under the center of the foundation footing from the action of load P_0.

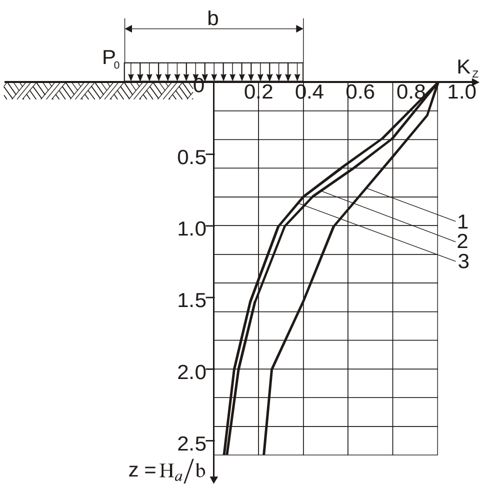


Figure 2 – Curves of influence coefficients K_z for determination of normal stresses under the center of the foundation footing from the action of the load P_0: 1 – for the case of a plane problem

(for strip foundations); 2 and 3 – for the case of a spatial problem (respectively, for square and circular foundations)

At the same time, the transition coefficient K_σ^c should be determined for the values of H_a/b, calculated by Eqs. (5) and (6). For example, at given values of $H_a/b = 1.0;\ 1.5;\ 2.0;\ 2.5$, the following values of the transition coefficient $K_\sigma^c = 1.17;\ 1.27;\ 1.36;\ 1.45$ are calculated by Eq. (12). For intermediate values of H_a/b, the corresponding values of the coefficient K_σ^c are recommended to be determined by linear interpolation.

In such a case, the actual compaction depth of the strip foundations can be determined according to the equations:

$$H_a^{fl}/b = K_\sigma^c \cdot (0.596 + 0.046/\sigma_{str}) \tag{13}$$

$$H_a^{fl}/b = K_\sigma^c\ (1.05 + 1.95/E_0) \tag{14}$$

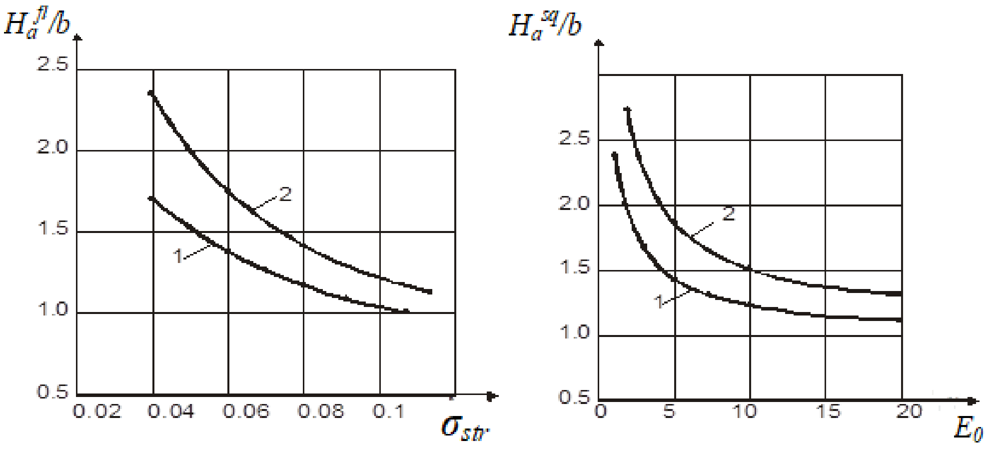
3. Results and Discussion

The boundary of the soil compaction deformation zone beneath foundation footings is a line that represents the geometric locus of points where the action of compressive normal stresses is balanced by the strength of the soil's structural bonds.

The settlement of the foundation footing is mainly influenced by the deformation resulting from the compaction of soils within a specific volume, which is commonly referred to as the foundation compaction deformation zone.

Figure 3 a and b show the graphs of dependence $H_a^{sq}/b = f(\sigma_{str})$; $H_a^{sq}/b = f(E_0)$ obtained by Eqs. (5) and (6) for square foundations (curves 1), as well as the graphs $H_a^{fl}/b = f(\sigma_{str})$ and $H_a^{fl}/b = f(E_0)$ plotted by Eqs. (13) and (14) for strip foundations (curves 2).

The width (B_a^{fl}) or extension (l_a^{fl} of the compaction zone of the strip foundations should be taken according to the dimensions of the isobars of the largest main normal stresses σ_1 at the given values $Z = H_a^{fl}/b$, determined from Eqs. (13) and (14).



a) b)

Figure 3 – The graph shows the relationship of the relative depth of the compaction deformation zone beneath foundation footings to the strength and deformation properties of the soil: a) The graph illustrates the dependency of the actual depth of the foundation compaction zone on the compressive strength of the soil's structural bonds; b) The graph depicts the relationship of the actual depth of the foundation compaction zone on the soil's deformation modulus

Figure 4 shows the graphs of dependence of l_a (or B_a) on H_a in relative coordinates, obtained from experimental data (graph 1) using Eqs. (3) and (4) and from theory - graphs 2, 2' and 3, plotted according to isobar dimensions σ_z (for square and strip foundations) and σ_1 at values $Z = H_a/b = 1;\ 1.5;\ 2.0;\ 2.5$, respectively.

As can be seen from the comparison of graphs 1 and 3, our proposal to estimate the width (or removal) of the soil compaction zone under strip foundations by the size of isobars σ1 has experimental confirmation, since these graphs within the most probable values of $H_a = 1b...2b$ converge.

At the same time, the known recommendations [12] on determining the dimensions of the deformed zone of the foundation base by constructing isobars of vertical normal stresses σ_z (for square - curve 2 and for strip foundations – curve 2') are not confirmed by the results of experimental investigations - graphs 1 and 2, 2' diverge.

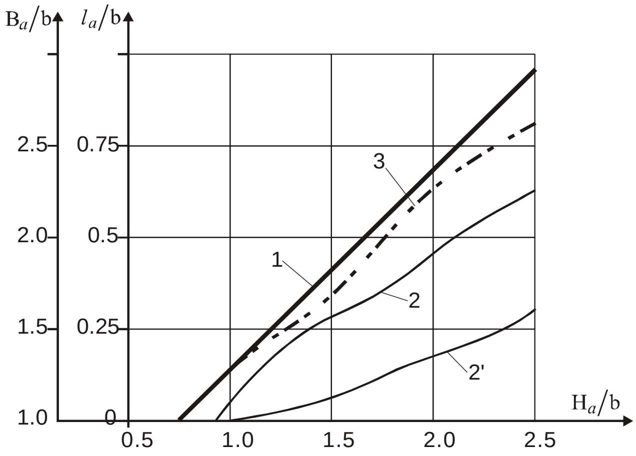


Figure 4 – Graphs depicting the relationship of the width and extension of the soil compaction zone beneath foundation footings to its depth in relative coordinates: 1 – Experimental graph obtained from the empirical equation; 2 and 2' – Theoretical graphs constructed based on the sizes of the σ_z isobars (for square and strip footings) at values of $z=H_a/b=1; 1.5; 2.0$ and 2.5; 3 – Theoretical graph obtained from the sizes of the σ1 isobar at values of $z=H_a/b=1; 1.5; 2.0$ and 2.5

This engineering method for estimating the dimensions of the deformable zone beneath the foundation can also be applied to rectangular footings, if the transition coefficient according to Eq. (13) is determined considering the aspect ratio of their base. In this case, the influence coefficient curve, K_z will lie between curves 1 and 2 (Figure 2). The calculation is then conducted similar to a strip footing using Eqs. (13) and (14), but with the consideration of the transition coefficient, K_σ^c, calculated for the rectangular footing.

4. Conclusions

The analysis of the research results has led to the following conclusions:

1. The proposed methodology for determining the actual dimensions of the soil compaction deformation zone beneath various types of foundations, including square, circular, strip, and rectangular footings, has demonstrated high accuracy and reliability of the results. This signifies a significant contribution of the study to the field of geotechnical engineering.

2. The experimental data have confirmed the theoretical assumptions regarding the influence of the soil's structural bond strength on the size of the foundation's deformation compaction zone. Understanding these interconnections is an essential aspect in developing the methodology and applying the results in practical projects.

3. The importance of considering the deformation characteristics of the soil during the process of determining the dimensions of the compacted zone has been validated by the research findings. This enables a more accurate representation of soil compaction processes and their impact on construction structures.

4. The studies have shown that the sizes of the deformation compaction zone can significantly vary for different types of soils and foundations. This highlights the importance of accounting for the specific characteristics of the soil foundation during the design and construction of various structures.

5. The developed empirical and theoretical methods have a wide range of potential applications in engineering projects. They can be used for determining safe design parameters, calculating foundation settlements under various conditions, and optimizing construction solutions.

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Received: 19.06.2023
Revised: 25.06.2023
Accepted: 25.06.2023
Published: 30.06.2023

Technobius, 2023, 3(2), 0038, DOI: https://doi.org/10.54355/tbus/3.2.2023.0038

Technobius

https://technobius.kz/

e-ISSN 2789-7338

Article

Fine-grained concrete for repair and restoration based on complex modifiers

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Abstract. Nowadays ensuring normal operation of buildings made of monolithic reinforced concrete is of great relevance. Bearing building structures in the process of operation or erection can have defects and damages of various origins. There is a need to develop repair compositions to ensure the durability and reliability of monolithic reinforced concrete buildings. In this paper we have conducted research and developed a repair composition of fine-grained concrete (FGC) on the basis of modification and improvement of its structure. In the composition of complex modifier on the basis of analysis were chosen: microsilica MCU-95 as a highly active pozzolanic mineral admixture; superplasticizer C-3, hydrophobizing component – soapstock. Cements of Novo-Karaganda, Ust-Kamenogorsk, Shymkent cement plants M400 were chosen as binders. In order to study the influence of modifiers on the setting time of cement dough and concrete structure, several concrete compositions were developed. The performance of the developed FGC was improved in terms of basic physical and mechanical parameters. Cement consumption was reduced by 15% (from 450 kg to 382.5 kg). The influence of the main factors and dependence of performance indicators on physical and mechanical properties of cement stone and the studied FGC has been established. Production waste and local construction materials were used to save binder.
Keywords: microsilica, concrete, repair, repair, composition, defects, damage.

1. Introduction

Currently, construction in Kazakhstan is developing rapidly. There is an increase in the construction of low-rise and high-rise residential buildings, various commercial and cultural centers. Monolithic construction of higher storey with load-bearing stone structures or reinforced concrete space frames has been widely used. In the monolithic buildings constructed in recent years, there is a significant number of defects of load-bearing structures [1–3] (Figure 1). The nature of defects and damages in buildings made of monolithic reinforced concrete depends on many factors and reasons: the quality of engineering surveys and design documentation, the quality of materials and products, compliance with the quality control system during construction, as well as violation of the technology of construction works, including the use of poor-quality formwork [4]. Physical deterioration of reinforced concrete structures under the factor of time: creep and shrinkage of concrete, exposure to aggressive external environments, mechanical loads lead to cracking of concrete, corrosion of steel, peeling of concrete coating, and, as a consequence, loss of serviceability and safety of the structure [5]. This has caused a huge need to improve the performance of existing reinforced concrete structures by repairing and reinforcing them in order to extend their service life. According to the results of the authors' research [6] – the main cause of defects in buildings and structures is the inability to ensure the necessary characteristics of concrete, both at the stage of concrete mixture preparation and at the stage of erection of the finished monolithic structure.

Ensuring reliability, durability and increasing the service life of buildings made of monolithic and prefabricated reinforced concrete structures is an urgent issue in the construction industry not only in Kazakhstan, but also in the whole world. The construction industry of all industrially developed countries spends about 42 % of financial injections for repair and operating costs on objects built of reinforced concrete. In [7], a repair cost analysis using PSLF (probabilistic service life function) is proposed, where changes in initial and extended service life due to repair are considered.

In view of the relevance of the use of concrete mortars for the repair of concrete and reinforced concrete structures, the works and research work of many researchers have been studied. Since concrete on coarse aggregate is not quite acceptable for its application in thin layers, it was decided to use crushed stone of 5...10 mm size as an aggregate. The concrete obtained on such aggregate is classified as fine-grained concrete.

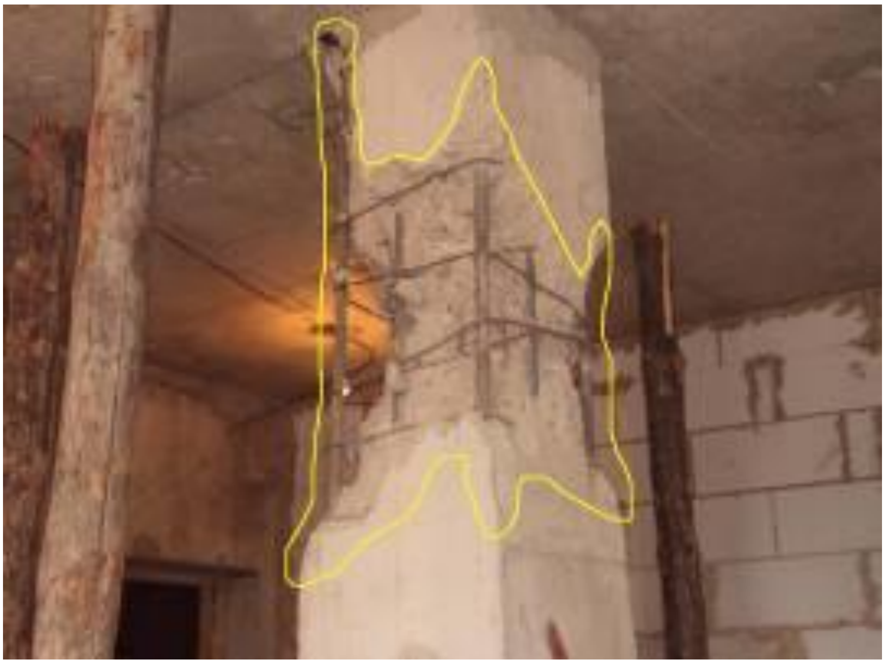

Figure 1 – Column concrete failure with bare and corroded reinforcement

In the present work, the research and development of repair composition of fine-grained concrete (FGC) based on modification and improvement of its structure, which consists in improving its physical-technical and hydraulic properties. To obtain FGC with high repair properties, the existing materials were reviewed and compared.

It is well known that the repair mortars used are made from ordinary Portland cement (OPC), which were developed to repair old concrete to achieve better compatibility with concrete bases [5]. To obtain concrete with improved technical and performance characteristics, one of the most promising directions in construction is to modify concrete with a complex of modifying additives and production wastes, which is economically feasible and develops domestic house building. The paper [8] presents the results of research work on the development of fine-grained polymer concrete modified with microsilica. The authors of [9] used 5%, 7.5% and 10% (by weight) of microsilica to improve the mechanical properties of mixtures as a partial replacement of cement, and the results show that the mixture with 25% recycled aggregates and 5% microsilica showed characteristics comparable to conventional concrete, while the use of a higher content of microsilica negatively affected the rheological properties.

In order to obtain concrete with enhanced performance characteristics under the condition of minimizing raw material, energy and labor costs in the present work, studies on the development of the optimal composition of FGC were carried out. The following materials were used: microsilica MCU-95, superplasticizer - C-3, hydrophobizer - Soapstock and curing gas pedal - sodium sulfate.

The purpose of the work is to study and develop an effective composition of concrete FGC for repair and restoration of reinforced concrete structures in the construction and operation of residential buildings.

The purpose of the work should be achieved by solving the following tasks:

– introduction of microsilica, hydrophobizer and curing gas pedal in addition to superplasticizer into the concrete composition to obtain an effective FGC composition;

– to find out the optimal ratio of components to obtain FGC with high performance characteristics, effectively used in the repair of residential buildings made of concrete and reinforced concrete;

– to establish the influence of the main factors and dependence of performance indicators on physical and mechanical properties of cement stone and the studied FGC.

2. Methods

In the course of research and practical work the following materials were used.

Cements of Novo-Karaganda, Ust-Kamenogorsk, Shymkent cement plants M400 were chosen as a binding material. Characteristics on mineralogical and chemical composition of cements are given in Tables 1 and 2.

Table 1 – Mineralogical composition of cements

Name of manufacturing plant	Mineral content, %			
	C_3S	C_2S	C_3A	C_4AF
Karaganda	58.16	18.92	8.61	13.52
Ust-Kamenogorsk	54.88	22.96	9.70	12.31
Shymkent	51.92	15.06	8.18	13.96

Table 2 – Chemical composition of cements

Name of manufacturing plant	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	SO_3	$CaO_{св}$
Karaganda	19.87	6.09	4.48	61.53	1.50	1.26	0.58
Ust-Kamenogorsk	21.68	6.29	4.02	67.60	0.80	0.44	0.67
Shymkent	19.42	6.05	4.83	61.67	1.63	1.74	0.71

The cements used in the work were tested in accordance with [10] and [11]. The results are attached in Table 3.

Table 3 – Cement test results

Name of plant, type and grade of cement	Bulk density, kg/m^3	Specific surface area, cm^2/g	Grinding fineness by residue on sieve No. 008, %	Normal density of cement dough, %	Ultimate strength at the age of 28 days, MPa	
					Compressive	Flexural
Karaganda Portland cement M400	1095	3100	8.4	27.0	43.0	5.9
Ust-Kamenogorsk Portland cement M400	1100	3200	8.2	27.0	42.0	6.0
Shymkent Portland cement M400	1100	3200	8.2	26.0	41.0	5.8

The tested cements, according to the obtained results, meet the requirements of [12].

Water for testing of cements and concretes corresponded to [13]. The analysis of total hardness of water used in the technology showed 12.8...100 mg-eq/l, from slightly brackish (dry residue up to 2.3 g/l) to strongly brackish (dry residue 3.0+19.5 g/l) and by acidity from slightly alkaline to slightly acidic (pH - 6.7...7.3).

Sand and crushed stone used in concrete mixtures meet the requirements of [14-15]. Characteristics of the used sand are given in Table 4.

Table 4 – Sand test results

Sand mine	Characteristics of sand		
	Coarseness modulus	Bulk density, kg/m^3	Impurity, %
Karagandanerud, JSC	2.6	1450	2…3
Gauhartas, LLP	2.2	1420	0.5…1
SBS Group, LLP	2.4	1480	1.5

The characteristics of the coarse aggregate used are given in Table 5.

Table 5 – Test results of coarse aggregate

Fraction of aggregate, mine	Name of indicators			
	Density, kg/m^3	Water absorption, %	Sparseness, %	Impurity, %
5-10 mm, Karagandanerud, JSC	1400	0.85	8.06	0.75
5-20 mm, Bektas Group, LLP	1420	0.96	9.24	2.06
5-10 mm, TechnoIndustry, LLP	1370	0.41	3.1	1.86

The composition of organomineral admixture of fine-grained concrete (FGC) for repair was developed on the basis of the following materials. In the composition of the complex modifier were chosen: microsilica MCU-95 as a highly active pozzolanic mineral admixture; superplasticizer C-3, contributing to the effective liquefaction of concrete mixtures [16], hydrophobizing component - soapstock [17-18], obtained by refining oils; curing gas pedal sodium sulfate (Na_2SO_4), meeting the requirements of TS 38.10742-84.

Requirements for superplasticizer C-3 and composition are given in Table 6.

Table 6 – Requirements for superplasticizer C-3

Name of indicators	Normal value
Appearance	The liquid is brown in color. A precipitate is allowed.
Active substance content in terms of dry product, % at least	69
Water content, % not exceeding	68
Ash content in terms of dry product, % exceeding	38
pH of 2.5 % aqueous solution	7-9

Superplasticizer C-3 is 20...40 % water concentrate and is supplied in tanks or drums. Percentage of water in the solution is regulated by passport data.

Soapstock is a production waste obtained during processing of vegetable oils, consisting of 41 % of fatty acids and 50 % of inactive inclusions (Table 7). Soapstock has the consistency of viscous dark brown paste, well and stable emulsifying with water.

Table 7 – Characteristics of soapstock (in accordance with TS-10-04-0280-91)

Name of indicators	Characterization		
	Light oil soapstock	Cottonseed oil	Animal fat soapstock

		soapstock	
Color	Light to light brown with a tinge of the color of the original oil	Brown to dark brown	Yellow to dark yellow with a grayish tint
Consistency at 20°C	Liquid or ointment	Ointment	Ointment
Smell	specific, characteristic of soapstock from various oils and fats fats and greases, a slight odor of decomposition products of organic substances is allowed, no odor of petroleum products is allowed, odor of decomposition products of organic substances is allowed, petroleum product odor is not allowed		
External solid impurities	Absence	Absence	Absence

Sodium sulfate (SN) is a powdered or granular product supplied in packaged form. According to the known method of concreting, its main application was to accelerate the processes of setting and hardening of concrete [19–22]. In the developed composition of FGC with modifying additives SN, stabilizing the action of soapstock, is simultaneously a reagent that actively hardens the components of the liquid structure on the surface areas of mineral particles.

The International Union of Experts and Laboratories for Testing of Building Materials, Systems and Structures RILEM proposes to classify mineral additives from industrial waste by assessing their best applicability, primarily in terms of pozzolanic and hydraulic activity.

Microsilica is an ultradisperse material consisting of spherical-shaped particles, obtained in the process of gas cleaning of furnaces in the production of silicon. The main component of the material is amorphous silicon dioxide.

Microsilica is supplied in saleable forms and is labeled accordingly: compacted, MCU-90, MCU-95.

The numerical index in the labeling indicates the minimum allowable amount of silica (SiO_2). Technical specifications TS 5743-048-02495332-96 for condensed condensed microsilica are presented in Table 9.

Table 9 – Specifications for condensed microsilica

Name of indicators	Standards for grades of condensed compacted microsilica	
	MCU-90	MCU-95
Appearance	Fine-grained powdery material of gray color with aggregate size up to 0.5 mm	
Mass fraction of water, %, not exceeding	5	5
Mass fraction of loss on ignition, %, not exceeding	5	5
Mass fraction of silicon dioxide (SiO_2), %, at least	90	95
Mass fraction of sulfur dioxide, %, not exceeding	0.6	0.6
Grinding fineness (specific surface area), m/g, at least	12	12
Bulk density of microsilica dry forms, kg/m^3	80-500	280-500

The chemical composition of microsilica is presented in Table 10.

Table 10 – Chemical composition of microsilica

Manufacturer	Content, %						
	SiO_2	C	Fe_2O_3	Al_2O_3	CaO	Impurities	other
Tau-Ken Temir, LLP	96.9	1.82	0.05	0.16	0.22	2.84	0.78

3. Results and Discussion

In order to study the effect of modifiers on the setting time of cement batter and structure of concrete, several concrete formulations were developed as shown in Table 11.

Table 11 – Experimental compositions of concrete mixtures

No.	Composition, kg/m^3							
	Portland cement	Sand	Crushed stone	Water	Superplastisizer C-3	Microsilica	Hydrophobicizer of vegetable oil soapstock	Hardening accelerant sodium sulphate
1 (Reference)	450	660	1060	167	3.6 (0.8%)	-	-	
2	455.5	650	1060	167	3.6 (0.8%)	4.5 (5%)	-	5.4 (1.2%)
3	382.5	650	1060	167	3.6 (0.8%)	67.5 (15%)	-	6 (1.3%)
4	382.5	650	1060	167	3.6 (0.8%)	67.5 (15%)	2.1 (0.46%)	9 (2%)

In the course of laboratory studies it was possible to obtain the most optimal ratio of aggregates, filler and modifiers in relation to the mass of binder, the distribution of which is as follows: Portland cement – 382.5 kg/m^3, Crushed stone – 1060 kg/m^3, Sand – 650 kg/m^3, Water – 167 kg/m^3 at W/C of 0.43; Microsilica – 67.5 kg/m^3 (15%); Superplastisizer C-3 – 0.5…0.9% (in fact – 0.8%); Soapstock – 0.2…0.5% (in fact – 0.46%); Sodium sulphate – 1…2% (in fact – 2%).

To determine the effect of the complex of modifying additives on the processes of structure formation, the influence of their dosage on setting time was studied.

Table 12 shows the results of cement dough tests to determine the normal density and setting time.

Table 12 – Effect of complex modifiers on normal density and setting time of cement dough

No.	Composition	Quantity by weight of cement, %	Normal consistency, %	Setting time, h	
				Start	End
1	Superplastisizer	0.8	22.2	4.1	6.1
2	Superplastisizer +Sodium sulphate	2 (0.8+1,2)	21	3.8	5.6
3	Superplastisizer +Sodium sulphate	2.1 (0.8+1.3)	20	2.6	3.5
4	Superplastisizer + Soapstock+Sodium sulphate	3.26 (0.8+0.46+2)	18.5	2.3	3.1

From the data obtained, it can be seen that the normal density of cement dough decreased from 22.2 % to 18.5 % when the dosage in composition No. 4 was changed. It is also observed that the onset of setting time decreased from 4.1 hours to 2.3 hours and the completion accelerated from 6.1 hours to 3.1 hours. Based on the results obtained, it is assumed that the optimum balance in dosage of superplasticizer C-3 and curing gas pedal sodium sulfate has been found. Applying only Superplastisizer (C-3) separately, it is impossible to simultaneously correct and comprehensively affect the normal cement dough density and setting time.

It should be noted that the acceleration of the beginning and end of setting time is of great importance for the use of complex modifiers of concrete during repair works.

According to the results of the experiments shown in Table 13 and Figure 2, it can be observed that the densest structure is characterized by the composition No. 4 containing Microsilica, Superplastisizer, Soapstock and Sodium sulphate with an open porosity of 6.1% and corresponding water absorption of 2.2%.

Also, from the obtained experimental results it can be concluded that Superplastisizer more densely packs different-sized grains, and in the process of cement hydration mineral additives and Microsilica react with other materials. All these processes lead to the fact that the surfaces of particles are enveloped by a thin layer of hydration products and the whole structure is united into a single whole.

Table 13 – Main microstructure indicators

No. of compositions	Age of concrete, day	Average density, kg/m^3	Water absorption of concrete, % by weight	Overall porosity (P_o), %	Volume of open pores, P_{op},%	Average size index of capillary pores, λ_2	Uniformity dimensions index of open capillary pores, α
1	28	2140	4.2	16.8	9.0	0.74	0.61
2		2188	3.8	16.3	7.4	0.60	0.68
3		2260	2.9	13.2	6.4	0.54	0.76
4		2272	2.2	12.8	6.1	0.56	0.77

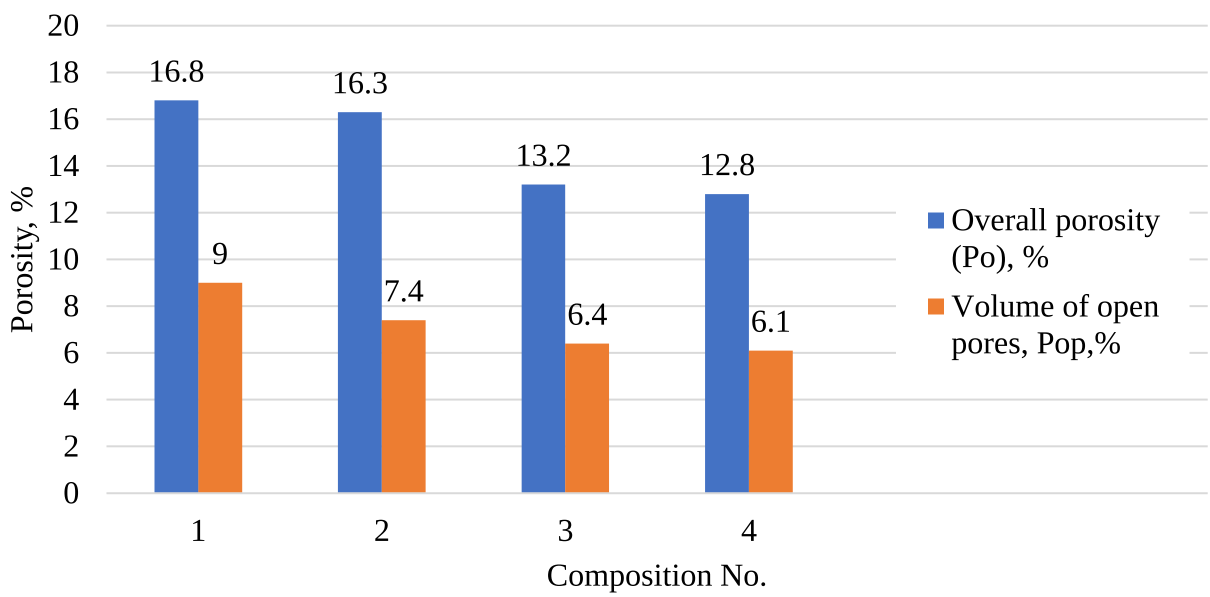

Figure 2 – Effect of modifying additives on density indices of repair concrete structure

It should be noted that the composition modified by the complex of additives has a lower capillary porosity in relation to the control composition, and, therefore, has less shrinkage, which is also an important property for repair mortars.

Determination of shrinkage deformation and corrosion resistance of the developed modified MZB formulation.

Before the developed modified concrete composition is the task of using it mainly in the repair of structures, which means that the prepared concrete mortar will need to be applied in layers with a thickness of 10 ... 40 mm. Due to the relative thinness of the layers there is a high probability of formation of shrinkage cracks in them. Repairs made in this way will be short-lived and will not provide proper protection of the structure from corrosion and destruction. In order to determine the shrinkage performance of concrete, the compositions below were tested:

– the reference composition - plasticized concrete of class B40 on coarse aggregate (crushed stone 5-20 mm), without additional modifiers;

– composition No. 1 – plasticized FGC class B40, without additional modifiers;

– composition No. 4 – developed by FGC for repair, with a modified composition (Microsilica + Superplastisizer C-3 + Soapstock + Sodium sulphate).

The results obtained are shown in Table 14.

Table 14 – Shrinkage strain values of compositions

Curing age, days	Reference composition		Composition No. 1		Composition No. 4	
	Shrinkage		Shrinkage		Shrinkage	
	Absolute, mm	Relative, %	Absolute, mm	Relative, %	Absolute, mm	Relative, %
1	0.119	0.09	0.108	0.08	0.064	0.03
2	0.169	0.10	0.153	0.10	0.085	0.04
3	0.200	0.14	0.178	0.12	0.094	0.05
5	0.210	0.17	0.184	0.12	0.097	0.05
7	0.210	0.17	0.188	0.13	0.097	0.05
14	0.220	0.17	0.194	0.13	0.100	0.05
28	0.220	0.17	0.205	0.14	0.100	0.05
42	0.220	0.17	0.205	0.14	0.100	0.05

The results in Table 14 show that compared to the samples of the control mortar and mortar No. 1, the developed mortar No. 4 is the least subject to shrinkage, which allows this concrete to be placed in relatively thin layers. This means that the developed composition can be recommended for use as a repair mortar or even for cladding, since shrinkage cracking is minimized.

As a fine-dispersed mineral additive in the developed composition of FGC is used microsilica from Karaganda plant of Tau-Ken Temir, LLP, which is, in fact, a waste product of ferrosilicon production. Soapstock is also a waste product of Karaganda margarine plant, obtained during processing of vegetable oils. Cement, sand and crushed stone are supplied by local Karaganda producers. All this gives grounds for significant savings on raw materials and transportation costs.

4. Conclusions

The conducted research allows us to conclude that a composition of fine-grained concrete with high performance characteristics has been developed, which is recommended to be used for repair and facing works of residential structures made of concrete and reinforced concrete.

The experimental results confirm that:

– Improvement of characteristics of the developed FGC on the basic physical and mechanical indicators at application in its composition of additives-modifiers of concrete, such as: superplasticizer C-3 on a naphthalene formaldehyde basis, highly active microsilica as a fine-dispersed pozzolanic mineral admixture, hydrophobizing component – soapstock, i.e., a by-product of oil refining and gas pedal of hardening sodium sulfate (Na_2SO4);

– Reduction of cement consumption by 15% (from 450 kg to 382.5 kg);

– The influence of the main factors and the dependence of performance indicators on the physical and mechanical properties of cement stone and the studied FGC was established.

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Received: 12.05.2023

Revised: 06.06.2023

Accepted: 06.06.2023

Published: 30.06.2023

Technobius, 2023, 3(2), 0039, DOI: https://doi.org/10.54355/tbus/3.2.2023.0039

Technobius

https://technobius.kz/

e-ISSN 2789-7338

Article

Enhancing dry mix mortar strength with natural fillers and polymers

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Abstract. Dry mix mortars are becoming more and more popular in the world's building materials market. Therefore, the issue of increasing the technological and mechanical properties of stucco mixes is relevant. The aim of the paper is modification of lightweight dry stucco mixes with fine limestone and perlite as well as with hydroxyethyl methyl cellulose and dispersible polymer. In order to investigate the different mixes, an 18-point experiment was designed. Density, compressive strengths and crack resistance of dry plaster mixes were studied using requirements of standard. Mathematical models were obtained for the compositions as a result of processing the experimental data. The regularities of the fillers' and additives' influence on the properties of the mixes were established, depending on their amount and combination. It was observed that methyl hydroxyethyl cellulose improves the crack resistance and compression strength, and contributes to a slight decrease in density. The crack resistance of plaster mortars changes more than 1.5 times, the most crack-resistant compositions have an average amount of porous fillers.
Keywords: dry mix mortar, cellulose ether, dispersiblolymer, crack resistance, compression strength.

1. Introduction

The modern construction industry places dry mixed mortar amongst the most versatile materials. There are different kinds of dry mortars for specific purposes: masonry mortar, plastering mortar, ground mortar, tile adhesive, insulating mortar, self-leveling, water-proof mortar, repair mortar, wall putty, and so on. Dry mix products provide excellent technical properties meeting the stringent performance requirements. Normally, the composition of the mixes contains a raw materials binder, fillers, and various modifiers are used to give mortars new properties. Currently, the most popular modifiers are cellulose esters [1-2], superplasticizers [3], thickening agents [4], air entraining agents [5], accelerating and retarding agents [6], defoaming agents [7], hydrophobic agents [8], plasticizing agents [7], shrinkage compensation agents [9]. All of the raw ingredients are mixed together proportionally in a special factory and transported to the construction site since the ingredients and their proportions in each product are sometimes very complicated. However, there are a number of advantages of dry mix mortar: they have consistent strength and other properties like adhesion, frost resistance, etc.; mixing time is reduced because it is stirred automatically, pumped, and applied by machine; there is 30% less wastage of sand and cement due to constant correct composition of mixtures; they are easily transported in a simple container; industrial working conditions are improving.

Nowadays, the application of sustainability principles in construction encourages the development of new materials with improved hygrothermal performance [10-11]. Enhancing energy efficiency and sustainability in the built environment is crucial, necessitating the mitigation of energy demand and optimization of energy preservation mechanisms within buildings. Developed nations primarily witness substantial energy utilization attributed to space conditioning (heating, cooling, air conditioning) and lighting systems in both residential and comme’cial constructions. In developed countries, buildings' energy consumption constitutes a substantial quantity, ranging from 20% to 40% of the overall energy consumption, exceeding that of the industrial and transportation sectors in both the EU and the USA [12–16]. Good thermal quality of buildings is necessary to ensure energy efficiency and healthy indoors [17]. Lightweight plasters are inherently characterized by low thermal conductivity and it is possible to improve the thermal performance of the outer walls by using them. To achieve enhanced thermal performance of mortars, lightweight aggregates are integrated into the composition of mortars. Expanded perlite [18], vermiculite [19–21] waste crushed ceramics [22-23], zeolite [24], sintered fly ash [25], pumice [26-27], etc. are typical representatives of porous aggregates.

The main goal of the present study is a comprehensive analysis of the effect of additives and porous fillers and the replacement ratio of expanded perlite by fine limestone in the composition of eighteen specimens of lightweight mortars on their mechanical properties.

2. Methods

2.1 Characteristics of Materials

The components of the mortars tested in this study are the following:

– Additive-free cement M500 mark (PC I-500-N D0), European quality certificate EN-197-1 [28], CEM I 42.5N, specific surface – 300 m^2/kg, fineness 11.3%;

– ground lime, content of CaO+MgO – 73% by weight, water demand 70%, bulk density – 0.5 kg/dm^3;

– Quartz sand European quality certificate EN DIN 12904 [29], density – 2.04 g/cm^3, particle size modulus – 1.1. Content of dust and clay particles is 0.3%, with no clay in the lumps, moisture content – 3.6%, sifted through a sieve of 0.63, SiO_2–99.4%, Al_2O_3–0.35%, Fe_2O_3–0.005±0.01%, CaO–0.28%, MgO–0.16%;

– Ground limestone – shell rock, with specific surface $S_{s.d.}$=400 m^2/kg, sifted through a sieve of 0.63 mm. The chemical and mineralogical composition: SiO_2–2.52%, Al_2O_3+Fe_2O_3–2.02%; CaO–52.1%, MgO–1.32%, SO_3–0.22%;

– expanded perlite sand fraction 0.16–1.05, the porosity of granules 34.6%, average density (including pores) 1.56 g/cm^3, heat conductivity at 25±5°C not more than 0.052 W_t/m°C;

– Water-retaining additive, methyl hydroxyethyl cellulose Tylose MH60010 P4 (Shin-Etsu Chemical, Japan) [30], water-soluble non-ionic cellulose ether a derivative of the natural cellulose, active substance content of cellulose methyl ether: 2-hydroxyethyl ether 90–95%, NaCl ≤ 1.5%, particle size < 125 μm, minimum 90%, viscosity 50000-90000 MPa/sec, water solubility > 10 g/L (20°C);

– Adhesion improving additive, re-dispersible powder Vinnapas 5034N (Wacker Chemie AG, Germany) [31], copolymer powder of vinyl acetate and ethylene, the active substance content being min. 98% of copolymer powder of vinyl acetate and ethylene, particle size > 400 μm.

– Air-entraining additive, wetting, and plasticizing agent Hostapur OSB (Clariant AG, Switzerland) [32]. Active substance content 90–98% of olefin sulfonate and sodium salt, particle size 72μm;

– Water-repellent Vinnapas 8031H (Wacker Chemie AG, Germany) [31], active substance content of a triple copolymer of ethylene, vinyl laurate, and vinyl chloride ≥ 98%, particle size > 400 μm.

2.2 Design of Experiment

The mix design was carried out using the D-optimality criterion [33] of the experiment. The study involved the determination of material property values denoted as *Y* across 18 compositions, using an experiment design with 4 factors and 3 levels. The experiment encompassed variations in the dosages of four key components, expressed in weight parts per 1000 weight parts of the dry mix:

limestone (designated as X_1), perlite sand (X_2), methylcellulose ether (X_3, Tylose MH60010), and dispersible polymer (X_4, Vinnapas 5034N). Other component proportions remained constant. Table 1 provides the natural factor levels (component dosages, represented as X_i) within the normalized range ($X_{i.min} \leq X_i \leq X_{i.max}$).

Table 1 – Levels of composition factors in the experiment – contents of components in 1000 w.p. of dry mix

i	Composition Factors (*X*)	Minimal, Central, and Maximal values		
		$x_i = -1$	$x_i = 0$	$x_i = +1$
1	Mass parts of limestone, X_1	60	80	100
2	Content of perlite, X_2	30	40	50
3	Dosage of Tylose, X_3	1	1.15	1.3
4	Dosage of Vinnapas, X_4	1	1.5	2

The construction of second-order models in this design facilitates a quantitative representation of the gathered data. These models delineate the individual and combined impacts of composition factors on properties denoted as *Y* by use of second-order polynomial experimental-statistical (ES) models, as described in Eq. (1), wherein the coefficients *b* possess specific physical interpretations [34].

$$Y(\boldsymbol{x}) = b_0 + \sum_{i=0} b_i x_i + \sum_{i=1} b_{ii} x_i^2 + \sum_{i<j} b_{ij} x_i x_j \quad (1)$$

where: *b* – parameters (coefficients) to be –stimated; *x* – vector of normalized factors; $x_i = (X_i - X_{0i})/\Delta X_i$, $X_{0i} = (X_{i.min} + X_{i.max})/2$, $\Delta X_i = (X_{i.max} - X_{i.min})/2$; $X_i = x_i \cdot \Delta X_i + X_{0i}$.

2.3 Preparation of specimens and research methods

All raw materials underwent precise measurements using a laboratory scale. Subsequently, thorough blending of all components was performed with a spatula to achieve uniform ingredient distribution. Water was introduced into the dry mixture and mixed for 60 seconds at low speed using a hand mixer. The water quantity was precisely regulated to attain mortars with consistent spread diameters of 16–17.0 cm, by DIN 18555 [35], as illustrated in Figure 1.

Figure 1 – Testing the freshly prepared mixture to evaluate the needed consistency

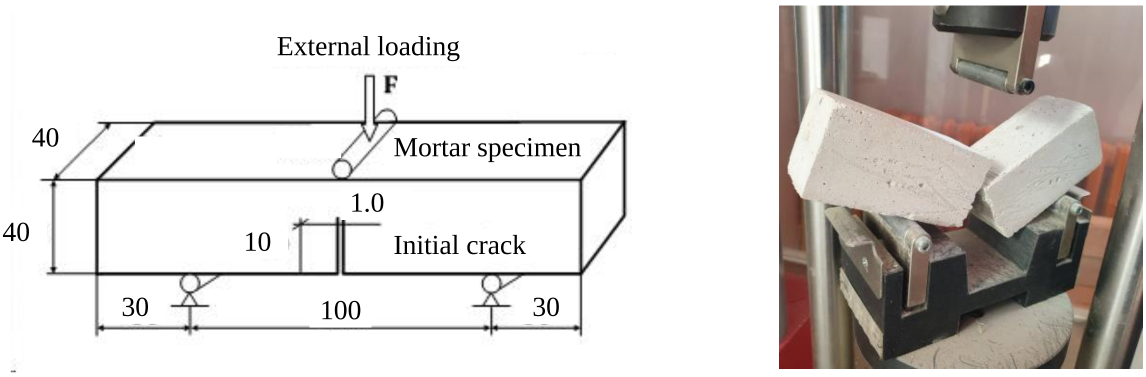
The bulk density was assessed by measuring the dimensions and dry mass of 40 × 40 × 160 mm beam samples, following a 28-day curing period at an air temperature of 20 ± 2 °C and a relative humidity of 65 ± 5%.

The compressive strengths of matured mortar samples were evaluated by the guidelines outlined in EN 1015-11:2020 [36]. For each composition, as per the experiment design, three specimens were subjected to testing. The compressive strength (f_c, MPa) was definite by examining the remnants of the specimens following tensile strength testing, which was conducted using a hydraulic press following the standard protocol.

The crack resistance of the composite was determined by the critical stress intensity factor (K_{1c}) (2), on cracked specimens, on a 3-point test scheme (Figure 2). The dimensions of the specimen are 160 mm × 40 mm × 40 mm and the initial crack length is 10 mm × 1 mm K_{1c} is determined from the Eq. (2):

$$K_{1c} = G_{n.s.}\sqrt{\pi \cdot l} \qquad – \qquad (2)$$

where: $G_{n.s.}$ - nominal stresses (without considering their concentration) in the weakene– section at the tip of the crack at critical load; l - notch length.



a) Three-point notched specimen test b) Specimen after the test

Figure 2 – Three-point test scheme

Nominal stresses in the cross section can be found by the Eq. (3):

$$G_{n.s.} = \frac{(G\cdot M)}{b(h-2)2} \qquad (3)$$

where: M – critical bending moment; b – sample width; h – s“mple height.

3. ”esults and Discussion

The diagram "squares on square" in Figure 3 displays the joint influence of 4 composition factors on density in coordinates of composition factors normalized to dimensionless $-1 \leq x_i \leq +1$.

The results presented in the diagram indicate that the amount of cellulose contributed to a slight decrease in the density of the plaster mortars by increasing the uniformity of the mixture. An increase in the percentage of perlite in most of the factor space of the experiment, naturally, reduced density of the plaster mortar. On the other hand, in mixtures with a large dosage of re-dispersible powder Vinnapas, the amount of perlite practically did not affect density. The effect of the amount of ground limestone (x_1) on density is even more dependent on the dosage of the re-dispersible powder (x_4). However, density is reduced with the increasing amount of limestone in the mix with $x_4=-1$ (about 2 w.p. of Vinnapas), which is the desired technical result. Density reduction is attributed to the porous nature of limestone. Overall, density exhibited nearly 1.3 times. The lowest density recorded at 1046 kg/m^3 at the maximal values of all factors, except for the re-dispersible powder Vinnapas, were applied minimally. The impact of Vinnapas can be elucidated by its effect on the distribution of aggregates within the mortar; it promotes even dispersion instead of surface floating, thereby leading to a more densely packed mortar. Consequently, this has the potential to improve the thermal insulation characteristics not only of the mortar but also of the entire construction.

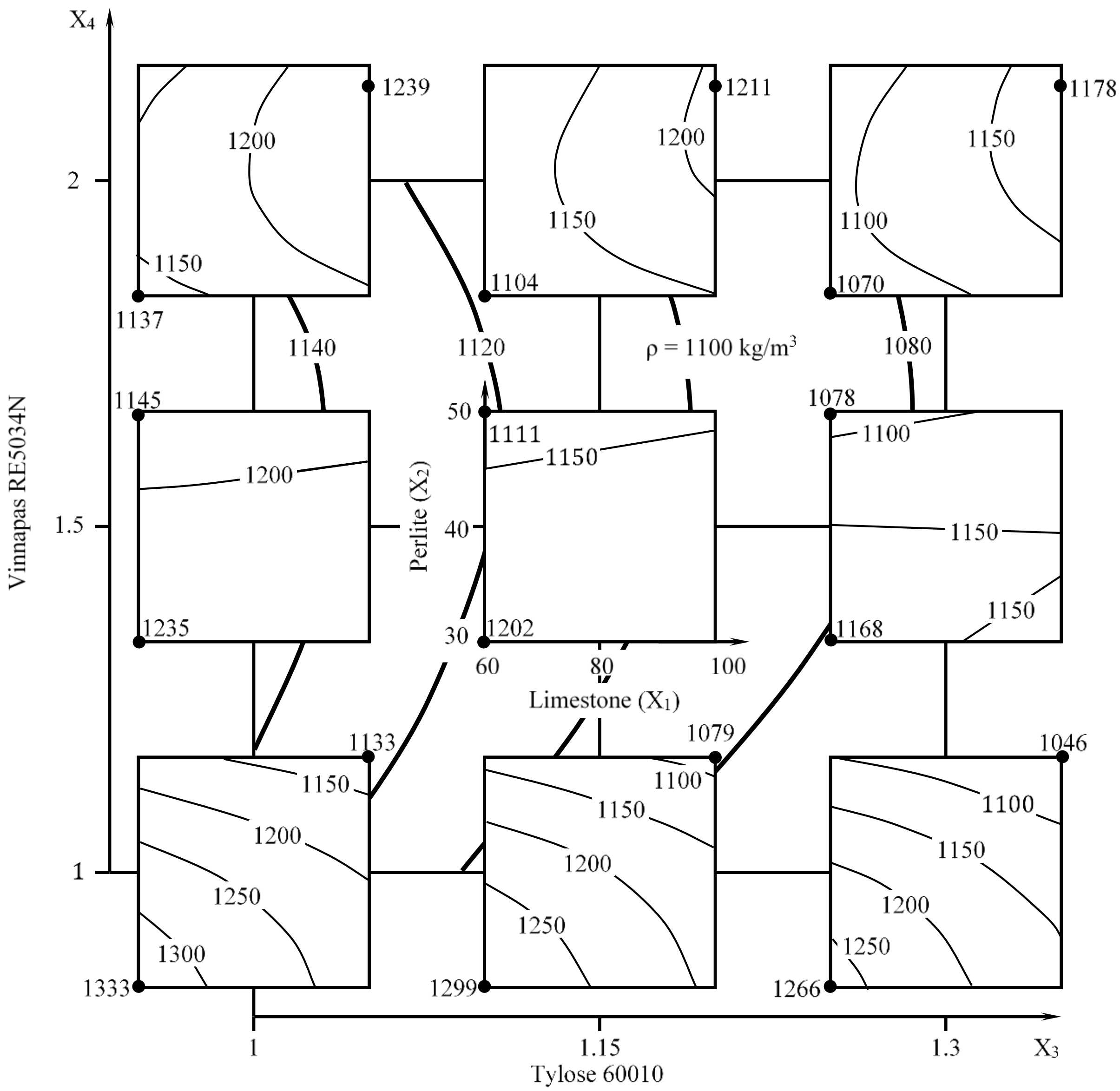


Figure 3 – The density isolines, influenced by varying limestone and perlite contents and changing with Tylose and Vinnapas dosages

The curves in Figure 4 show 1-factor local fields of the properties at fixed values of other factors providing the minimum and maximum level of compressive strength. Looking at the measured data, one can see that the compressive strength of mortars ranges from 4.08 to 8.8 MPa and changes more than twice. In this case, the most durable are the compositions with an average amount of limestone (x_1), at the highest content of perlite (x_2) and Tylose (x_3) and at a rather high dosage of Vinnapas (x_4). The amount of perlite at which the mortars reach maximum strength depends on the dosage of Tylose methyl hydroxyethyl cellulose. This is a positive effect, because some studies have reported that, regardless of the content of cellulose, the mechanical strength of mortar significantly decreased [6-37]. However, with the higher level of x_3 and the higher amount of perlite x_2, the maximum strength can be achieved. The above mentioned effects of the influence of the mineral frame can be explained by a change in the packing of its particles, but it cannot be regarded that the strength of the composite is ensured by low-strength perlite and limestone. An indirect confirmation of the latter stems from the fact that the most durable mortars in the region are obtained with the maximum amount of re-dispersible Vinnapas powder and methyl hydroxyethyl cellulose, that is, factors that affect the cement matrix more.

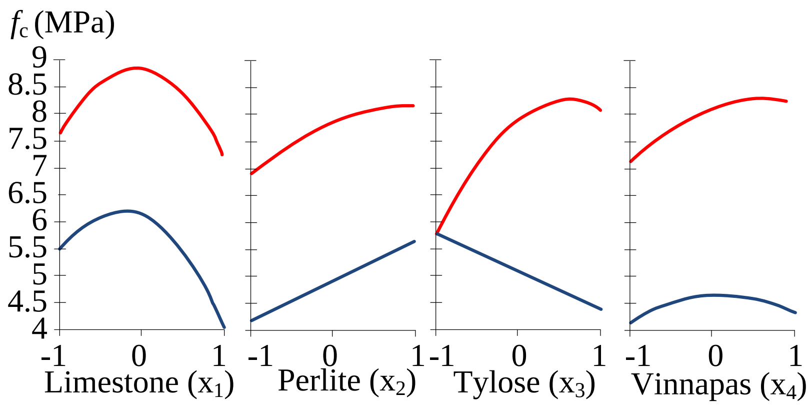


Figure 4 – The individual impacts of component content on compressive strength (f_c) in regions characterized by both minimum and maximum values

Mineral binders, providing high compressive strength, cannot always provide good bending tensile strength. Inhomogeneities, which include aggregate and filler grains, also affect the formation and development of critical microcracks and significantly change the pattern of concrete destruction.

Figure 5 shows the crack resistance of investigated mixes. The curves in Figure 5 show that under the influence of varying compositional factors, the crack resistance varied from 0.12 to 0.16 MPa. The most crack resistant (K_{1c} ≈ 0.16 MPa·m$^{0.5}$) compositions are the ones with an average amount of limestone x_1≈0. The effect of both limestone and redispersible powder was quite apparent in the amount range of –1 to 0 with the crack resistance growing, while a further increase of mentioned additives leads to a decrease of K_{1c}.

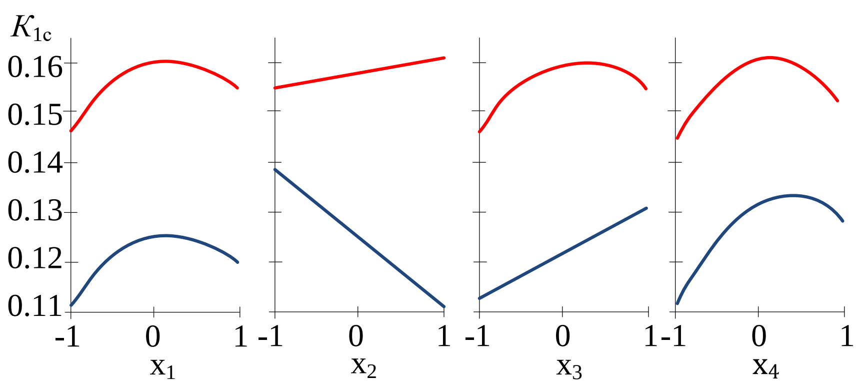


Figure 5 – The individual impacts of component content on crack resistant (K_{1c}, MPa·m$^{0.5}$) in regions characterized by both minimum and maximum values

Changing the amount of perlite did not have a noticeable effect on K_{1c}. While cellulose ethers can slightly increase crack.

4. Conclusions

To achieve the reduction of the heat transfer coefficient, the effects of two types of porous fillers and two polymer modifiers on the technological and mechanical properties of dry mix mortars were evaluated and discussed. The following conclusions can be drawn as the main outcomes of the current study:

– The effectiveness of additives varies widely, depending on the composition of the mortar. The application of expanded perlite resulted in a decrease in density. However, as compared with the mixtures with a large dosage of re-dispersible powder Vinnapas, the amount of perlite did practically not affect density;

– Re-dispersible powder induces the aggregate to be evenly distributed inside the mortar. This leads to a density increase due to denser packing of the mortar;

– The inclusion of methyl hydroxyethyl cellulose into the lightweight plasters improved the crack resistance and compression strength. This effect can be attributed to the fiber-bridging action by cellulose molecules;

– In general, when the content of porous fillers was no more than the medium amount, this had the best improvement effect on the mechanical properties).

Acknowledgments

The present research is financed by the Un“versity North scientific support grant of 2023 named "Rheological and mechanical properties of”plaster mortars blended mineral and polymer additives".

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Received: 29.05.2023
Revised: 16.06.2023
Accepted: 17.06.2023
Published: 30.06.2023

Technobius, 2023, 3(2), 0040, DOI: https://doi.org/10.54355/tbus/3.2.2023.0040

Technobius

https://technobius.kz/

e-ISSN 2789-7338

Article

Properties of modified bitumen in road construction

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Abstract. The increase in the construction of asphalt concrete roads in Kazakhstan has led to the development of research on raw materials in this industry. Bitumen binder as the main component affecting the deformation properties, as well as the physical and mechanical properties of coatings, in general, is of the greatest interest to builders. The analysis of scientific sources in the field of the application of various modifiers of bitumen binders is carried out. Polymer-based additives have been widely used in road construction. The use of half-dimensional additives contributes to the improvement of asphalt concrete indicators: strength properties, water resistance, deformative stability of the coating at high and low operating temperatures, wear resistance, and resistance to rutting, which generally increases the repair time and contributes to a corresponding reduction in operating costs. This article presents a study of the properties of bitumen modified with an additive based on rubber crumbs. Indicators such as penetration depth and softening temperature by the ring and ball method are investigated in the article. The samples with different content of additives were taken: control sample without additives, and samples with the addition of modifiers in the amount of 5%, 10%, 12%, and 15% out of bitumen weight. The highest viscosity is shown by the sample with a modifier of 12%. Bitumen samples with a modifier content of 12% and 15% correspond to the standard indicators regarding softening temperature.
Keywords: bitumen, modifier, polymer additive, crumb rubber, bituminous binder.

1. Introduction

Bitumen modifiers for asphalt concrete are used to improve their basic properties such as strength, water resistance, fatigue resistance, and durability. Several types of modifiers can be added to asphalt bitumen such as polymers, rubber particles, plasticizers, adhesion additives, nanotechnology-based additives, and modified bitumen itself [1]. For example, polymers are added to the bitumen to improve its resistance to ultraviolet radiation and fatigue with the potential to increase the strength and durability of the asphalt concrete [2]. Rubber particles are added in the form of crumb rubber or powder from recycled tires to improve the elasticity and fatigue resistance of the asphalt concrete [3]. Plasticizers improve bitumen performance at low temperatures by increasing flexibility. Adhesion additives improve adhesion between bitumen and mineral aggregates, thereby increasing the strength of the asphalt concrete [4]. Modified bitumen is already modified with polymers or other additives to provide specific properties related to temperature resistance or chemical resistance. Nanotechnology-based additives use nanoparticles to improve bitumen properties [5]. All of the above modifiers can be combined depending on the project. However, the main purpose of these modifiers is to create durable and stable asphalt pavements [6].

Polymers were found to be an effective bitumen modifier despite relatively low mixing and compaction temperatures [7]. Tests have shown positive effects on the physical properties of bitumen: an increase in stiffness after modification, meaning that pavements become more resistant to permanent deformation and become more resistant to heavy loads.

Another additive is a composite thermoplastic elastomer, which is used to modify bitumen to improve resistance to plastic deformation, increase the shear stability of asphalt concrete, and reduce the effect of temperature during the technological process when working with bitumen [8].

In this paper, we consider the possibility of using modified crumb rubber as an additive for bituminous binder [9]. It is assumed that the use of this modifier will increase the deformation resistance of the material [10].

2. Methods

For testing of materials, samples of bitumen of Pavlodar petrochemical plant grade BND 70/100 and modified rubber crumb, were obtained as a result of processing of automobile tires. Tests of material samples were carried out by the approved methods in compliance with the current regulatory documents in the Republic of Kazakhstan.

2.1 Sample preparation

Bitumen sampling was carried out by ST RK 1809-2008 [11]. Two-point samples of bituminous mastic weighing at least 0.5 kg were selected and poured into the mold for casting.

2.2 Method for determining needle penetration depth

Bitumen samples to determine the penetration depth of the needle are taken by the requirements of Interstate standard 2517-2012 [12]. The test sample is heated to a mobile state, in the presence of moisture, it is dehydrated by heating to a temperature 90 °C above the softening temperature, but not higher than 160 °C. The heating time should not exceed 30 minutes (Figure 1).

Figure 1 – Heating samples

Further, the bitumen samples are filtered, poured into two penetration cups, and thoroughly mixed until the air bubbles are completely removed (Figure 2).

Figure 2 – Pouring bitumen samples into penetration cups

The studied bitumen samples are cooled in air at a temperature of 15-30 °C for 60-90 minutes. After cooling, the cups with the samples are placed in a bath.

Bitumen consistency is determined by the method of determining the depth of needle penetration according to ST RK 1226-2003 [13]. The essence of the method is to measure the depth of penetration of the penetrometer needle into the bitumen sample under certain conditions: a given load, temperature and time interval (Figure 3). The accuracy of the needle penetration depth is 0.1 mm.

Figure 3 – Testing samples in a penetrometer

2.3 Determination of the softening point by the ring and ball method

Bitumen sampling is carried out by Interstate standard 2517-2012 [12]. Bitumen is poured with a small excess into two stepped rings placed on a plate lubricated with dextrin. Samples must be dehydrated and free of air bubbles (Figure 4).

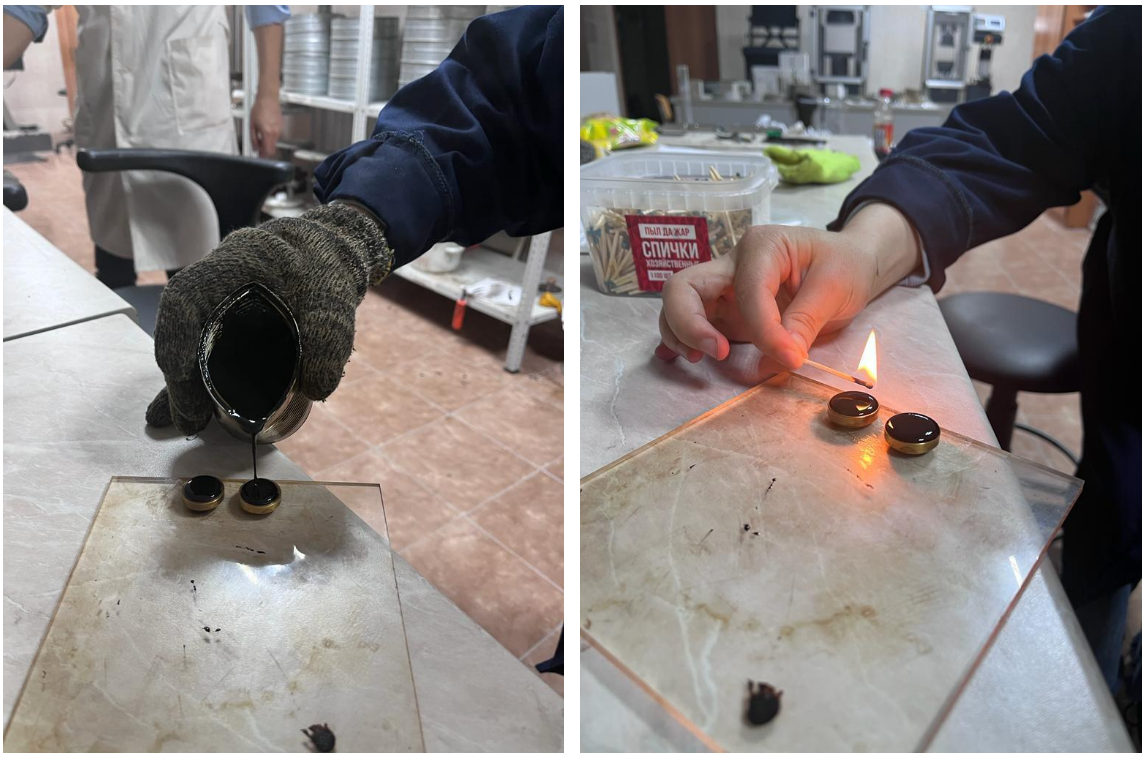

Figure 4 – Pouring bitumen samples into stepped rings

After cooling the sample rings for 30 minutes, the excess bitumen is cut off with a heated knife (Figure 5).

Figure 5 – Cutting of excess bitumen

When testing samples, rings with bitumen are placed in the holes of the upper plate of the device. A tripod with test samples in rings is placed in a bath filled with distilled water with a temperature of 5±1 °C. After 15 minutes, the tripod is removed from the bath, a steel ball is placed on each ring in the center of the surface, pre-cooled in the bath to a temperature of 5±1 °C, and lowered back into the bath on the heating device. The water temperature after the first three minutes should rise at a rate of (5.0 ± 0.5) °C per minute. For each ring and ball, the temperature at which the bitumen squeezed out by the ball touches the light beam of the device is noted (Figure 6).

Figure 6 – Testing of samples

3. Results and Discussion

In order to determine the effectiveness of using modifier for bitumen, samples with different content of additive were taken: control sample without additive, samples with addition of modifier 5%, 10%, 12% and 15% of bitumen weight. The tests carried out on laboratory equipment showed different results. Processing of results and comparative analysis of indicators was carried out in accordance with the requirements of the National standard [14]. According to Table 2 of this standard, physical-mechanical and performance indicators of binder samples were evaluated for PMB (polymer modified bitumen) 70/100 grade.

3.1 Needle penetration depth

The results of the tests carried out by this method allow us to judge the consistency of bitumen binders. High accuracy of the results is provided by calibrated laboratory equipment (Figure 7).

Figure 7 – Testing of the sample on the Infratest penetrometer

The obtained indicators of the penetration depth of the needle of the test device into the bitumen samples are shown in the following graph (Figure 8):

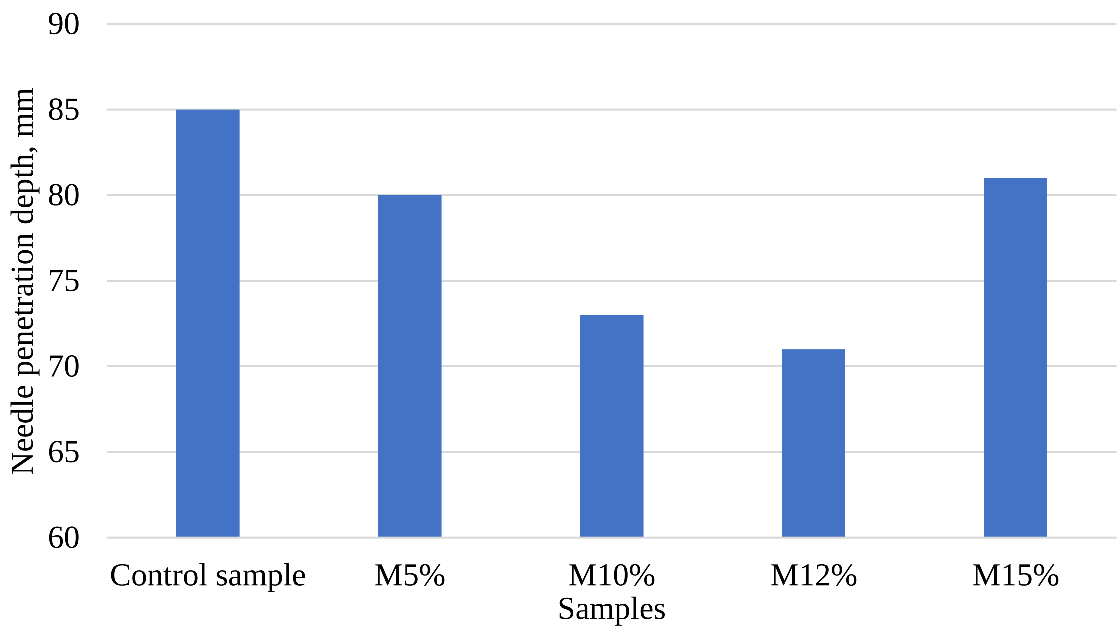

Figure 8 – Indicators of penetration of bitumen samples

As we can see, the samples of the modified binder have sufficient viscosity for mobility. The highest indicator is a sample with a modifier content of 12 %.

3.2 Determination of the softening temperature of bitumen

Determination of the softening temperature of bitumen characterizes the upper temperature limit of its application. This indicator also indirectly characterizes the adhesion of bitumen and is related to the nature of its components. The data obtained experimentally are summarized and shown in the Figure 9.

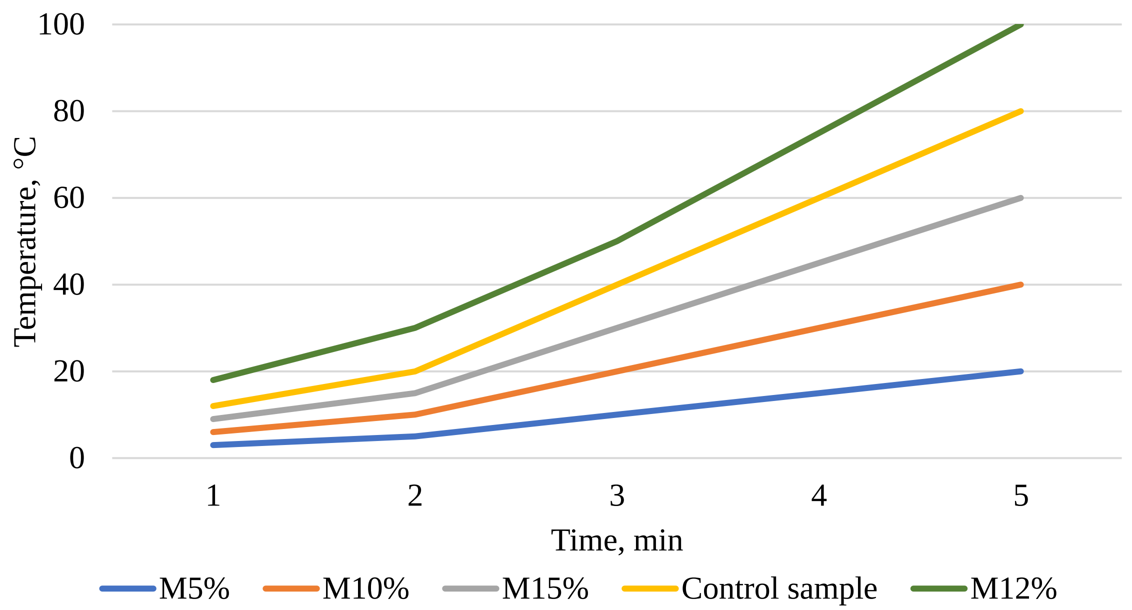

Figure 9 – Indicators of the softening temperature of samples by the ring and ball method

Taking into account the test results obtained, bitumen samples with a modifier content of 12% and 15% correspond to the standard indicators [15].

4. Conclusions

The obtained test results of bitumen samples with different percentages of a modifier based on rubber crumbs indicate the effectiveness of the additive on the physical and mechanical properties of the bitumen binder. The problem of recycling worn-out car tires is of great ecological importance all over the world. The accumulation of worn tires pollutes the environment, and their uncontrolled burning has an irreversible impact on the environment. According to many scientific studies, the use of rubber crumbs in the production of asphalt concrete solves the issues of noise reduction, and also increases the strength properties, water resistance, deformative stability of asphalt concrete pavement at high and low operating temperatures. The conducted research on the use of rubber crumbs as a modifier of bitumen binder has shown the effectiveness of its use.

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Received: 05.06.2023
Revised: 16.06.2023
Accepted: 16.06.2023
Published: 30.06.2023