

# PROSPECTS FOR EVALUATING THE DAMAGEABILITY OF ASPHALT CONCRETE PAVEMENTS DURING COLD RECYCLING

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**Abstract.** The article considers improvement of the methodology for accounting for the degradation of asphalt concrete working in the upper layers of the pavement. Development of recycling technologies for road structures is an

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ongoing process; it allows reaching a higher quality of reclaimed materials and using them for subsequent construction of structural layers, including the upper layers without the protective ones, as well as during repair and reconstruction of roads of various technical categories. At the same time, the system of pre-project assessment (diagnostics) of the state of asphalt concrete pavements cannot be considered optimal and effective because the determined indicators demonstrate that, firstly, various surface and structural defects are present, and, secondly, that the indicators mentioned above are more relevant to the road structure as a whole. The joint handling of the theoretical and experimental data allows concluding that damageability level depends on the physical, mechanical and structural properties, the main being maximal structural strength and the number of elastic bonds involved in the deformation process. A variant of modelling of asphalt concrete damageability depending on the work capacity is proposed, when the reduced amount of dissipated energy is replaced with sufficient accuracy for practice by the ratio of the actual number of load application cycles (freezing and thawing cycles) to the limit. A correlation between the level of damageability and the kinetics of changes of the inter-pore space of asphalt concrete under the influence of strain (temperature, climatic factors) has been established. Results allow fixing (predicting) the level of damageability by measuring the level of water permeability. The research methodology and equipment for implementation thereof was developed earlier, it can be effectively used at the stage of pre-project diagnosis.

**Keywords:** asphalt concrete, damageability, maximal structural strength, pavement recycling, permeability, water resistance, work capacity.

## Introduction

The current road construction and repair industry is characterized by constantly growing material consumption primarily caused by an increase in the volume of freight transport and increase in the number of private vehicles. Growing level of traffic requires an increase in the durability of road structures under construction and road traffic safety, which is clearly associated with an increase in costs, including the volume of used basic road construction materials – crushed stone, sand, bitumen, cement, etc.

Repair and reconstruction of road pavements is still associated with the accumulation of solid waste from the processing of structural layers, both with organic and inorganic binders and without them, in many countries of the world. The volume of repair work is constantly increasing, therefore, the load on the environment is steadily growing as well (Pasetto, Pasquini, Giacomello, & Baliello, 2017; Pei, 2017; Sui, 2012).

In these conditions, different technologies of recycling of structural layers of pavements and the whole road structures are used more widely.

Recycling of the existing pavement (pavement recycling) includes various methods (ways) of their modification that provide for

reconstruction, repair and maintenance, homogeneity recovery of road pavements with damages and defects, as well as for the recovery of their bearing capacity depending on climatic and traffic conditions. The main feature of this type of construction technology is reuse of materials from the existing structures, which in most cases leads to preservation of material and monetary resources and energy, as well as to the increased environmental friendliness of the projects. At the same time, it becomes possible to preserve the thickness and geometry of the road structure to the degree possible. These technologies can be used for reconstruction, repair, and maintenance of highways with high traffic, as well as regional (local) roads of low categories.

Currently, the most effective and common technologies for reconstruction, repair, and maintenance with the reuse of road structures materials are:

- 1) asphalt plant hot recycling;
- 2) hot in-place recycling (surface recycling including thermal profiling, repaving, remixing);
- 3) cold central plant recycling;
- 4) cold in-place recycling;
- 5) full-depth reclamation.

The development of technologies for reprocessing road pavements is ongoing. Both recycling equipment and machines are being improved, as well as the scientific approaches to obtaining regenerated materials for the construction of new structural layers with increased reliability and durability characteristics, which allows them to be used more widely for the repair and restoration of the wearing courses and upper layers of pavements for roads of various technical categories.

Almost all preliminary studies of the condition of road pavements carried out during the implementation of recycling projects include determination of:

- surface defects;
- structural defects (damage);
- functional features.

The following characteristics are basically determined in assessing the road surface condition:

- evenness (IRI);
- pavement strength (elastic deflection);
- pavement defectiveness.

Based on the results of appropriate processing of the received data, a conclusion is made about the feasibility of carrying out repairs with the selected type. So, for example, by plotting the accumulated amounts by elastic deflection value, typical sections of road pavement are set, subject, if necessary, to various recycling options (Wirtgen GmbH, 2012).

The defined characteristics indicate, firstly, the presence of various surface and structural defects, and, secondly, they are more related to the whole road structure (elastic deflection, evenness, etc.). All of that suggests that the specified set of diagnostic procedures is insufficient for establishing the most optimal time to perform restoration measures by various methods of regeneration for the upper layers (wearing courses) of pavements. At the same time, when assessment is made based on the visual fixation of defects, it often leads to the situation that repairs (recovery) are already required not only for the surface layer but also for the lower ones, up to the ground of the roadbed. In this case, the one-time cost of performing the repairs increases significantly, and it is impossible to restore the bearing capacity of road pavements by thin-layer recycling.

With that said, in view of modern realities concerning the development of technologies for thin layer (up to 5 cm) regeneration of road surfaces, it is reasonable to form a new methodological and technical base for assessing the state of the upper structural layers.

Previous studies (Wang & Shi, 2017) have allowed us to develop a methodology for assessing water permeability of road asphalt concrete surfaces based on recording changes in their temperature when drying after wetting, which is still used today for the evaluation in the cold recycling process, as an optional one.

According to the results of the study, the following dependency was obtained in order to determine the water permeability of road asphalt concrete surfaces based on the calculation of the degree of temperature change  $U$  depending on the current temperature of the “waterproof” surface segment  $T_0$ :

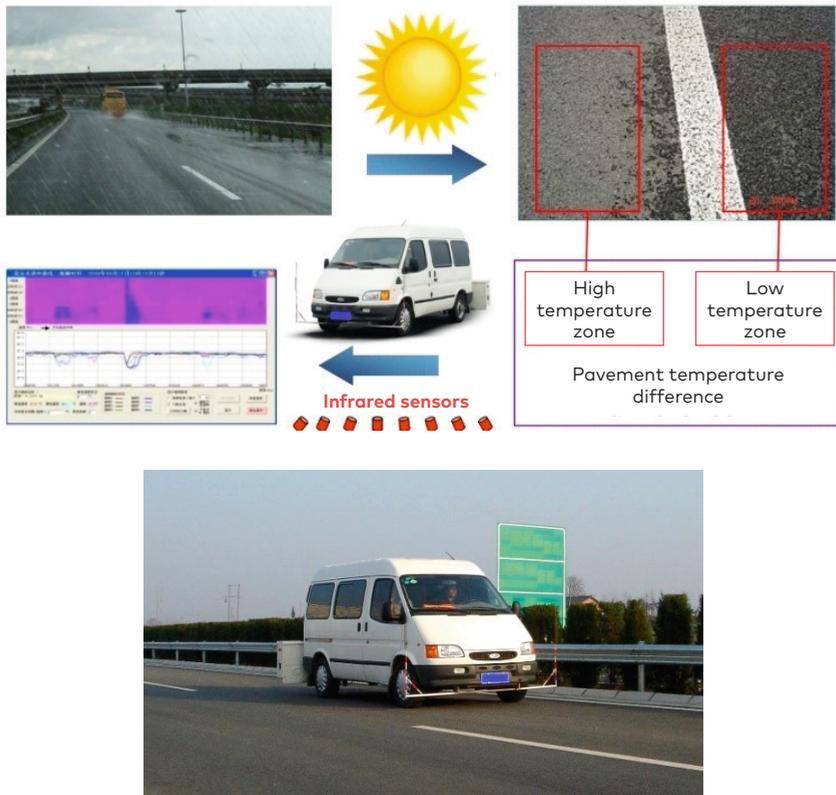
$$U = k \frac{\Delta T}{T_0} \cdot 100, \% , \quad (1)$$

where  $k$  – the correction coefficient.

At the same time, the test is implemented at high speed (up to 80–100 km/h), which allows for effective use of this method for pre-project diagnostics of the road asphalt pavements condition within the preventive measure assignment (Figure 1).

A wider use of this method is hindered by the lack of adequate evaluation of water permeability index considering the presence of defects of various severity in the structure of asphalt concrete pavement that affect its durability and reliability, that is, if we take the intra-structural damageability into account, and not just the peculiarities of changes in the interpore space.

In this regard, when performing this study, attempts were made to obtain the dependencies that link the indicators of changes in the



**Figure 1.** Assessment of water permeability of road asphalt pavements

interpore space of asphalt concrete road surfaces with the value of changes in intra-structural damageability, which directly affects the development of defects and leads to reduction of structural layer durability.

In order to solve this problem, the following steps were consistently performed:

- assessment of existing methods for evaluating the degree of degradation of properties, including those based on own research results performed earlier;
- evaluation of possible criteria for accounting for the reduction of structural strength and the relationship with the initial indicators of physical and mechanical properties;
- evaluation of correlations between the requirements for accounting for structural strength reduction and indicators of changes in the interpore space;

- development of primary methodological bases and dependencies for the implementation of a rapid method for assessing the intra-structural damageability of asphalt concrete pavements in the process of evaluating the prospects and effectiveness of cold recycling.

## 1. Damageability of road asphalt concrete pavements

The mechanical strain and climate factors cause breaks in intermolecular bonds, emergence of microcracks, and dislocations in the structure of asphalt concrete of the pavements. The repeated strain application and long-term exposure to it increase the level of damageability accumulation; its level becomes critical at a certain point and leads to the destruction of asphalt concrete.

In the 1950s, the continual theory of description of damageability and destruction in creep conditions for uniaxial stress state was formulated (Kachanov, 1974; Rabotnov, 1987). The main idea of this approach was to introduce a dimensionless abstract quantity that characterizes damageability as a feature of the development of spatial damage (microcracks, porosity) in the material. In this case, the process of appearance and distribution of microdefects is random and continues throughout the entire volume of the sample under consideration. Subsequently, the theory developed and became more complex (Chaboche, 1988; Lemaitre, 1985; Odqvist & Hult, 1962; etc.). Within this theory, material's mechanical or thermodynamic characteristics that take kinematic and/or dynamic spatial changes into account are most often chosen as the criterion for material destruction.

In the general case, the classical equations for describing the damage model obtained by Kachanov (1974) and Rabotnov (1987) can be written in the following form:

$$\frac{d\psi}{dt} = C \left( \frac{\sigma}{1-\psi} \right)^m, \quad (2)$$

$$\frac{d\psi}{dt} = C \left( \frac{\sigma}{1-\psi} \right)^m \psi^\beta, \quad (3)$$

where  $\psi$  – damageability;  $t$  – time;  $C$ ,  $m$ ,  $\beta$  – factors that depend on material properties;  $\sigma$  – stress.

Based on Eqs. (2) and (3), assessment of material properties and the achieved damage level can be performed using  $\psi$  scalar determined from the generalized condition:

$$\psi = f\left(\frac{R_t}{R_0}\right), \quad (4)$$

where  $R_t$  – the material properties during mechanical stress;  $R_0$  – initial material properties.

If  $\psi = 1$ , no damage to the material structure is present; if  $\psi \rightarrow 0$ , complete degradation occurs.

Many researchers have studied the development of damageability of composite road materials. This field has been developing with sufficient success until now.

In the 1990s (Verenko, 1993), the influence of the composite road material structure on the processes of damageability accumulation was analysed using the ratio of elastic and viscous bonds established as a factor that takes external influence (temperature and loading mode) into account.

If the proportion of elastic bonds responsible for the state of asphalt concrete denote by  $n_r$  scalar and viscoplastic - by  $n_v$ , then next condition must be met:

$$n_r + n_v = 1. \quad (5)$$

In accordance with this theory and from the point of view of the mechanics of deformation and destruction, the structure of composite road material is represented as a phenomenological model with a complex set of elastic, viscous, and plastic bonds alternating in sequential and parallel schemes. Each bond of the model has its own mechanical characteristics, resulting in the material as a whole having a range of elastic-viscoplastic properties. Changes in temperature, load value, and loading mode lead to the replacement of some bonds with different ones (elastic by viscous and vice versa). Due to the fact that a mathematical description of this type of model is almost impossible, it is used as a physical analogue of the processes occurring in the material.

If only elastic bonds are deformed, there is a complete reversibility of deformation and destruction occurs by the mechanism of a brittle body, it is not affected by the load time. Conversely, viscoplastic bonds are responsible for the appearance of residual deformations and are affected by temperature and loading time.

The accumulation of damageability in the structure of asphalt concrete can occur in two ways: breaking of elastic bonds and reaching the limit deformation of viscoplastic bonds.

Regardless of the composition and structure, asphalt concrete with an equal number of elastic (viscoplastic) bonds is characterized by the same relaxation ability, relaxation rate, the coefficient of transverse deformation, and the ratio between the amount of dissipated and accumulated energy. It may be explained by the fact that at the same number of bonds, slowing down the relaxation rate implies a decrease in the relaxation rate constant by the same amount. This is the most important feature of the behavior of the studied materials, which allows us to develop new approaches to the analysis of their properties (Li, Liu, Shi, Veranko, & Zankavich, 2017).

As a result of deformation of viscoplastic bonds, the applied energy is completely dissipated. It can be theoretically assumed that the ratio of  $n_r$  and  $n_v$  is determined by the ratio of the dissipated energy to the applied energy. In this case, the number of  $n_r$  and  $n_v$  depends primarily on the relaxation properties of asphalt concrete and the time of action of the load. The number of elastic bonds involved in the deformation process can be determined from the following dependency (Li *et al.*, 2017; Verenko, 1993; Verenko & Makarevich, 2010):

$$n_r = \frac{E_t}{E_c} = \left( \frac{R_t}{R_c} \right)^{\frac{1}{m}}, \quad (6)$$

where  $E_t$  and  $R_t$  are the relaxation modulus and the strength of asphalt concrete under specific conditions of strain and temperature, MPa;  $E_c$  and  $R_c$  – maximum values of the relaxation modulus and strength over the entire range of temperature and loading rate (time), MPa;  $m$  – the coefficient depending on the properties (type) of asphalt concrete.

The value of the  $m$  factor for asphalt concrete varies from 0.75 to 0.90 and, first of all, is determined by its modulus of elasticity (stiffness). The higher the modulus of elasticity (stiffness), the higher is the  $m$  factor, which can be assumed to equal 0.8 for comparative calculations.

The value of  $n_r$  changes during creep and relaxation, since  $E_t$  is a function of the stress and time of action of the load. This leads to a complex influence of creep and relaxation processes on the properties of asphalt concrete and fracture mechanics.

When working asphalt concrete in the elastic stage ( $n_r \rightarrow 1$ ), its strength will equal the maximum over the entire temperature range (load time) and correspond to  $R_c$ . Since the number of cycles before failure depends on the ratio of acting stresses to strength, the higher the  $R_c$ , the greater is the cyclic stability of asphalt concrete in the elastic stage of work, and the higher is the level of damageability in the material that can be achieved at the time of failure. Consequently, the  $R_c$  value can serve as a criterion for cyclic durability under constant stress in the elastic stage of work (Li *et al.*, 2017).

The maximum asphalt concrete strength over the entire temperature range (loading exposure time) can be calculated using the results of a split test, or indirect tensile test, of the standard cylindrical samples according to the following dependency (Verenko, 1993; Verenko & Makarevich, 2010):

$$R_c = \frac{\bar{R} \cdot \lg \frac{V_1}{V_2}}{\lg \frac{V_1}{V_2} - \lg \frac{R_1}{R_2}}, \quad (7)$$

where  $V_1$  and  $V_2$  are the deformation rate during two tests ( $V_2 > V_1$ ), mm/min;  $R_1$  and  $R_2$  are the split strength, MPa, at the deformation rate of  $V_1$  and  $V_2$  ( $R_2 > R_1$ );

$$\bar{R} = \frac{R_1 + R_2}{2}. \quad (8)$$

If the loading mode corresponds to the operation of asphalt concrete in the viscous stage ( $n_r \rightarrow 1$ ), then the asphalt concrete possesses a higher cyclic durability, which is able to dissipate a higher amount of energy  $W_d$  before destruction, which is correlated with the maximum deformation value  $\varepsilon_m$ , realized over a wide range of temperature and load time. The work of asphalt concrete in the viscous stage is observed in the course of relaxation processes, creep, etc.

Since an increase in  $R_c$  increases the probability of an improvement in durability in the elastic stage of acting and an increase in  $\varepsilon_m$  – in the viscous stage, in general ( $0 < n_r < 1$ ), the materials with the maximum value of the product of  $R_c \varepsilon_m$  will have the maximum cyclic durability (Li *et al.*, 2017).

If the value of the maximum amount of energy that can be dissipated by material  $W_m$  is known, the level of intra-structural damageability of material  $\psi$  can be calculated using the following dependency (Verenko & Makarevich, 2010):

$$\psi = \left(1 - \frac{W_d}{W_m}\right)^{K \frac{W_d}{W_m} - A}, \quad (9)$$

where  $K$  and  $A$  – the coefficients that uniquely depend on the number of elastic bonds  $n_r$  involved in the deformation process;  $W_d/W_m$  is the reduced amount of the dissipated energy.

The ultimate level of damageability theoretically equals 0. However, in actual fact, destruction occurs much earlier due to intensive association of microcracks at a certain stage and the avalanche-like formation of the main cracks. According to the percolation theory, the

bound threshold for a spatial cubic lattice is 0.25 (Radovskiy, 1992; Ustova, Kozlitin & Udodov, 2016).

The progressing of damageability of asphalt concrete of road pavement takes place in three stages: damage at the micro and sub-micro levels that cannot be detected using the existing methods and instruments; formation of microdefects and propagation of intra-structural damage, which can be recorded by special devices if observed for a number of years; propagation of the macrodefects that can be detected visually, i.e. defects that are visible on the surface. The coverage area of all the three stages is uneven. Therefore, it is important to ensure appropriate monitoring in order to identify the priority areas for prevention and repair of a certain type.

The kinetics of damageability accumulation depends on the temperature, level, and the time of action of loads, their application mode, and structural features of asphalt concrete. This situation implies the need to compile and solve complex kinematic equations including a large number of parameters that should be determined experimentally, which further complicates calculations.

At the same time, the issues of damage accumulation kinetics from the point of view of continual theories in the structure of composite road materials are valid in case of continuous mechanical action, whereas in case of application of load by degrees with “rest”, the process changes significantly, and it is necessary to take the recombination of the material structure into account. Recently, the account of recombination processes in the structure of composite materials has been described in a number of works, which opens up the opportunities for their wider assessment at the stage of calculation and diagnostics of road pavements.

The level of accumulated damageability in the material structure  $\psi$  is a simple yet effective indicator for evaluating fatigue and cyclic durability and can be used for a more effective and timely assessment, in comparison with the evaluation of elastic deflection and defects of the state of road pavements in the development of projects for their thin-layer recycling.

Therefore, it is of interest to adapt the provisions mentioned above to a high-speed method to monitor the condition of the road surface (up to 5 cm in thickness) in order to determine the optimal time for its regeneration.

There are a large number of works devoted to experimental and theoretical approaches to the study of the processes of continual accumulation of damageability and destruction of materials, the main attention is paid to the formation and development of pores, their shape change, merging or splitting (Bergheau, Leblond & Perrin, 2014; Besson, 2009; Gologanu, Leblond, Perrin, & Devaux, 1997;

Keralavarma, Hoelscher & Benzerga, 2011; Li & Yu, 2005; Tvergaard & Needleman, 1995).

Since the establishment of the continual theory of damageability, Kachanov (1974) and Rabonov (1987) have proposed the principle of linear damage summation. Later on, for example, in (Kanaun & Chudnovskiy, 1970), accumulation of microdefectness was modelled by changes in the material structure by the growth of inclusions with lower elastic properties, and the elastic properties of the damaged structure depended entirely on the concentration of such inclusions. When performing experimental work, these positions can be used to study damageability caused by one type of mechanical impact. The research result can be extended to other types through the parameters of the deformation process. A similar approach is described in (Pobedria, 1984), when the stress tensor at the time under consideration is determining via the strain tensor, which is known at an earlier time, and the dependence of the material properties, for example to temperature, can be established through the damageability function.

To date, a number of studies have been carried out on the progressing of asphalt concrete damageability, for example, in the process of alternate freezing and thawing, which confirm the development of microcracks, their integration and enlargement with changes in the interpore space in the structure of asphalt concrete (Darabi, Abu Al-Rub, Masad, & Little, 2013; El-Hakim & Tighe, 2014; Kayhanian, Anderson, Harvey, Jones, & Muhunthan, 2012; Li, Wang, Yi, Ma, & Lin, 2019; Shakiba *et al.*, 2013; Sung & Kim, 2012; Underwood, 2016; Varveri, Avgerinopoulos, Kasbergen, Scarpas, & Collop, 2014; Si *et al.*, 2015; Teltayev, Rossi, Izmailova, & Amirbayev, 2019; Xu, Guo & Tan, 2015, 2016).

In (Verenko, 1993; Verenko *et al.*, 2015), while considering the processes of recombination of the structure of the composite road materials in cases where they are experiencing multiple mechanical impacts, for example, subjected to freeze-thaw action and traffic load, the methods of summation of damages and work capacity by bringing various impacts to a single equivalent are used. In general, the study of damageability can be performed based on the studies of the features of asphalt concrete structural strength loss (degradation) during the exposure to a single cyclic factor (temperature or load of the appropriate type).

In this regard, we took on the task of studying the possibility of using the methods for assessing damageability through specifics of changes in the interpore space in the structure of asphalt concrete pavement under the influence of climatic factors and transport strain employing the tests of frost resistance during cyclic freezing-thawing of water-saturated samples.

## 2. Experimental studies of damageability of asphalt concrete pavements

Experimental studies were carried out from 2016 to 2019 in the process of laboratory tests of asphalt concrete mixes of AC and SMA types in the People's Republic of China and in the Republic of Belarus. Asphalt concrete mixes were accepted with both optimal and non-optimal binder content to expand the range of possible variation in their properties. Samples were produced according to the Marshall methodology in accordance with the requirements of T 0702 (Ministry of Transport of the People's Republic of China, 2011) standard in the amount of 14 pcs. for each series of asphalt concrete mixes (12 series for SMA type and 9 series for AC type).

The first four samples were tested for splitting (indirect tensile test) under T 0716 (Ministry of Transport of the People's Republic of China, 2011) at a temperature of  $-15\text{ }^{\circ}\text{C}$  and the deformation rate of 3 mm/min and 10 mm/min ( $R_{-15}^3$  and  $R_{-15}^{10}$ , MPa) with the calculation of the maximum strength of asphalt concrete over the entire temperature range and load time ( $R_c$ , MPa) using the Eq. (7).

The duration of one freeze to a temperature of  $-18\text{ }^{\circ}\text{C}$  from the moment of stabilization was no less than 2.5 hours. Thawing of samples after their discharge from the freezer was carried out for 2 hours in a water bath while maintaining the temperature of  $+18\text{ }^{\circ}\text{C}$ . For one day, at least one cycle of freezing-thawing was performed (on average, at least 2 cycles per day). If it was not possible to perform the test for several days, the sample was left in the freezer. Preliminary water saturation of the samples was performed in a vacuum installation, where the residual pressure of 2000 Pa (15 mm Hg) was created and maintained for 1 h. After that, the pressure was brought to 94 250–104 150 Pa, and the samples were kept in the same container with water at a temperature of  $+18\text{ }^{\circ}\text{C}$  for 30 min. Then, before being placed into the freezer, the samples were weighed in the water at a temperature of  $+18\text{ }^{\circ}\text{C}$ , with the determination of the weight of the water-saturated sample ( $g_w$ , g).

The first two samples after going through preliminary water saturation and being held at a temperature of  $+18\text{ }^{\circ}\text{C}$  for 30 minutes were placed in a bath with water and ice (in order to maintain a temperature of  $0\text{ }^{\circ}\text{C}$ ) and after 1 hour were tested for splitting (indirect tensile) at a deforming rate of 3 mm/min ( $R_0^3$ , MPa). The remaining 8 samples were placed into a freezer and a 120-cycle of freezing and thawing procedure was performed. After thawing for 5, 30, 60, and 120 cycles, the strength ( $R_0^3$ , MPa) and weight of water-saturated samples ( $g_w$ , g) were determined.

Table 1. Asphalt concrete mixes test results

Mix No.	Maximum strength ( $R_c$ , MPa)			Split strength at the deformation rate of 3 mm/min ( $R_0^3$ , MPa) after the number of cycles					Change in weight of water-saturated samples ( $\Delta g_w$ ) after the number of cycles				
	$R_{-15}^3$	$R_{-15}^{10}$	$R_c$	0	5	30	60	120	0*	5	30	60	120
SMA asphalt concrete mixes													
1	2.62	3.41	3.87	2.01	1.96	1.86	1.84	1.52	1/1201.05**	1.0007	1.0018	1.0025	1.0035
2	2.77	3.51	3.92	1.95	1.97	1.91	1.82	1.53	1/1201.97	1.0007	1.0018	1.0026	1.0036
3	2.99	3.67	4.01	2.04	2.08	1.95	1.9	1.63	1/1220.18	1.0007	1.0016	1.0023	1.0033
4	3.74	3.84	3.88	2.56	2.29	1.98	0.96	0.32	1/1213.33	1.0019	1.0047	1.0066	1.0093
5	3.53	3.60	3.62	2.29	2.00	1.81	0.83	0.26	1/1224.75	1.0019	1.0046	1.0066	1.0093
6	2.29	2.84	3.12	1.75	1.61	1.41	1.04	0.44	1/1240.62	1.0017	1.0043	1.0061	1.0086
7	2.63	3.24	3.56	1.78	1.84	1.83	1.57	1.31	1/1224.91	1.0007	1.0017	1.0024	1.0034
8	3.43	3.68	3.77	2.48	2.44	1.82	0.67	0.18	1/1230.56	1.0019	1.0048	1.0067	1.0095
9	2.65	3.51	4.02	1.95	1.96	1.82	1.92	1.69	1/1205.10	1.0011	1.0027	1.0038	1.0053
10	3.50	3.60	3.64	2.35	2.04	1.74	1.08	0.38	1/1221.16	1.0020	1.0048	1.0068	1.0096
11	2.57	3.59	4.27	1.97	2.03	2.01	1.96	1.76	1/1198.76	1.0006	1.0015	1.0021	1.0029
12	2.73	3.10	3.26	1.81	1.74	1.52	1.28	0.79	1/1223.48	1.0018	1.0044	1.0062	1.0088
AC asphalt concrete mixes													
1	2.21	2.59	2.77	1.67	1.53	1.44	0.71	0.28	1/1193.16	1.0013	1.0031	1.0043	1.0061
2	2.24	2.89	3.25	1.82	1.84	1.39	1.55	0.99	1/1196.72	1.0010	1.0025	1.0035	1.0050
3	2.33	3.03	3.44	1.73	1.81	1.64	1.66	1.33	1/1206.10	1.0009	1.0022	1.0031	1.0044
4	2.01	2.68	3.09	1.67	1.72	1.42	1.39	0.77	1/1199.81	1.0010	1.0025	1.0036	1.0051
5	2.36	2.80	3.01	1.68	1.52	1.44	1.22	0.81	1/1202.59	1.0011	1.0027	1.0038	1.0053
6	2.35	2.48	2.52	1.73	1.27	0.72	0.12	***	1/1194.35	1.0032	1.0079	1.0112	***
7	2.55	2.76	2.84	1.85	1.81	0.82	0.42	***	1/1201.37	1.0026	1.0065	1.0092	***
8	2.36	2.77	2.96	1.88	1.82	1.33	0.76	0.17	1/1195.55	1.0022	1.0054	1.0076	1.0108
9	2.20	2.79	3.12	1.73	1.74	1.42	1.31	0.78	1/1202.64	1.0015	1.0038	1.0054	1.0076

\* initial weight of the water-saturated samples –  $g_w^0$ , g.

$\Delta g_w^i = \frac{g_w^i}{g_w^0}$ , where  $i$  – test time (after 5, 30, 60, 120 cycles)

\*\* the denominator is the initial weight of water-saturated samples, g.

\*\*\* loss of sample shape and severe structural damage.

The test results are presented in Table 1.

In order to construct a computational model of asphalt concrete degradation during freezing and thawing, as previously assumed, describing the general structural changes from any type of impact (load, temperature), the value of the number of elastic bonds  $n_r$  was calculated for all studied mixes using Eq. (6). The calculation results are shown in Table 2.

During the process of mechanical or all other types of impacts, there is not just accumulation of damageability, there is also a decrease

Table 2. The results of calculating the relative number of elastic bonds

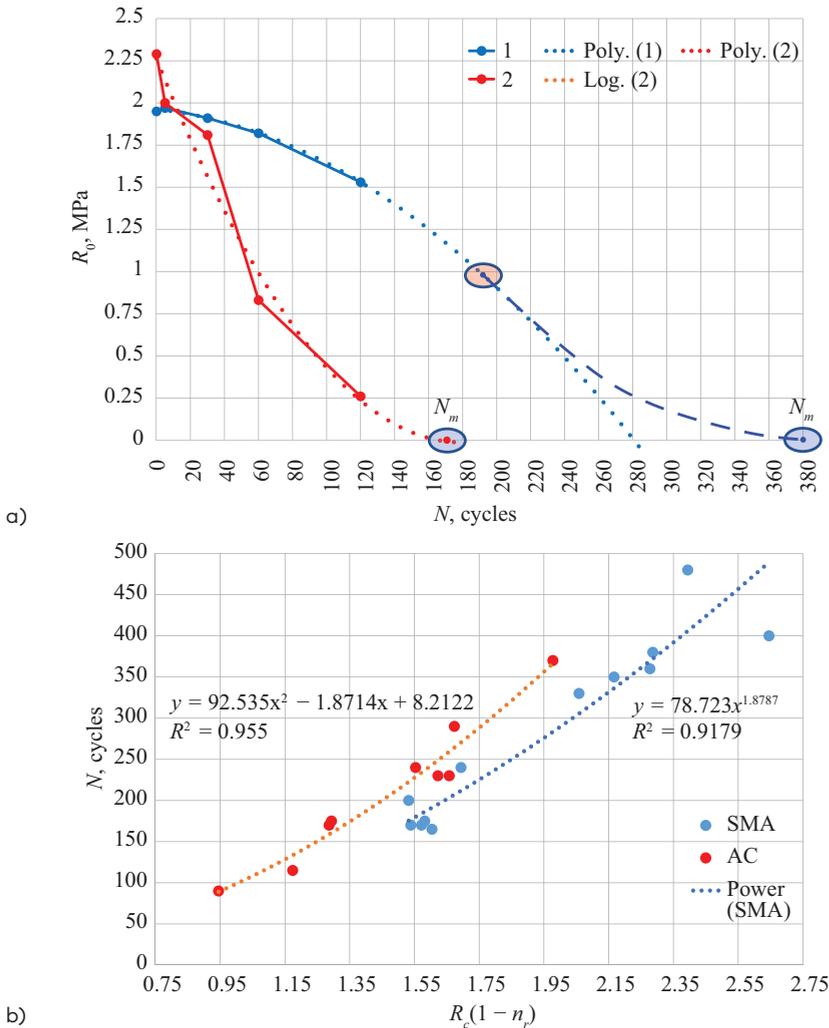
No.	1	2	3	4	5	6	7	8	9	10	11	12	
$n_r$	SMA	0.440	0.419	0.430	0.595	0.563	0.486	0.422	0.592	0.404	0.579	0.380	0.480
	AC	0.533	0.485	0.425	0.464	0.484	0.625	0.587	0.565	0.480			

in the work capacity. In this regard, in order to obtain the dependence of the damageability accumulation kinetics and while considering the relationship with the work capacity  $F$  of the dependence (Eq. (9)) ( $F = W_d/W_m \approx N_f/N_m$ ) as a basic model, the value of the reduced amount of dissipated energy can be replaced with sufficient accuracy for practice by the ratio of the actual number of cycles of load application  $N_f$  to the limit  $N_m$  (Li *et al.*, 2017). When processing experimental data, there were the limit values of cycles before the destruction of samples from repeated freezing-thawing, which are set graphically by extrapolation methods considering the type of the degradation curve (Figure 2a). The results of determination of  $N_m$  are shown in Table 3.

Studies have shown that the drop in structural strength (split strength) of asphalt concrete can generally be described by two types of descending curves (“ $R-N$ ” in the coordinates) (Figure 2a): Curve 1 approaching the logistic, or S-shaped; Curve 2 approaching the descending part of the parabolic. This situation allows us to say that the description of the curves can be performed with sufficient accuracy in relation to dependence (Eq. (9)). Thus, the calculated values of the limit number of  $N_m$  cycles before destruction (Table 3) determined with a high degree of confidence may be used in further research. At the same time, an additional criterion attesting reliability of the graphical extrapolation method can be seen as the proof that the value of the limit number of cycles before destruction in the freezing-thawing process is uniquely related to the value of the dissipated energy (Figure 2b), depending on the product of  $R_c(1 - n_r)$ , which was confirmed earlier (Li *et al.*, 2017). As it was noted above, it is due to the fact that the damageability occurs in the material as a result of breaking elastic bonds or reaching the limit of deformation. In the first case, the damageability is caused by the action of stress, which leads to the rupture of individual weakest bonds followed by formation of microcracks. In the second case, viscoplastic

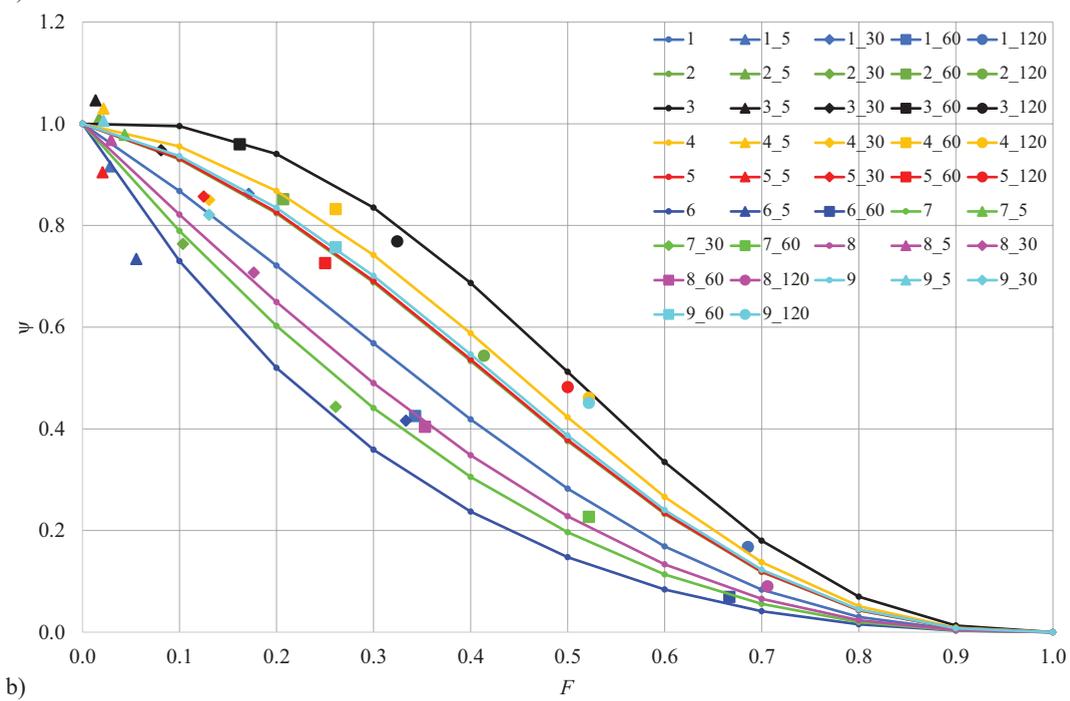
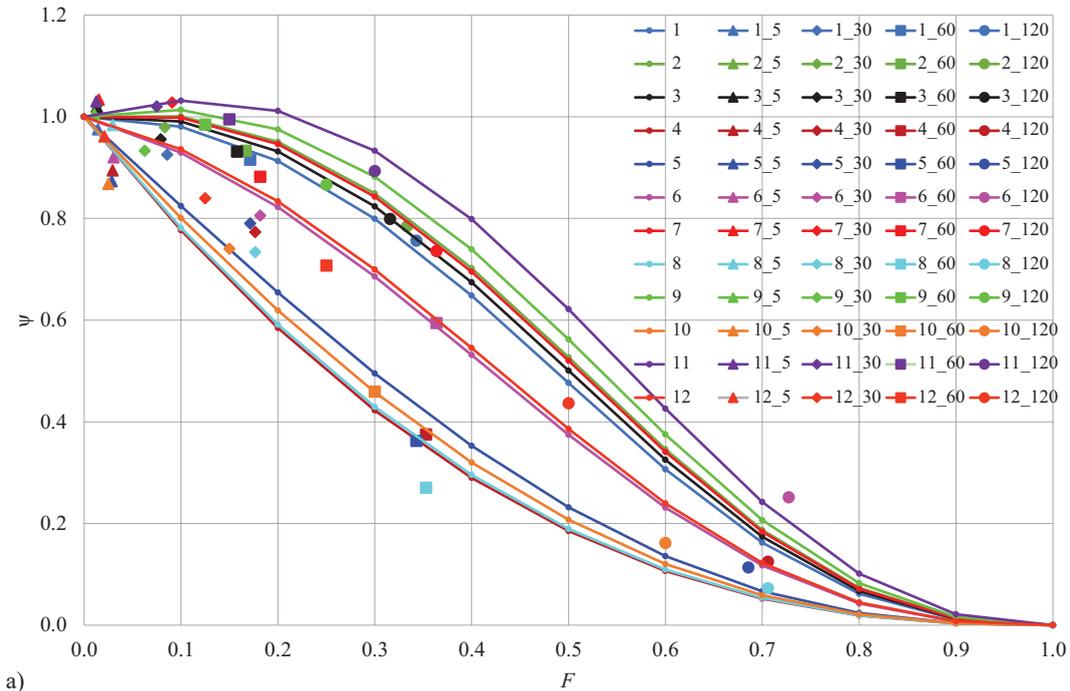
Table 3. Results of determining the limit value of freezing-thawing cycles

No.	1	2	3	4	5	6	7	8	9	10	11	12	
$N_m$	SMA	350	360	380	170	175	165	330	170	480	200	400	240
	AC	175	290	370	230	240	90	115	170	230			



**Figure 2.** Features of determining the maximum number of freeze-thaw cycles: a) extrapolation diagram; b) dependence between the structural asphalt concrete features and a limit number of cycles to complete destruction

deformation leads to the removal of structural aggregates from each other at a distance, at which the force of the inter-contact interaction decreases, and the loss of contact between them occurs. It all eventually leads, among other things, to an increase in the porosity of the structure. In this regard, the maximum cyclic durability will be demonstrated by materials that are capable of long-term resistance, break the limit number of elastic bonds and will also lead to the exhaustion of viscous



**Figure 3.** Experimental data and model curves: a) SMA; b) AC

deformations, i.e. materials characterized by a maximum value of  $R_c n_v$  (where  $n_v = 1 - n_r$ ).

Post-processing of experimental data (Table 1) in general confirms the possibility of using dependence (Eq. (9)) to calculate the level of damageability  $\psi$ , including the one from variable freezing-thawing (Figure 3a,b). This confirms the earlier assumption that damageability accumulation occurs similarly regardless of the type of impact – temperature and stress based on their level and features of application, taking into account the ratio of elastic and viscous bonds into account.

Model curves in Figure 3 were obtained by approximating experimental data, based on the fact that the level of performance  $F$  and the level of damage  $\psi$  were calculated as follows:

$$F = \frac{N_{f(i)}}{N_m}, \quad \psi = \frac{R_{0(i)}^3}{R_{0(0)}^3}, \quad (10)$$

where  $i$  – the test moment (after 5, 30, 60, 120 cycles; 0 – the initial result).

As a result, after theoretical and empirical analysis of experimental data in relation to Eq. (9), taking Eq. (10) into account, the following approximating dependency was developed:

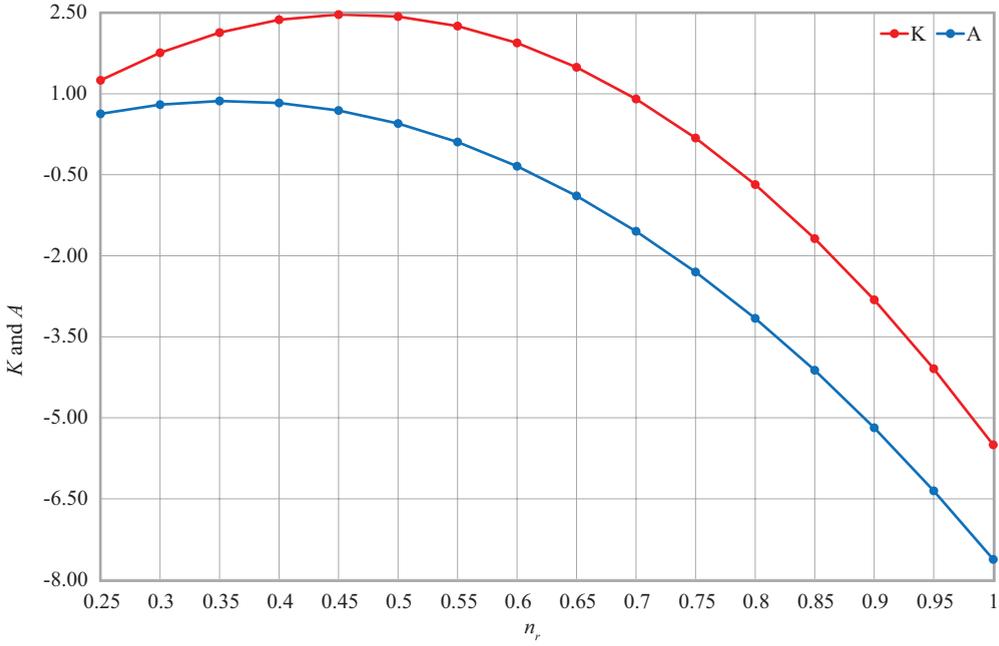
$$\psi = (1 - F)^{KF-A}, \quad (11)$$

and coefficients  $K$  and  $A$  can be calculated depending on the value of  $n_r$ , using the following empirical equations obtained from the results of statistical processing of experimental data (Figure 4a):

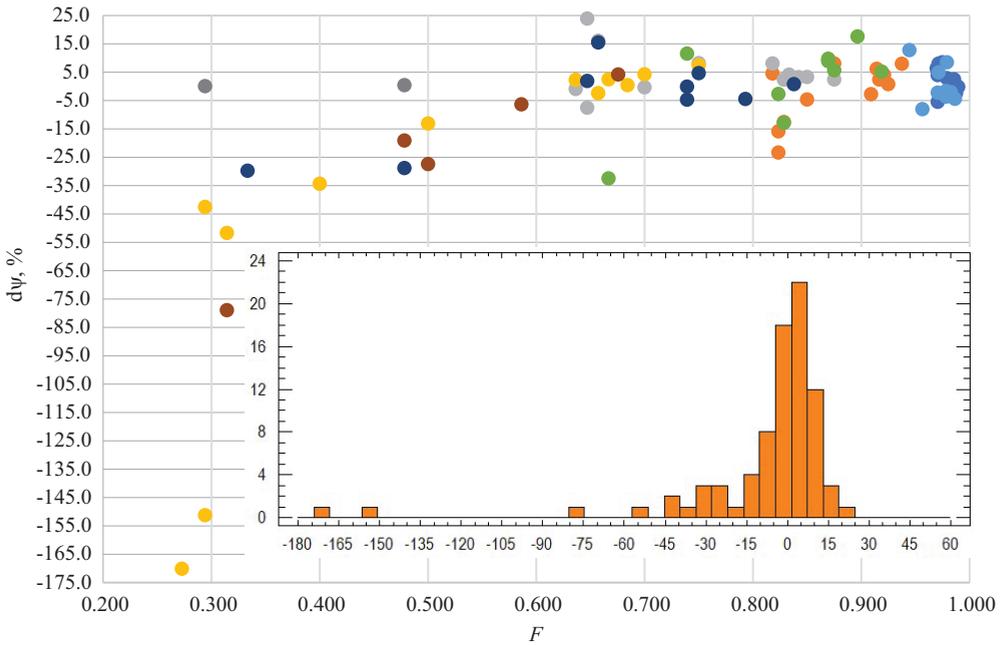
$$K = -3.36 + 25.3n_r - 27.44n_r^2, \quad (12)$$

$$A = -1.76 + 14.7n_r - 20.56n_r^2. \quad (13)$$

As the data of the accuracy estimation model, which links the level of damageability  $\psi$  and the level of work capacity  $F$ , shows (Figure 4b), in general, the model is adequate and can be used for a comparative assessment of the properties of asphalt concrete. For most model values (about 70% of the sample), the relative error in the damageability values  $d\psi$  does not exceed 15% with an absolute difference from  $-0.08$  to  $0.12$ . At the same time, it should be noted that the growth of the relative error is observed at the level of work capacity  $F$  being less than 0.5, when, first, the value of the level of damageability is quite small (about 0.15–0.60 from 1.00) and the model becomes more sensitive to the accumulation of error, and, second, the material begins to degrade more intensively, which leads to large errors in the experiment. This situation may also be associated with a change in the pattern of behavior of the material with



a)



b)

**Figure 4.** The results of plotting the model approximating dependence: a) empirical coefficients  $A$  and  $K$ ; b) model accuracy in the coordinates of work capacity  $F$

more intensive development of damage in its structure, for example, by reducing recombination processes efficiency. This issue requires further research with a more accurate and reliable recording of changes in the strength and deformation characteristics.

The research has also clearly confirmed the fact that with the increase of damageability levels, the interpore space of the composite material (asphalt concrete) changes, the number of micro-cracks increases, and the degree of pushing the big particles apart in structural frame increases. So, the experimental data presented in Table 1 indicates an increase in the degree of water saturation of the samples under study with an increase in the damage to the structure of asphalt concrete during variable freezing and thawing (Figure 5a).

The mass index of the water-saturated sample  $g_w$  is the most significant value (up to 90–95%) in the equation for determining changes in the water saturation index of asphalt concrete  $WS$  (Gosstandard of the Republic of Belarus, 2013):

$$WS = \frac{g_w - g_0}{g_{wa} - g_{ww}} \cdot 100, \% \quad (14)$$

where  $g_0$  is the weight of a dry (unsaturated with water) sample weighted in air, g;  $g_{wa}$  is the weight of the sample kept for 30 minutes in water and weighted in air, g;  $g_{ww}$  is the weight of the same sample weighted in water, g;  $g_w$  is the weight of a water-saturated sample weighted in air, g.

Taking Eq. (14) into account, the indicators of water saturation  $WS$  close to the actual were calculated depending on the  $\Delta g_w$  value (Table 1) and on the initial indicators, i.e. the ones before testing with alternate freezing and thawing, included in the equation. The calculation results are presented in Table 4.

As a result of additional data processing, Table 4 regression dependencies were obtained that link the level of  $\psi$  damage to the value of change (increase) in the  $\Delta WS$  water saturation index for SMA and AC asphalt concrete (Figure 5b).

The dependence in Figure 5b cannot be considered general, it is not final and requires further development and improvement. At the same time, it can be concluded that the development of damage is associated with a significant change in the interpore space and with the subsequent development of destruction due to the loss of connections between particles, which was proved by other authors earlier (El-Hakim & Tighe, 2014; Li *et al.*, 2019; Shakiba, Al-Rub, Darabi, You, Masad, & Little, 2013; Si *et al.*, 2015; Teltayev *et al.*, 2019; Varveri *et al.*, 2014; Xu, Guo & Tan, 2015, 2016). The development of these processes can be recorded by measuring, for example, the water saturation (residual

Table 4. The results of calculation of water saturation WS

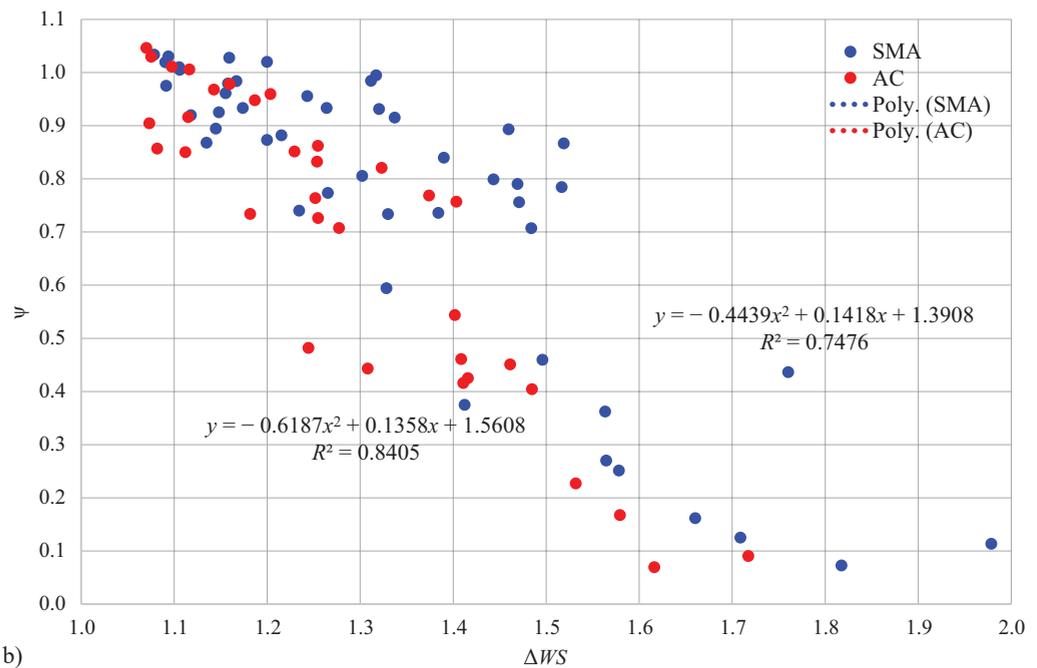
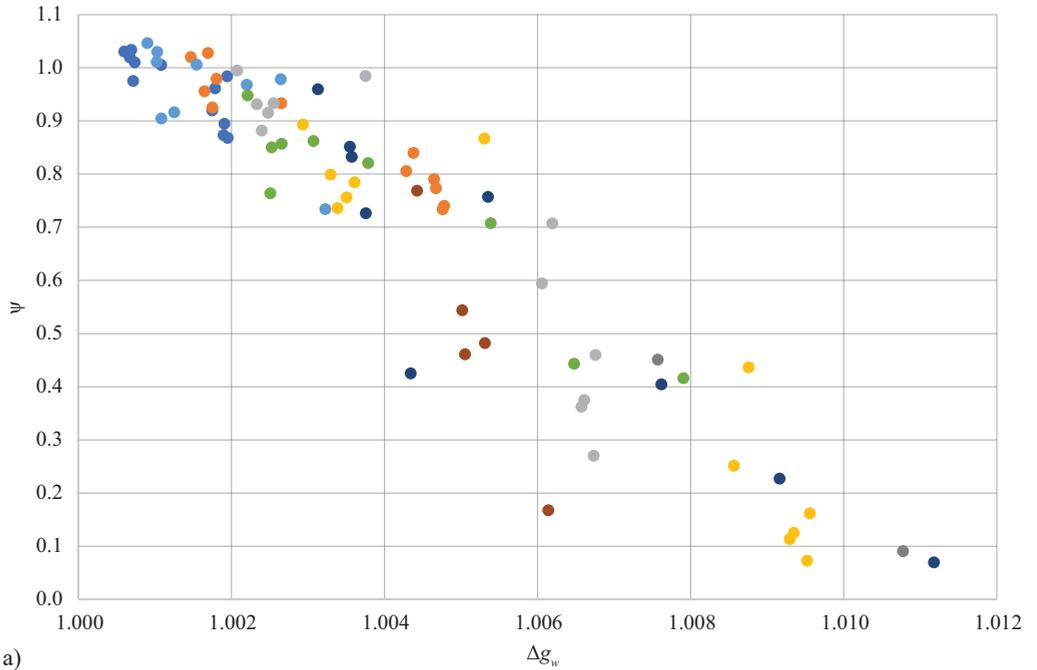
WS No.	Value of the indicator for the number of freezing and thawing cycles				
	0*	5	30	60	120
SMA asphalt concrete mixtures					
1	1.94	2.12	2.23	2.59	2.85
2	1.73	1.91	2.00	2.19	2.62
3	1.85	2.02	2.30	2.44	2.67
4	3.26	3.73	4.12	4.60	5.57
5	2.34	2.81	3.44	3.66	4.63
6	3.59	4.02	4.68	4.77	5.67
7	2.18	2.35	2.53	2.65	3.02
8	2.84	3.31	3.78	4.44	5.16
9	2.52	2.79	2.96	3.31	3.83
10	3.54	4.02	4.37	5.30	5.88
11	1.58	1.73	1.90	2.08	2.31
12	2.82	3.26	3.92	4.18	4.96
AC asphalt concrete mixtures					
1	2.61	2.91	3.28	3.70	4.12
2	2.56	2.81	3.20	3.15	3.59
3	3.16	3.38	3.75	3.80	4.34
4	3.33	3.58	3.70	4.17	4.69
5	3.58	3.84	3.87	4.49	4.45
6	4.27	5.04	6.02	6.90	-**
7	4.03	4.67	5.27	6.17	-**
8	3.71	4.24	4.74	5.51	6.37
9	3.22	3.59	4.26	4.52	4.71

\* the initial value of the indicator (Eq. (14)).

\*\* loss of the sample shape and severe structural damage.

porosity) of asphalt concrete during it being at work under appropriate influence (temperature, load, solar radiation, etc.). Thus, with a known value of the initial properties (strength, modulus of elasticity, water saturation, residual porosity), it is possible to predict the development of damageability and correct the curves of the “damageability – work capacity” dependence in the context of the life cycle of road asphalt pavement during special monitoring.

As seen in Figure 5b, SMA asphalt concrete is more resistant to the propagation of damageability in its structure in comparison with AC asphalt concrete, which is known from numerous studies conducted around the world, it confirms the adequacy of the assessment methodology. On average, before full destruction, water saturation in SMA asphalt concrete will increase 1.9–2.0 times, while in AC asphalt

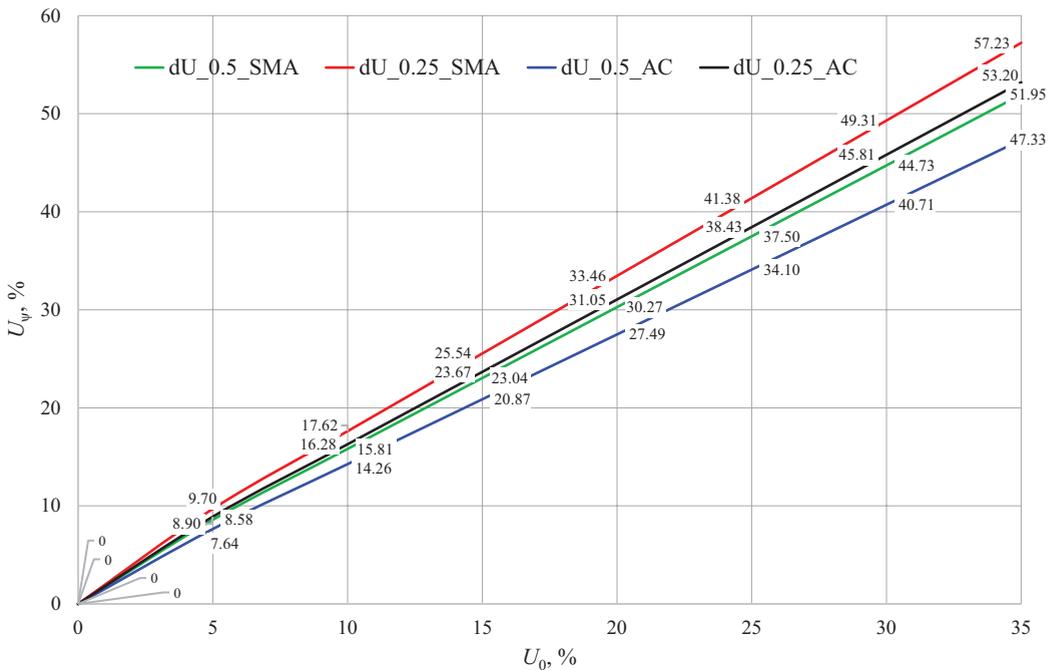


**Figure 5.** Change in water saturation characteristics of the samples depending on the asphalt concrete damageability level

concrete, it will increase 1.70–1.75 times. At the same time, it should be noted that at the level of damageability below 0.5, asphalt concrete of the two types shows itself almost equally, i.e. it can be assumed that when this level of destruction is reached, the degree of increase in defects on the road surface may differ slightly, and the speed of its development over time will increase exponentially. In this regard, the limit value of the level of damageability should be in the range of values from 0.5 (increase in the speed and intensive growth of defects) to 0.25 (considering structural bonds percolation, when their stability is lost).

Figure 6 shows the data of joint post-processing of experimental data (Table 1) and the data obtained earlier (Wang & Shi, 2017), when developing the method for assessing water permeability of road asphalt concrete surfaces.

After analysing the data in Figure 6 it can be concluded that permanent, for example, from the beginning of construction, high-speed monitoring of asphalt pavements using the method of assessing water permeability of road asphalt surfaces on the basis of recording changes



**Figure 6.** Change in indicator  $U$  (Eq. (9)) for the damageability level of SMA and AC type asphalt concrete of 0.25 and 0.5 depending on their initial properties

in their temperature, when drying after wetting, can simultaneously assess the development of damageability in the upper layer of road pavement, as well as predict the intensity of development of defects. This allows increasing efficiency of assigning the type and timing of preventive and repair work in future, including regeneration of asphalt concrete of the top constructive layer.

So, for instance, with the initial level of surface temperature change upon drying  $U_0$  equalling 5% (Figure 5), the SMA asphalt concrete will reach a damageability level of 0.5 at  $U_\psi$  value of 8.58% (or +72% in absolute value from  $U_0$ ), and for AC asphalt concrete – 7.64% (+53%); a damageability level of 0.25 with  $U_0$  value equalling 9.70% (+94%), and for AC asphalt concrete – 8.90% (+78%). At the initial level  $U_0$  of 25%, SMA asphalt concrete will reach a damageability level of 0.5 with  $U_\psi$  value of 37.50% (+50%), and for AC asphalt concrete – 34.10% (+36%); a damageability level of 0.25 with a  $U_\psi$  value equalling 41.38% (+66%), and for AC asphalt concrete – 38.43% (+54%). In this case, the service life of the asphalt concrete with the initial level  $U_0$  of 5% until the damageability level reaches, for example, 0.5 will be significantly longer than that of the asphalt concrete with the initial  $U_0$  level of 25%. The comparison of the initial  $U_0$  level with the achieved at the time  $i$  ( $U_\psi^i$ ), and also considering the limit  $U_{\psi=0.50}$  and  $U_{\psi=0.25}$  value, will allow measuring the intensity of the defectiveness development and, with appropriate processing, to predict it.

The relationship in Figure 6 can be used for comparative assessment, as well as for the development of methodological foundations using a set of experimental research data in the process of road surface diagnostics. This will improve the adequacy of the entire model study of damageability of asphalt concrete of the upper pavement layer in future. However, obtaining dependencies that take the actual service life of asphalt concrete pavement into account, as well as the changes in the interpore space in the process of destruction, requires additional research and further work.

We intend to use the developed primary methodology for rapid assessment of damage development in future in the process of monitoring the state of road asphalt concrete pavements including a thin-layer recycling projects, especially while optimizing the timing of regeneration work and selecting homogeneous sections during pre-project diagnostics.

## Conclusions

The existing diagnostic methods of road asphalt concrete pavements require constant improvement. This is vital when carrying out repair work by recycling to a depth of 5 cm, followed by construction of thin

layers on the regenerated materials. This recovery method can be used if the visible defects on the surface are insignificant and the service life reaches the limit. In order to increase effectiveness of pre-project diagnostic measures when performing the recycling of this type, it is proposed to assess the level of intra-structural damageability of asphalt concrete of the upper layer. The level of damageability characterizes the degree of accumulation of microdefects, which are almost impossible to determine experimentally, but they reflect the degree of exhaustion of pavement work capacity and the predicted service life.

The theory of damageability propagation in the structure of asphalt concrete has been considered and theoretical dependences of the level of damageability have been obtained. It has been established that the level of damageability depends on physical, mechanical, and structural properties, the index of maximum structural strength and the number of elastic bonds involved in the deformation process being the main parameters. Models and dependencies that relate these parameters to damageability and work capacity have been obtained.

Based on the theoretical and experimental studies, the correlation between the level of damageability and the kinetics of changes in the pore space of asphalt concrete over time has been established. This allows determining the level of damageability and predicting the parameters for estimating the service life by measuring the level of water permeability, measurement methods and equipment for which were developed earlier.

The obtained dependencies and methodological principles of damageability assessment can be used when performing surveys in the process of pre-project diagnostics in order to select sites (roads, streets) that require priority implementation of preventive and repair work, as well as in order to select the sites for regeneration of asphalt concrete of the upper layer and to identify them by current condition for more detailed studies.

The studies mentioned above will be carried out in order to collect data, post-process them, and improve the adequacy of the proposed models.

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