Limiting Pressure on Sandy Foundations of Pavements

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Abstract. The purpose of the article is to determine the limit value of pressure on the sandy foundation of pavement. Such pressure is understood as the first critical load on the soil, which separates the stages of linear and nonlinear deformation of the base. The method of experimental research is testing the base with a hard round stamp. The stamp load was applied in steps, and its value increased until a nonlinear dependence of the shrink from pressure appeared. The tests were performed on a built sandy foundation of pavement on the street Donetskaya in the city of Omsk. For the experiment, 15 points were chosen at which was installed the stamp and loaded with measurement of load and shrink. The test results are processed by methods of mathematical statistics with a reliability of 95%. As a result, a confidence interval is established within which the true value of the limiting pressure on the sand base varies. The value of limiting pressure can be used when designing pavements as a value that limits pressure on the sand base and eliminates shear deformation of the sand.

1. Introduction

The development of methods for calculating pavement is one of the important tasks. Modern methods of calculation should ensure the design of structures that work mainly in the elastic stage without the accumulation of significant plastic deformations. However, operating experience of various pavements shows that often a rut forms on the pavement. Specialists distinguish three types of ruts [1, 2]. A rut caused by surface wear appears in winter as a result of the impact of tire spikes and anti-icing abrasive materials [3–5]. The place of concentration of such a rut is the place of braking cars. Therefore, the rut of wear is not associated with the accumulation of residual deformations in the layers of the pavement.

A rut formed as a result of the instability structure of asphalt concrete occurs as a result of plastic shear deformations occurring in the layers of asphalt concrete. The reason for the formation of such a rut is associated with the mechanical properties of the asphalt mixture, the content of air voids and resistance of the mixture to the shear [1, 6].

The structural rut is due to the accumulation of residual deformations in all layers of road pavement and the soil of subgrade. In this case, large deformations of shear are absent, and residual deformations are caused by compaction of granular materials and soils [6, 7].

Ruts formed on the surface of pavements of granular materials are distinguished into three types [8–10]. The first type includes ruts formed as a result of compaction deformations arising in layers of...
granular materials. Such ruts have the smallest depth of all three types. The reasons for the formation of ruts of the second type are deformations of shear in the layer from granular material. The depth of such ruts is greatest. The third type of ruts is caused by deformations of shear in the base, underlying a coating of granular materials.

From the analysis of the classification of ruts it follows that in order to reduce the depths of ruts to the limit values regulated by different authors [6, 11–13], deformations of shear must be excluded. To do this, at the design stage, methods for calculating pavement are used, which make it possible to check the structure for shear resistance in the soil of subgrade and sandy additional layers of the base. Such calculations are conduct on tangential stresses using normative calculation methods, or their modified versions [14–16].

An alternative calculation of shear resistance is to calculate the limiting pressure on the soil base and compare its value with the actual pressure transmitted by the load. Currently, solutions have been obtained for calculating the limiting pressures from various loads [17–21], including the wheel [22]. The advantage of these solutions is the use of an exact mathematical apparatus based on the solution of a system of differential equations. However, these limiting pressures calculated by these solutions tend to exceed the first critical load. The first critical load is equal to the pressure value at which the base soils work in the compaction stage without experiencing deformations of shear.

The first critical load is calculated from a plasticity condition, for example, the Mohr Coulomb’s Criterion, with the substitution of the formulas for calculating the main stresses into it. Depending on the adopted equation of limiting state and the formulas for calculating the main stresses, a certain formula is derived for calculating the first critical load \( p_{lim} \). Currently, various formulas are known [23, 24], designed to calculate the first critical load. Using the data of these works, the criterion of shear resistance can be written as:

\[
p \leq k_{st} \cdot p_{lim},
\]

where \( p \) – pressure transmitted by the overlying layers to the test layer or subgrade, \( p_a \); \( k_{st} \) – strength factor depending on the required level of reliability; \( p_{lim} \) – limiting pressure corresponding to the first critical load, \( p_a \).

Considering the methods for determining the first critical load, we note that the simplest mathematical solutions are formulas whose derivation is based on the use of linearly deformable medium methods. This method is based on using the soil plasticity condition recorded in main stresses and formulas for calculating these stresses. The method was developed by N.P. Puzyrivsky for calculating the value of safe pressure from the strip foundation on the soil foundation. Nevertheless, the formula of N.P. Puzyrivsky is often used by experts in the road industry to calculate the first critical load on the soil of subgrade. Below is the conclusion of the formula N.P. Puzyrivsky.

To determine the value of the main stresses, the Michell representation is used, which, taking into account the volume forces, is written in the form:

\[
\sigma_1 = \frac{p - \gamma \cdot h}{\pi} \left( \alpha \pm \sin \alpha \right) + \gamma \cdot \left( z + h \right),
\]

where \( \gamma \) – soil weight, \( \text{N/m}^3 \); \( h \) – side load height (the height of the soil located on the side of the foundation, or the depth of the foundation), \( m \); \( z \) – the distance from the base of the foundation to the most dangerous point of the soil base, in which the limit state arises according to the plasticity condition, which is the basis for the further solution, \( m \); \( \alpha \) – angle of load visibility.

As a condition of plasticity, the Coulomb - Mohr criterion is adopted, into which formulas (2) are substituted. The resulting equation is solved with respect to value \( z \), as a result of which:

\[
z = \frac{p - \gamma \cdot h}{\pi \cdot \gamma} \left( \frac{\sin \alpha}{\sin \varphi} - \alpha \right) - \frac{c}{\gamma} \cdot \cot \varphi.
\]

From the analysis of equations (3), it is concluded that the maximum value of \( z \) takes place when determining the angle \( \alpha \) by the formula:

\[
\alpha = \frac{\pi}{2} - \varphi.
\]
Substituting dependence (4) into equation (3), we obtain a formula for determining the value of \( z_{\text{max}} \), which was called the maximum depth of development of unstable regions:

\[
z_{\text{max}} = \frac{p - \gamma \cdot h}{\pi \cdot \gamma} \left( \cotg \varphi + \frac{\pi}{2} - h \cdot \frac{c}{\gamma} \cdot \cotg \varphi \right).
\]  

Having completed the solution of equation (5) with respect to pressure, we obtain a general solution to the problem of the safe pressure value, which transfers the flexible strip foundation to the soil base, from the condition that the limiting state occurs at its most dangerous point according to the Coulomb – Mohr condition.

\[
p = \left( z_{\text{max}} + h \cdot \frac{c}{\gamma} \cdot \cotg \varphi \right) \frac{\pi \cdot \gamma}{\cotg \varphi + \frac{\pi}{2}} + \gamma \cdot h.
\]

V.G. Berezantsev gives formula (6) as a general solution to the problem of safe pressure from a flexible strip foundation, indicating that all particular solutions follow from this generalizing model when substituting different values into it to determine \( z_{\text{max}} \). If we substitute \( z_{\text{max}} = 0 \) in dependence (6), then after the corresponding transformations we obtain the formula N.P. Puzyrevsky:

\[
p_{1\text{lim}} = \gamma \cdot h \cdot \frac{\cotg \varphi + \frac{\pi}{2}}{\cotg \varphi + \frac{\pi}{2}} + \gamma \cdot \frac{\pi \cdot c \cdot \cotg \varphi}{\cotg \varphi + \frac{\pi}{2}}.
\]

The physical meaning of formula (7) is that, at \( z_{\text{max}} = 0 \), the most dangerous points at which the limiting state occurs according to the Coulomb – Mohr condition are located under the edges of the cross section of the strip foundation. This is equivalent to the fact that the development of unstable areas of soil from its edges in depth to the middle is impossible.

Of course, that formula (2) can be replaced by formulas designed to calculate the main stresses from a load distributed over a circular area, and repeat all the mathematical steps we have outlined. Then, instead of dependence (7), we obtain a solution for calculating the first critical load on the subgrade. Such a decision will be more consistent with the real conditions of the subgrade. But, as we said above, formula (7) is widely used by experts in the road industry. From the analysis of criterion (1), follows the relevance of work aimed at the experimental determination the value of safe pressure.

2. Materials and methods

The first and second critical loads on soil bases are closely related to the stages of soil work. Classical performances developed by N.M. Gersevanov, on the stages of work of soil bases and critical loads are shown in figure 1.

![Figure 1. Stage of work of the soil base: I, II and III – respectively stages of compaction, shear and loss of bearing capacity; \( p_0 \) – soil structural strength; \( p_{1\text{lim}} \) and \( p_{2\text{lim}} \) – first and second critical load; \( R \) – design soil resistance; \( S_R \) – value of shrink, limiting the linear dependence of settlement on pressure.](image)
first critical load $p_{lim}$ limits the first stage of work of base for which is characterized a linear relationship between settlement and pressure, but the settlement is elastoplastic. This stage is called the compaction stage. If the pressure exceeds the first critical load ($p>p_{lim}$), then the second stage of work the base, called the shear stage, occurs. In this case, the settlement is connected nonlinearly with pressure, and zones of limit equilibrium are formed in the soil. Inside these zones, the Mohr–Coulomb plasticity criterion is not executed. Moreover, the higher the pressure, the larger the zone of limit equilibrium. The calculated soil resistance $R$ is calculated by the solutions obtained by the method of limiting soil equilibrium [17–22, 25]. The value of this resistance is greater than the first critical load, but less than the second critical load, that is $p_{lim}<R<p_{2lim}$. The second critical load limits the stage of shear. If the pressure exceeds the second critical load, a third stage occurs, which is called the stage of loss of bearing capacity. At this stage, beyond the edges of the stamp, there is soil uplift formed as a result of extrusion of the material along the sliding surface. At this stage, an increase of shrink occurs without an increase in pressure.

The full-scale experiment was implemented by performing stamp tests during the construction of road pavement of the highway along Donetskaya Street in the city of Omsk. The experiments were carried out using stamp test equipment, which includes: a hard round stamp with a diameter of 33 cm, equipped with a hydraulic jack, an electronic sensor for measuring the load, and dial gauges for measuring shrink. The load was applied in steps. Before the application of each step, the first counting was taken from the dial indicator. The load was kept until the conditional stabilization of shrink. The criterion for conditional stabilization of shrink was considered to be a decrease in its velocity to 0.01 mm/min. At the end of the soil shrink, the second indicator report was taken. Based on the difference between the first and second reports, the shrink of base from the degree of load was determined. Total shrink from the pressure determines by the sum of shrink from a given steps of load and all the previous steps. When the shrink was increased to the value of the corresponding expressed nonlinear dependence on pressure, the base was unloaded. The load was removed in steps, and the elastic deformation was calculated by the difference in the indicator readings before the removal of the load stage and after conditional stabilization, the restored deformation.

To process the test results, mathematical statistics of the normal distribution law of a random variable are used.

3. Results

Figure 2 shows illustrations of a graphical determination of the structural strength $p_0$ and the first critical load on a sandy base.

![Graphical determination of structural strength and first critical load](image)

Figure 2. Experimental dependence of shrink from pressure and graphical method of determining the first critical load.

Stamp tests were carried out at 15 points of the sand base. The test results are processed by methods of mathematical statistics using the normal distribution law. When processing the results, the
average values of the structural resistance and the first critical load, the mean square deviations, the variation coefficients, and the calculated values of these random variables that limit the confidence interval are calculated. The results of statistical processing are shown in table 1.

### Table 1. Results of statistical processing of experimental data.

| Name of characteristic | Pressure limit |  |
|------------------------|----------------|---|
|                        | $P_0$          | $P_{lim}$ |
| Average value, kPa     | 67,3           | 140,8     |
| Mean square deviation, kPa | 6,7          | 13,4      |
| The coefficient of variation, % | 9,96       | 9,52      |
| Estimated values, kPa  | minimum       | 63,5      |
|                        | maximum       | 71,1      |
|                        | 133,3         | 148,3     |

### 4. Discussion

As a result of experimental studies, it was found that the true value of the limiting pressures limiting the stage of elastic work of the base and the stage of linear elastoplastic deformations are in the range of $63.5 - 71.1$ kPa and $133.3 - 148.3$ kPa. When transmitting to the base of fine sand the pressure equal to the first critical load the elastoplastic shrink is approximately 5 mm. In this case, the plastic shrink is about 4 mm, and the elastic deformation is 1 mm.

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