Розглянуто перспективність використання нелінійних моделей міцності при визначенні початкового критичного навантаження на ґрунт, а також нормативного та розрахункового опорів основи, що дозволяє зменшити трудомісткість процесу визначення міцнісних властивостей ґрунтів.

На підставі аналізу та узагальнення результатів теоретичних досліджень геомеханічних процесів з використанням аналітичних математичних методів отримані модифікації формул, які призначені для визначення початкового критичного навантаження на ґрунт, а також нормативного та розрахункового опорів основи.

Встановлено взаємозв'язок міцності, зокрема питомого зчеплення і кута внутрішнього тертя, що входять в умови міцності Кулона-Мора та А. Шашенка, що дозволяє вдосконалити методику розрахунку зовнішніх навантажень на ґрунт.

Проаналізовано залежності критичних навантажень на основу від середнього тиску під підошвою фундаменту в діапазоні тисків P=100...500 кПа з використанням умов міцності Кулона-Мора та А. Шашенка.

Встановлено, що при використанні загальноприйнятих розрахункових формул для визначення критичних навантажень на основі обов'язково слід враховувати діапазон тисків, при яких визначалися властивості міцності ґрунту. При цьому використання критерію міцності А. Шашенка для визначення критичних навантажень на основу дозволяє коректно врахувати вплив на них середнього тиску під підошвою фундаменту.

На відміну від залежностей, які використовуються в даний час в українських, білоруських, російських та нормативних документах інших країн, отримані формули дозволяють враховувати залежності міцнісних властивостей ґрунту від середнього тиску на ґрунт під підошвою фундаменту. Отримані результати дозволяють підвищити достовірність визначення початкового критичного навантаження на ґрунт, а також нормативного та розрахункового опорів основи. Це досягається за рахунок урахування нелінійності обвідної граничних кіл Мора з використанням умови міцності А. Шашенка

Ключові слова: критерії міцності Кулона-Мора і А. Шашенка; навантаження на ґрунт, зчеплення, кут внутрішнього тертя

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DETERMINING THE INFLUENCE OF PHYSICAL NONLINEARITY OF SOIL STRENGTH PROPERTIES ON THE ESTIMATED BASE RESISTANCE

O. Shashenko
Doctor of Technical Sciences, Professor*

V. Shapoval
Doctor of Technical Sciences, Professor*

O. Kovrov
Doctor of Technical Sciences, Associate Professor
Department of Ecology and Environmental Technology**
E-mail: kovralex1@gmail.com

A. Skobenko
PhD, Associate Professor*

O. Tiutkin
Doctor of Technical Sciences,
Associate Professor, Head of Department
Department of Bridges and Tunnels
Dnipro National University of Railway Transport named after Academician V. Lazaryan
Lazaryana str., 2, Dnipro, Ukraine, 49010

K. Babii
PhD, Senior Researcher
Department of Geomechanical Foundations of Open Field Mining Technologies
M. S. Polyakov Institute of Geotechnical Mechanics NAS of Ukraine
Simferopolska str., 2A, Dnipro, Ukraine, 49005

O. Samorodov
Doctor of Technical Sciences,
Associate Professor, Head of Department
Department of Geotechnics and Underground Structures
Kharkiv National University of Civil Engineering and Architecture
Sumska str., 40, Kharkiv, Ukraine, 61002

S. Slobodyanyuk
Doctor of Technical Sciences, Professor, Head of Department
Department of Theoretical Mechanics
Prydniprovska State Academy of Civil Engineering and Architecture
Chernyshevskoho str., 24a, Dnipro, Ukraine, 49600

*Department of Construction, Geotechnics and Geomechanics**
**Dnipro University of Technology
Dmitro Yavornytskyi ave., 19, Dnipro, Ukraine, 49005

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1. Introduction

When constructing the foundations for buildings and facilities, one of the key stages of engineering calculations is determining the initial critical load on soil, as well as the standardized and estimated base resistances. In the practice of designing and building foundations, important stages of engineering calculations are determining such loads on base as:

1. The pressures prior to which soil performs under a load as a linear medium.

2. The pressures under which there is a loss of s structure's stability, associated with the outburst of soil from under the sole of the foundation and a significant increase in settling.

Under the initial critical load on soil of the base, the limiting condition under the sole of the foundation occurs only at some of its points. External factors related to the geographical and climatic features of a territory, the engineering, geological and hydrogeological characteristics of a construction site, affect the stability of the foundation. The most significant parameter in its construction is the estimated soil resistance (R), which makes it possible to determine the maximally possible values of mass for the overlying structure of a facility. Exceeding the limiting values of the bearing capacity soil leads to that, under the sole of the foundation, there form the regions of excess stresses, which is manifested in the form of deformations and disruption of its integrity and stability.

Many scientific papers addressed the impact of vertical loads on the bearing capacity of foundations on soils; their comprehensive analysis is of practical interest [1]. However, the increasing pace of construction in the world and the high safety and reliability requirements to modern buildings necessitate critical analysis and evaluation of existing building codes.

2. Literature review and problem statement

Paper [2] shows that the use of composite soils using gravel-clay and sand-clay mixtures as a cushioning layer under a shallow foundation is expedient when the underlying ground does not provide for a carrying capability. However, the authors did not study the effect of the height of an embankment layer on the bearing capacity of a foundation.

Work [3] reported, based on a unified solution to shear strength for unsaturated soils under conditions of a flat deformation, a unified solution to critical load for the foundation in unsaturated soils. The results obtained provide a theoretical basis for determining the carrying capacity and optimizing the structure of foundations in unsaturated soils. However, the cited work did not take into consideration the impact of soil strength characteristics with respect to consolidation.

Article [4] gives an algorithm for the geodesic processing of results from leveling the control indicators of deformations along the perimeter of a foundation, which makes it possible to correctly determine the direction of the foundation's defects. This allows effective management decisions to be made when building a foundation, taking into consideration the redistribution of loads along its perimeter. The article's downside is the lack of results from numerical modeling of a foundation's stability.

The results from studying the carrying capacity of soils with the help of geo-packages are reported in work [5]. The use of physical models and numerical modeling has shown that the use of geo-packages under supports significantly increases the carrying capacity of foundations. However, in order to improve the reliability of results obtained, it would be expedient to carry out analytical studies into the impact of geo-materials on the strength of a foundation.

Choosing a shape of the frame structures for foundations considerably affects their carrying capacity. Work [6] explores two alternative shapes of foundations: rectangular strips and folding strips. Numerical simulation of the effect of changes in the shape of a foundation on stresses in the concrete base of the foundation and the underlying soils showed that the folded strip supports were effective in reducing the amount of reinforcement required. For a comprehensive coverage of the set problem, the cite work lacks analysis of the use of various frame structures of foundations in construction.

Similar results from using ribbon foundations for unsaturated soils are presented in paper [7]. It was established that the strength characteristics of unsaturated rocks and the stressed states of array affect the permissible carrying capacity of a foundation. The disadvantage of the cited paper is the lack of influence of moisture saturation on the strength characteristics of soil under the sole of a foundation.

Study [8] proposes a modeling technique by the method of finite elements to assess the carrying capacity of shallow foundations in unsaturated bound soils. The proposed procedure is very consistent with the results from testing model supports on unsaturated bound soils. However, the study does not take into consideration the factor of soil porosity and its effect on the parameters of cohesion and the angle of internal friction in array.

Article [9] reports a fundamental model for describing the non-drained cyclical stressed-deformed reactions of soft clays based on equivalent theories of viscosity and creep. Based on the model, a method of finite elements was developed to analyze the deformation of an anchor foundation in soft clay exposed to static and cyclical loads. A comparison of projected and model test results shows that the method could be used to assess cyclical stability and to identify the destruction process of anchor foundations. Despite the fundamental level of the reported research, the model does not take into consideration the factor of a foundation's stability over time, taking into account the territorial and climatic features of a region.

Work [10] explores the impact of a cushioning layer of the gravel-clay and sand-clay mixture under the shallow base of a foundation, which is subjected to an eccentric vertical load. According to the results of the calculation, the base above the sand-clay mixture demonstrates a relatively better performance compared to the gravel-clay mixture, which makes it possible to control the carrying capacity of a foundation. However, the work does not study the cushioning ability of the proposed mixtures on the size of grains in a filling material.

Article [11] gives simple formulae for assessing the parameters of a concentrated system that reproduces a frequency-dependent dynamic stiffness of foundations of the end piles. The model makes it possible to analyze the inertial interaction between soil and the facilities' structures, taking into consideration performance of the system «soil – foundation». However, the reported model is rather simplified and does not take into consideration the impact of static and dynamic loads on the structure of a foundation.

The above studies address the task on determining such a load on the base at which it is possible for the soil to outburst from under a foundation. However, we have not found any paper in modern technical literature on improving the methods for determining the initial critical pressure on soil, as well as the standardized and estimated base resistances.

Currently, the estimated soil resistance R [1, 12, 13] is used to determine the region of pressures applying which it is accepted to perform base calculations within the linear model.

The issue is related to that, at present, in determining the estimated resistance of soil one does not take into consideration the non-linearity of dependence of the soil strength properties (that is specific cohesion, grip, and an internal friction angle) on normal pressure.

A similar problem emerges in determining the initial critical pressure on soil, as well as the estimated standardized soil resistance.

3. The aim and objectives of the study

The aim of this study is to determine the effect of the physical non-linearity of soil strength properties on the estimated resistance of a base. This will make it possible to assess the magnitude of pressures, the standardized and estimated resistance of the base.

To accomplish the aim, the following tasks have been set:

– to determine the initial critical load on a base, taking into consideration the non-linear dependence of soil characteristics (rock) on mean pressure under the sole of a foundation;

– under the same assumptions, determine the standardized estimated resistance of a base;

– to determine the estimated base resistance by analogy.

4. Materials and research methods aimed at taking into consideration the non-linear dependence of soil strength on load on a base

A variety of non-linear models of strength are used to solve the problems on strength and stability, specifically a Shashenko failure criterion [1]. Accounting for the non-linearity makes it possible to represent the strength properties of soil as a function of pressure under which they were determined, thereby significantly expanding the region of their change. The approach below avoids the systematic error that typically occurs at present in geotechnical calculations – a mismatch between the pressure ranges, at which the properties of soil were determined, and the pressures under which the estimation is performed. In contrast to existing procedures, the proposed formulae make it possible to take into consideration changes in the physical and mechanical characteristics of soil depending on mean pressure under the sole of a foundation, which improves the reliability of engineering calculations related to a critical load on soil and the standardized base resistance.

The following assumptions regarding the soil and foundation characteristics are accepted as initial data for the current study:

– one knows the dependence of strength properties of soil or rock (specific cohesion c and the angle of internal friction φ) on vertical pressure σ acting on it;

– one knows the specific weight of soil (rock) above the sole of a foundation γ' and the specific weight of soil (rock) below the sole of a foundation γ;

– one knows the depth of laying the sole of a foundation d and the width of a foundation's sole b.

In the course of a theoretical study, we compared results obtained in determining the initial critical load on a base $P^{i.cr.}$ and the standardized base resistance R^{st} with the solutions reported in the academic and scientific literature. At the same time, for the convenience of presenting the material, we did not compare the estimated resistances regulated by standards. This is due to the fact that the standardized R^{st} and the estimated base resistance R differ only in a multiplier $(\gamma_{c1}\cdot\gamma_{c2})/k$, where γ_{c1}, γ_{c2} and k are the empirical coefficients, depending on many factors) [1, 12, 13].

To determine the estimated resistance, let us consider the estimation scheme shown in Fig. 1, *a*.

This scheme shall be divided into two components, one of which corresponds to a semi-space whose upper boundary is exposed to distributed load q, infinite in plan, which is numerically equal to:

$$q = \gamma' \cdot d, \tag{1}$$

where $q = \gamma' \cdot d$ is the specific weight of soil above the sole of a foundation, and $q = \gamma' \cdot d$ is the depth of laying its sole (Fig. 1, *c*), and a second one to the semi-space whose upper boundary is exposed to a local distributed load of width b, which is numerically equal to:

$$q_1 = P - \gamma' \cdot d, \tag{2}$$

where P is the mean pressure under the sole of a foundation.

Next, we find principal stresses σ_1 and σ_3 that correspond to the estimation schemes shown in Fig. 1, I, IV.

The vertical and horizontal stresses that correspond to the estimation scheme shown in Fig. 1, III are equal to:

$$\left.\begin{aligned} \sigma_z &= \gamma' \cdot d + \gamma \cdot z; \\ \sigma_x &= \xi \cdot \sigma_z = \xi \cdot (\gamma' \cdot d + \gamma \cdot z); \\ \sigma_y &= \xi \cdot \sigma_z = \xi \cdot (\gamma' \cdot d + \gamma \cdot z); \\ \tau_{xy} &= \tau_{xz} = \tau_{yz} = 0; \\ \xi &= \frac{\nu}{1-\nu}. \end{aligned}\right\} \tag{3}$$

where σ_z is the vertical normal stress; τ_{xy}, τ_{xz}, τ_{yz} are the tangential stresses; $\xi=\nu/(1-\nu)$ is the rebuff ratio; v is the Poisson coefficient [1, 14].

When the soil approaches its limiting state, its Poisson coefficient tends to a value equal to 0.5. Considering that, we obtain:

$$\sigma_z = \sigma_x = \sigma_z = \gamma' \cdot d + \gamma \cdot z. \tag{4}$$

It follows from (4) that in a given case the principal stresses σ_1 and σ_3 are equal to each other and equal to the normal stresses acting in a base, which is why:

$$\sigma_{1,\text{III}} = \sigma_{3,\text{III}} = \sigma_z = \sigma_x = \sigma_z = \gamma' \cdot d + \gamma \cdot z. \tag{5}$$

In formula (5), the «III» indices indicate the correspondence of the principal stresses, calculated by using it, to the estimation scheme in Fig. 1, III.

To determine the principal stresses σ_1 and σ_3 corresponding to the estimation scheme shown in Fig. 1, IV, we use a known Mitchell formula [1]. We obtain:

$$\left.\begin{aligned} \sigma_{1,\text{IV}} &= \frac{-\gamma' \cdot d}{\pi} \cdot \left[m \cdot \alpha + \sin(\alpha)\right]; \\ \sigma_{3,\text{IV}} &= \frac{-\gamma' \cdot d}{\pi} \cdot \left[m \cdot \alpha - \sin(\alpha)\right]. \end{aligned}\right\} \tag{6}$$

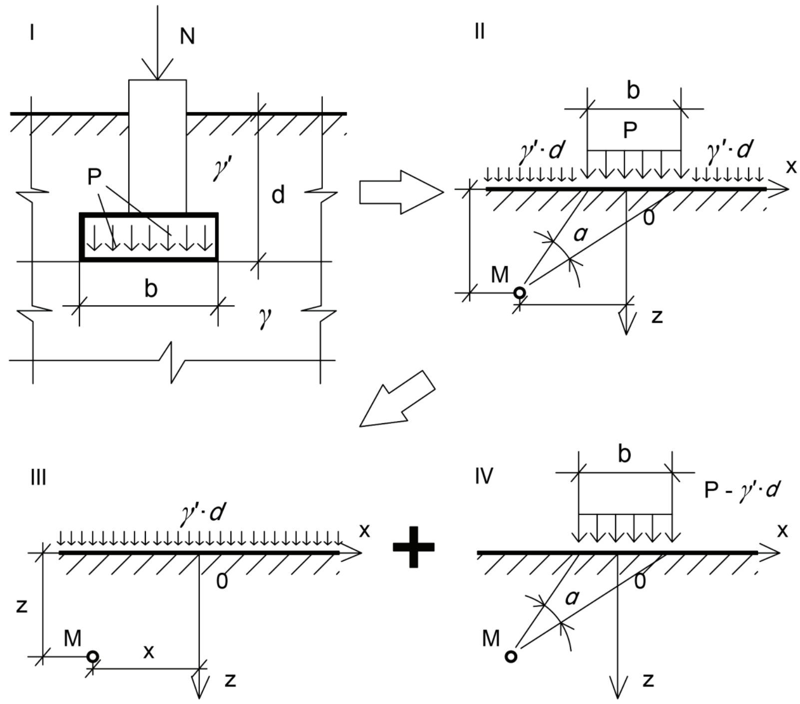

Fig. 1. On determining the initial critical load on soil, as well as the standardized and estimated base resistances: I – system «base – foundation»; II – the corresponding estimation scheme; III, IV – components of the estimation scheme

Here, α is the angle of visibility (Fig. 1, IV), which should be taken in radians, and the «*m*» coefficient is equal to unity and has dimensionality 1/radian. Hereafter, this coefficient is omitted.

In formula (6), the «IV» indices indicate the correspondence of the principal stresses, calculated by using it, to the estimation scheme shown in Fig. 1, IV. Since we determine the initial critical pressure on soil (it typically occurs at one or more points in the base), the linear medium is investigated, so the principle of superposition is applicable. Therefore, the principal stresses acting in a base can be found as the sum of stresses corresponding to the estimation schemes in Fig. 1, III, IV (formulae (5) and (6) respectively). We obtain:

$$\left.\begin{aligned}\sigma_1=\sigma_{1,\mathrm{III}}+\sigma_{1,\mathrm{IV}}=\gamma'\cdot d+\gamma\cdot z+\frac{-\gamma'\cdot d}{\pi}\cdot\left[\alpha+\sin(\alpha)\right];\\ \sigma_3=\sigma_{3,\mathrm{III}}+\sigma_{3,\mathrm{IV}}=\gamma'\cdot d+\gamma\cdot z+\frac{-\gamma'\cdot d}{\pi}\cdot\left[\alpha-\sin(\alpha)\right].\end{aligned}\right\}\quad(7)$$

Next, substitute (7) in the strength condition by Mohr-Coulomb, which for the spatial case takes the form:

$$\left.\begin{aligned}\frac{\sigma_1-\sigma_3}{\sigma_1+\sigma_3+2\cdot c\cdot\mathrm{ctg}(\varphi)}\le\sin(\varphi),\\ \sigma_1>\sigma_2>\sigma_3.\end{aligned}\right\}\quad(8)$$

Here, c is the specific cohesion, φ is the angle of soil internal friction.

We obtain:

$$\frac{\sin(\varphi)\cdot\left\{\begin{aligned}&\langle c\cdot\cos(\varphi)+[\gamma'\cdot d+\gamma\cdot z]\cdot\sin(\varphi)\rangle\cdot\pi+\\&+[\sin(\varphi)\cdot\alpha-\sin(\alpha)]\cdot P\end{aligned}\right\}}{\langle c\cdot\cos(\varphi)+[\gamma'\cdot d+\gamma\cdot z]\cdot\sin(\varphi)\rangle\cdot\pi+\sin(\varphi)\cdot\alpha\cdot P}=0.\quad(9)$$

Next, find, from (9), the maximum depth $z_{\max}$ at which there is critical pressure. To this end, first determine, from (9), depth z. We obtain:

$$z=-\frac{\sin(\varphi)\cdot\alpha-\sin(\alpha)}{\sin(\varphi)\cdot\gamma\cdot\pi}\cdot P-\\-\frac{c\cdot\cos(\varphi)+\sin(\varphi)\cdot\gamma'\cdot d}{\sin(\varphi)\cdot\gamma}.\quad(10)$$

Next, find the value of angle α at which depth z is maximum. We obtain:

$$\frac{\partial z}{\partial\alpha}=-\frac{\sin(\varphi)-\cos(\alpha)}{\sin(\varphi)\cdot\gamma\cdot\pi}\cdot P=0,\quad(11)$$

hence

$$\alpha=\frac{\pi}{2}-\varphi.\quad(12)$$

Next, find, from (10), the critical pressure on soil $P^{i.cr.}$ corresponding to depth $z_{\max}$. In this case, one should take into consideration that the value for pressure P, derived from (10), should be added with pressure due to the distributed load, numerically equal to $P_1=\gamma_1\cdot d$. We obtain:

$$P^{i.cr.}=\frac{\pi\cdot\left[\gamma'\cdot d+c\cdot\mathrm{ctg}(\varphi)\right]}{\mathrm{ctg}(\varphi)+\varphi-\pi/2}+\gamma'\cdot d.\quad(13)$$

Formula (13) poorly corresponds to the field data on observations of foundation settling. In particular, it was found that when a zone of plastic deformations increases to a depth of $z=b/4$, the «settling – load» dependence takes a straightforward form. In addition, the initial critical load on a base, established from (13), does not depend on the specific weight of soil below the sole of the foundation, which also contradicts the experimental data. Therefore, at present, the critical load R^b (also denoted as the standardized estimated soil resistance) is determined by adopting (9) in the form:

$$\frac{b}{4}=-\frac{\sin(\varphi)\cdot\alpha-\sin(\alpha)}{\sin(\varphi)\cdot\gamma\cdot\pi}\cdot P-\frac{c\cdot\cos(\varphi)+\sin(\varphi)\cdot\gamma'\cdot d}{\sin(\varphi)\cdot\gamma},\quad(14)$$

hence:

$$\left.\begin{aligned}P=R^b=\frac{\pi\cdot\left[0.25\cdot\gamma\cdot b+\gamma'\cdot d+c\cdot\mathrm{ctg}(\varphi)\right]}{\mathrm{ctg}(\varphi)+\varphi-\pi/2}+\gamma'\cdot d=\\=M_\gamma\cdot\gamma\cdot d+M_q\cdot\gamma'\cdot d+M_c\cdot c;\\ M_\gamma=\frac{0.25\cdot\pi}{\mathrm{ctg}(\varphi)+\varphi-\pi/2};\\ M_q=\frac{\pi}{\mathrm{ctg}(\varphi)+\varphi-\pi/2}+1;\\ M_c=\frac{\pi\cdot\mathrm{ctg}(\varphi)}{\mathrm{ctg}(\varphi)+\varphi-\pi/2}.\end{aligned}\right\}\quad(15)$$

Formula (15) also does not accurately determine the estimated resistance of base R^b, as it is not possible, when using it, to take into consideration such parameters of the actual systems «soil base – foundation (or foundations) – a structure above foundation» as:

– a scale effect;

– structural features of the system;

– eatures in the manifestation of properties of different types of soils under load;

– the non-linear dependence of a Mohr-Coulomb envelope on the principal stresses.

In this regard, acting regulatory documents imply that the estimated base resistance is calculated using the following formula:

$$\left.\begin{aligned} R&=\frac{\gamma_{c1}\cdot\gamma_{c2}}{k}\cdot\left[M_\gamma\cdot k_z\cdot\gamma\cdot d+M_q\cdot\gamma'\cdot d+M_c\cdot c\right];\\ M_\gamma&=\frac{0.25\cdot\pi}{\mathrm{ctg}(\varphi)+\varphi-\pi/2};\\ M_q&=\frac{\pi}{\mathrm{ctg}(\varphi)+\varphi-\pi/2}+1;\\ M_c&=\frac{\pi\cdot\mathrm{ctg}(\varphi)}{\mathrm{ctg}(\varphi)+\varphi-\pi/2},\end{aligned}\right\}\quad(16)$$

where γ_{c1} and γ_{c2} are empirical coefficients that depend on the type and condition of soil, as well as the features of a structure above a foundation; k is a factor that takes into consideration a technique for determining specific weight and soil strength properties; k_z is a factor that takes into consideration the scale effect.

Experience has shown that even the use of a semi-empirical formula (16) to calculate the settling of foundations does not make it possible to obtain satisfactory results. In this regard, of considerable interest are the results from study [16] that compared the actual S_{act} and estimated S_{est} resulting (stabilized) settling of 143 objects. In accordance with the accepted categorization of buildings and structures, it was found that the discrepancy between the estimated and actual settling of the bases of buildings and structures is about 50 %.

Thus, the results of forecasting the settling of buildings and structures using the methodology acting in Ukraine are unsatisfactory.

In our opinion, this is predetermined by the following reasons:

1. The above-established values of initial critical load $P^{i.cr.}$ on soil, as well as the standardized R^{st} and estimated R^{est} base resistances, are not the extrema of function of two variables α and z.

2. The Mohr-Coulomb strength law produces inflated values for a shear strength (Fig. 2).

To illustrate the statement outlined in point 1, let us solve (9) with respect to pressure P and find an extremum of the function of variables α and z obtained in this way.

We obtain:

$$P=\frac{\sin(\varphi)\cdot\gamma\cdot\pi}{\sin(\alpha)-\sin(\varphi)\cdot\alpha}\cdot z-\pi\cdot\frac{\sin(\varphi)\cdot\gamma'\cdot d+c\cdot\sin(\varphi)}{\sin(\alpha)-\sin(\varphi)\cdot\alpha}.\quad(17)$$

The prerequisite for an extremum is to meet the conditions:

$$\left.\begin{aligned}\frac{\partial P}{\partial z}&=\frac{\sin(\varphi)\cdot\gamma\cdot\pi}{\sin(\alpha)-\alpha\cdot\sin(\varphi)}=0;\\ \frac{\partial P}{\partial\alpha}&=\pi\left\langle\begin{aligned}&-\frac{\sin(\varphi)\cdot\gamma\cdot z\cdot\left[\cos(\alpha)-\sin(\varphi)\right]}{\sin(\alpha)-\alpha\cdot\sin(\varphi)}+\\&+\frac{\left[\sin(\varphi)\cdot\gamma'\cdot d+c\cdot\cos(\varphi)\right]\left[\cos(\alpha)-\sin(\varphi)\right]}{\sin(\alpha)-\alpha\cdot\sin(\varphi)}\end{aligned}\right\rangle=0.\end{aligned}\right\}\quad(18)$$

Since the equation system (16) has no valid roots, there is no any extremum of function (17) for variables α and z at all. Therefore, solutions (12), (14) and (15) are not mathematically rigorous.

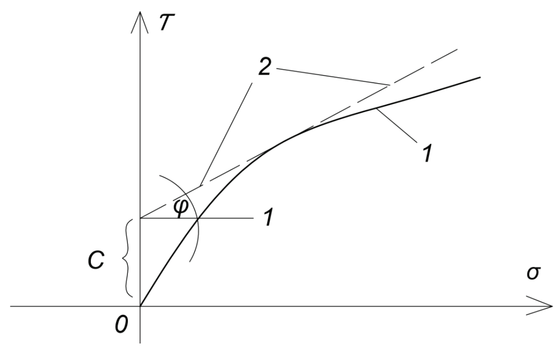

Fig. 2. Actual envelope by Mohr-Coulomb (curve 1) and its linearization (straight line 2)

To take into consideration the non-linearity of dependence of the Mohr-Coulomb envelope, we shall use the condition of strength proposed by A. Shashenko, which, in a one-dimensional case, takes the form [1, 15]:

$$\tau\le\sqrt{c\cdot\left[2\cdot P\cdot\mathrm{tg}(\varphi)+c\right]},\quad(19)$$

where τ is the destructive tangential stress; P is the vertical normal stress; φ is the angle of internal friction; c is the specific cohesion.

In order to be able to determine the critical load $P^{i.cr.}$ on soil, as well as the standardized R^{st} and estimated R^{est} resistances, by using ready solutions (12), (14) and (15), we shall perform linearization (19). To this end, expand (19) into a Taylor series in the vicinity of point:

$$P=P_0,\quad(20)$$

where P_0 is the mean pressure under the sole of a foundation. In this case, let us confine ourselves to the first degree of a polynomial. We obtain:

$$\tau\approx\sqrt{c\cdot\left[2\cdot P_0\cdot\mathrm{tg}(\varphi)+c\right]}+\frac{c\cdot\mathrm{tg}(\varphi)\cdot(P-P_0)}{\sqrt{c\cdot\left[2\cdot P_0\cdot\mathrm{tg}(\varphi)+c\right]}}.\quad(21)$$

Next, assume in (21):

$$\left.\begin{aligned}c^*&=\frac{c\cdot\left[P_0\cdot\mathrm{tg}(\varphi)+c\right]}{\sqrt{c\cdot\left[2\cdot P_0\cdot\mathrm{tg}(\varphi)+c\right]}};\\ \varphi^*&=\mathrm{arctg}\left[\frac{c\cdot\mathrm{tg}(\varphi)}{\sqrt{c\cdot\left[2\cdot P_0\cdot\mathrm{tg}(\varphi)+c\right]}}\right].\end{aligned}\right\}\quad(22)$$

Then (21) takes the form:

$$\left.\begin{aligned} \tau &= P\cdot \mathrm{tg}\left(\varphi^{*}\right)+c^{*};\\ c^{*} &= \frac{c\cdot\left[\ {}_{0}\cdot \mathrm{tg}(\varphi)+c\right]}{\sqrt{c\cdot\left[2\cdot P_0\cdot \mathrm{tg}(\varphi)+c\right]}};\\ \varphi^{*} &= \mathrm{arctg}\left[\frac{c\cdot \mathrm{tg}(\varphi)}{\sqrt{c\cdot\left[2\cdot P_0\cdot \mathrm{tg}(\varphi)+c\right]}}\right]. \end{aligned}\right\} \quad (23)$$

Upper equality (23) completely coincides in the form with a known condition of strength by Mohr-Coulomb. Therefore, by using (23), the initial critical load on a base – equality (12) – can be represented in the form:

$$\left. P^{i.cr} = \frac{\pi\cdot\left[\gamma_1\cdot d + c^{*}\cdot \mathrm{ctg}\left(\varphi^{*}\right)\right]}{\mathrm{ctg}\left(\varphi^{*}\right)+\varphi^{*}-\pi/2}+\gamma'\cdot d;\right\} \quad (24)$$

$$\left.\begin{aligned} c^{*} &= \frac{c\cdot\left[P_0\cdot \mathrm{tg}(\varphi)+c\right]}{\sqrt{c\cdot\left[2\cdot P_0\cdot \mathrm{tg}(\varphi)+c\right]}};\\ \varphi^{*} &= \mathrm{arctg}\left[\frac{c\cdot \mathrm{tg}(\varphi)}{\sqrt{c\cdot\left[2\cdot P_0\cdot \mathrm{tg}(\varphi)+c\right]}}\right]. \end{aligned}\right\} \quad (25)$$

In this case, the standardized estimated base resistance R^b (15) will be equal to:

$$\left.\begin{aligned} R^{b} &= \frac{\pi\cdot\left[0.25\cdot\gamma\cdot b+\gamma'\cdot d + c^{*}\cdot \mathrm{ctg}\left(\varphi^{*}\right)\right]}{\mathrm{ctg}\left(\varphi^{*}\right)+\varphi^{*}-\pi/2}+\gamma'\cdot d =\\ &= M_\gamma\cdot\gamma\cdot d + M_q\cdot\gamma'\cdot d + M_c\cdot c;\\ M_\gamma &= \frac{0.25\cdot\pi}{\mathrm{ctg}\left(\varphi^{*}\right)+\varphi^{*}-\pi/2};\\ M_q &= \frac{\pi}{\mathrm{ctg}\left(\varphi^{*}\right)+\varphi^{*}-\pi/2}+1;\\ M_c &= \frac{\pi\cdot \mathrm{ctg}\left(\varphi^{*}\right)}{\mathrm{ctg}\left(\varphi^{*}\right)+\varphi^{*}-\pi/2};\\ c^{*} &= \frac{c\cdot\left[P_0\cdot \mathrm{tg}(\varphi)+c\right]}{\sqrt{c\cdot\left[2\cdot P_0\cdot \mathrm{tg}(\varphi)+c\right]}};\\ \varphi^{*} &= \mathrm{arctg}\left[\frac{c\cdot \mathrm{tg}(\varphi)}{\sqrt{c\cdot\left[2\cdot P_0\cdot \mathrm{tg}(\varphi)+c\right]}}\right], \end{aligned}\right\} \quad (26)$$

and the estimated base resistance –

$$\left.\begin{aligned} R &= \frac{\gamma_{c1}\cdot\gamma_{c2}}{k}\left(M_\gamma\cdot\gamma\cdot d + M_q\cdot\gamma'\cdot d + M_c\cdot c\right);\\ M_\gamma &= \frac{0.25\cdot\pi}{\mathrm{ctg}\left(\varphi^{*}\right)+\varphi^{*}-\pi/2};\\ M_q &= \frac{\pi}{\mathrm{ctg}\left(\varphi^{*}\right)+\varphi^{*}-\pi/2}+1;\\ M_c &= \frac{\pi\cdot \mathrm{ctg}\left(\varphi^{*}\right)}{\mathrm{ctg}\left(\varphi^{*}\right)+\varphi^{*}-\pi/2}; \end{aligned}\right\} \quad (27)$$

$$\left.\begin{aligned} c^{*} &= \frac{c\cdot\left[P_0\cdot \mathrm{tg}(\varphi)+c\right]}{\sqrt{c\cdot\left[2\cdot P_0\cdot \mathrm{tg}(\varphi)+c\right]}};\\ \varphi^{*} &= \mathrm{arctg}\left[\frac{c\cdot \mathrm{tg}(\varphi)}{\sqrt{c\cdot\left[2\cdot P_0\cdot \mathrm{tg}(\varphi)+c\right]}}\right]. \end{aligned}\right\} \quad (28)$$

Next, we shall analyze how the use of the condition of strength by A. Shashenko would affect the results of determining the initial critical pressure on soil and the standardized base resistance.

Assume that by using the DSTU procedure, we derived, in the pressure interval 50...150 kPa (mean pressure P_0=100 kPa), the following values of strength characteristics: specific cohesion c^{*}=25 kPa and the angle of internal friction φ^{*}=9°. Such strength characteristics are typical of weak soils [17]. Next, substitute these values in the last two equalities from equation system (26). We obtain:

$$\left.\begin{aligned} &\frac{c\cdot\left[P_0\cdot \mathrm{tg}(\varphi)+c\right]}{\sqrt{c\cdot\left[2\cdot P_0\cdot \mathrm{tg}(\varphi)+c\right]}} = 25;\\ &\mathrm{arctg}\left[\frac{c\cdot \mathrm{tg}(\varphi)}{\sqrt{c\cdot\left[2\cdot P_0\cdot \mathrm{tg}(\varphi)+c\right]}}\right] = 9. \end{aligned}\right\} \quad (29)$$

A solution to equation system (29) is the following strength characteristics values, which are included in the condition of strength by A. Shashenko: c=20 kPa and φ=18°, so that the last two equalities (26) will take the form:

$$\left.\begin{aligned} c^{*} &= \frac{20\cdot\left[P_0\cdot \mathrm{tg}(18)+20\right]}{\sqrt{20\cdot\left[2\cdot P_0\cdot \mathrm{tg}(18)+20\right]}};\\ \varphi^{*} &= \mathrm{arctg}\left[\frac{20\cdot \mathrm{tg}(18)}{\sqrt{20\cdot\left[2\cdot P_0\cdot \mathrm{tg}(18)+20\right]}}\right]. \end{aligned}\right\} \quad (30)$$

Formulae (30) link the strength constants from the linear law of strength of the condition of strength by Mohr-Coulomb c^{*} and φ^{*} with material constants c and φ in the failure criterion by A. Shashenko.

5. Results from studying the effect of non-linear dependence of soil strength properties on pressure on a base

Next, a numerical experiment was performed whose essence was to compare the initial critical pressure and the critical pressure on a base, calculated according to the regulatory documents [12, 13] procedure and taking into consideration the non-linear dependence of soil strength properties on pressure.

In a first case, known formulae (12) and (14) were used, and in a second case – the derived formulae (25), (26).

Fig. 3 shows in a graphic form dependences (30) in the pressure interval P_0=100...500 kPa.

In all cases, the depth of laying a foundation's sole was taken equal to d=2.0 meters; the width of a foundation is b=1.7 meters; the specific weight of soil is above the sole of a foundation is γ_1=18 kN/m^3, and the specific weight of soil lower than the sole of a foundation is γ=20 kN/m^3.

In this case, the range of pressure change under the sole of a foundation was accepted to be equal to: $P_0 \in (100...500)$ kPa, because mean pressure under the sole of foundations of actual buildings and structures is within the limits specified in [1, 14].

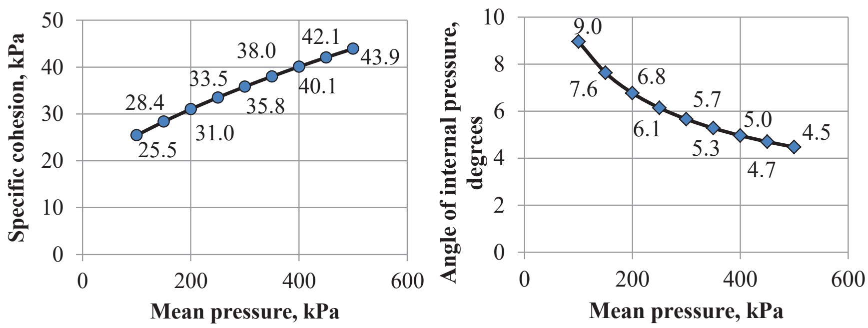

Fig. 3. Relationship between the strength material constants, which are part of the strength conditions by Mohr-Coulomb and A. Shashenko: *a* – specific cohesion c^*; *b* – angle of internal pressure φ^*. *Note: Values of parameters included in the condition of strength by A. Shashenko are equal to: c=20 kPa; φ= 18°*

Conditions for the experiment were stated as follows:

1. Material constants for the failure criterion by Mohr-Coulomb were calculated, by using equality (25), for pressures P_0=100 kPa and P_0= =500 kPa. In a first case, they were equal to c^*=25 kPa and φ^*=9°, and in a second case – c^*=44 kPa and φ^*=4°.

2. Next, by using a Mohr-Coulomb failure criterion, we calculated the initial critical and critical loads on a base (formulae (11) and (13)).

3. Next, the initial critical and critical loads on a base were calculated by using a failure criterion by A. Shashenko (formulae (22) and (23)).

The resulting dependences in a graphic form are shown in Fig. 4.

Analysis of the curves shown in Fig. 4 has made it possible to draw the following conclusions.

1. While the strength properties of soil were determined in the range of small pressures, the dependences $P^{i.cr}=f(P_0)$ and $R^b=f_1(P_0)$, calculated in line with a conventional procedure, take the form of a line parallel to the abscissa axis. In other words, in this case, these characteristics do not depend on the mean pressure under the sole of a foundation.

In this case, the data on pressure, except for one point (P_0=100 kPa), are less than those calculated using the failure criterion by A. Shashenko (formulae (22), (23), as well as rows 3 in Fig. 4, *a*, *b*).

Small pressures in this case are understood to be pressures in the range $P_0 \in (50...150)$ kPa and their mean value P_0=100 kPa.

2. While the strength properties of soil were determined in the range of large pressures, the dependences $P^{i.cr.}=f(P_0)$ and $R^b=f_1(P_0)$, calculated in line with a conventional procedure, take the form of a line parallel to the abscissa axis. In other words, in this case, these characteristics do not depend on the mean pressure under the sole of a foundation.

In this case, the data on pressure, except for one point (P_0=500 kPa), are larger than those calculated by using the failure criterion by A. Shashenko (formulae (25) and (26), as well as rows 3 in Fig. 4, *a*, *b*).

The large pressures in this case are pressures in the range $P_0 \in$ $\in (450...550)$ kPa and their mean value P_0=500 kPa.

3. Thus, when using generally accepted estimation formulae (12) and (14) to determine critical loads on a base, it is necessary to take into consideration the range of pressures at which the soil strength properties were determined.

4. In this case, the use of a failure criterion by A. Shashenko for determining critical loads on a base makes it possible to correctly account for the influence exerted on them by mean pressure under the sole of a foundation.

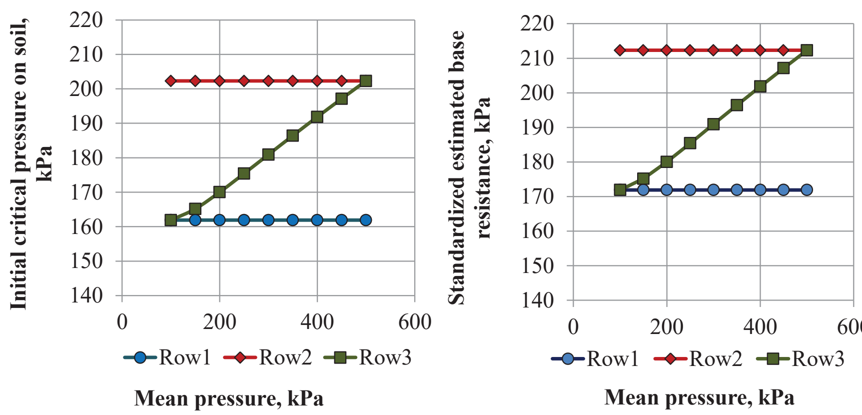

Fig. 4. Effect of mean pressure under the sole of a foundation: *a* – on the initial critical pressure; *b* – on the standardized estimated base resistance; 1 – calculation using the material constants from the strength condition by Mohr-Coulomb, determined at P_0=100 kPa; row 2 – the same, at P_0=500 kPa; row 3 – the same, using the strength condition by A. Shashenko

6. Discussion of results from studying the effect of soil strength properties on the estimated base resistance

In general, the following was established:

1. The theoretical results obtained are predetermined by accounting for the non-linearity of dependences of soil strength properties on normal pressure in the examined region (Fig. 3).

2. That has made it possible to reflect the effect exerted on the initial critical pressure, the standardized and estimated resistances of soil, by the mean pressure under the sole of a foundation. In other words, we managed to link a well-known fact of the non-linearity of a Mohr-Coulomb envelope and the critical loads on a base.

3. This fact, in contrast to the formulae adopted in the regulatory documents (including Ukrainian), makes it possible to properly account for the impact exerted on the initial critical pressure, the standardized and estimated resistance of soil, by the mean pressure under the sole of a foundation.

It is worth noting that this relationship is not currently taken into consideration.

4. A limitation for the application of the proposed approach is the linear form of an experimental envelope by Mohr-Coulomb (theoretically, this option does exist).

5. A drawback of the proposed formulae is their cumbersome form, compared to the formulae adopted in the Ukrainian regulatory documents.

In conclusion, note that the failure criterion by A. Shashenko is not the only criterion that reflects the non-linear relationship between the strength properties of soil and the normal pressure acting on soil. Other failure criteria should therefore be considered in the further research. That would make it possible to obtain the most acceptable solution, in terms of various types of soil, to the problem considered in this study.

7. Conclusions

1. We have derived new formulae for determining the initial critical load on a base, taking into consideration the non-linear dependence of soil strength characteristics (rock) on the mean pressure under the sole of a foundation. To account for the nonlinearity, a failure criterion by A. Shashenko was used. In the course of a numerical experiment, it was found that depending on pressure under the sole of a foundation, values of the initial critical load, calculated by using the proposed procedure and in line with regulatory documents [12, 13] methodology, differ by 0...25 %.

2. By applying the non-linear failure criterion by A. Shashenko, we have derived the estimation formulae that make it possible to determine the standardized estimated base resistance. That has made it possible to take into consideration the non-linearity of dependence of a Mohr-Coulomb envelope on the vertical normal load on soil (in the case in question, this load is the mean pressure under the sole of a foundation). In the course of a numerical experiment, it was found that depending on pressure under the sole of a foundation, values for the standardized estimated base resistance, calculated by using the proposed procedure and the regulatory documents [12, 13] methodology, differ by 0...23 %.

3. New formulae have been obtained to determine the estimated base resistance. Therefore, in a given case, the estimated base resistance also makes it possible to reflect the non-linearity of dependence of a Mohr-Coulomb envelope on the vertical normal load on soil. In conclusion, note that the values of empirical coefficients for operational conditions are given in the Ukrainian regulatory documents. In the course of a numerical experiment, it was found that depending on pressure under the sole of a foundation, values of the estimated base resistance, calculated by using the proposed procedure and the regulatory documents [12, 13] methodology, differ by 0...23 %.

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ESTIMATION OF THE AERODYNAMIC CHARACTERISTICS OF A STEPPED NACELLE FOR THE AIRCRAFT POWERPLANT

Yu. Yu. Tereshchenko
PhD*
E-mail: Terj-nau@ukr.net
Yu. M. Tereshchenko
Doctor of Technical Sciences, Professor*
E-mail: Terj@nau.edu.ua
A. Sklyarov
PhD, Senior Researcher
Research Department No. 22
State Aviation Research Institute
Andriushchenka str., 6v, Kyiv, Ukraine, 01135
E-mail: fly55@ukr.net
E. Doroshenko
Doctor of Technical Sciences, Associate Professor*
E-mail: Kiki_ua@ukr.net
P. Humeniuk
Postgraduate student*
E-mail: seafrise@gmail.com
*Department of Aviation Engine
National Aviation University
Kosmonavta Komarova ave., 1, Kyiv, Ukraine, 03058

Метою роботи є оцінка аеродинамічних характеристик ступінчастої мотогондоли газотурбінного двигуна з турбовентиляторною приставкою. Для проведення досліджень використовувався метод модельного фізичного експерименту. Аеродинамічна труба, в якій було проведено дослідження, забезпечена необхідним обладнанням, що включає в себе різні насадки статичного і динамічного тиску з координатними пристроями та ін. Для експериментальних досліджень було створено моделі мотогондол авіаційної силової установки з переднім розташуванням модуля вентилятора та з заднім розташуванням турбовентиляторної приставки. Проведено експериментальні дослідження аеродинамічних характеристик ступінчастої мотогондоли газотурбінного двигуна з турбовентиляторною приставкою.

Результати дослідження показали можливість зниження аеродинамічного опору ступінчастої мотогондоли двигуна з турбовентиляторною приставкою в порівнянні з мотогондолою турбореактивного двоконтурного двигуна з переднім розташуванням вентилятора. В діапазоні кутів атаки α=0...20° значення аеродинамічного опору ступінчастої мотогондоли для газотурбінного двигуна з турбовентиляторною приставкою знижується на 49...55 %.

Отримані результати показали, що коефіцієнт підйомної сили ступінчастої мотогондоли газотурбінного двигуна з турбовентиляторною приставкою збільшується на 24...64 %. Коефіцієнт аеродинамічного опору нижче на 18...28 % у порівнянні з коефіцієнтом аеродинамічного опору циліндричної мотогондоли двоконтурного турбореактивного двигуна в діапазоні кутів атаки α=2...20°. Отримані результати свідчать про перспективність використання двигунів з турбовентиляторною приставкою. Конструкційна особливість ступінчастої мотогондоли дозволить зменшити втрати ефективної тяги двигуна за рахунок зниження аеродинамічного опору майже в два рази і підвищити паливну економічність двигуна

Ключові слова: ступінчаста мотогондола, аеродинамічний опір, подйомна сила, газотурбінний двигун, турбовентиляторна приставка

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1. Introduction

Technical perfection of civilian aircrafts is determined by the aerodynamic layout of an aircraft, power unit, development of new materials, implementation of modern equipment and control systems.

The most important direction of improving the aerodynamics of mainline aircraft is minimizing the aerodynamic drag of an aircraft's elements. Among the essential issues related to this field is the optimization of the shape and location of nacelles, whose resistance is 1...5 % of the total aerodynamic drag of a plane. In addition, reducing the aero-