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CALCULATION OF THE BEARING CAPACITY AND STABILITY OF THE BASE OF A VERTICALLY LOADED SINGLE PILE BY PRESSING (PULLING) FORCE BASED ON THE RESULTS OF SOIL TESTS

ABSTRACT

The article discusses modern practical approaches to the calculation of pile bases, gives the actual picture of their deformation and loss of stability under the influence of vertical pressing/ pulling loads. It is stated that the currently used methods for assessing the load-bearing capacity and stability of the pile base (theoretical, engineering-practical, numerical), based on simplified idealized models, the theory of solid body elasticity, do not correspond to the actual state, properties and behaviour of dispersed soils under load. This greatly reduces the reliability of the results obtained for them (the difference between the calculated and experimental data reaches 100 %), and therefore they need to be clarified. Therefore, the aim of this work is to improve the accuracy and reliability of calculation methods by taking into account the actual operation of piles in the ground and its properties. Analysis of experimental data showed that the loss of stability of the pile base at the stage of exhaustion of its load-bearing capacity well corresponds to the assumptions and principles adopted in the theory of ultimate soil equilibrium. In this regard, the article offers a theoretical solution that develops the provisions of the theory of the ultimate stress state of soils in relation to the assessment of the bearing capacity and stability of the Foundation of piles. The solution is obtained for the condition of a flat problem when loading a multi-layer base of a *pile with a vertical pressing force. In the design scheme, the following basic assumptions are made:*

- the loss of stability of the pile base occurs as a result of shifts on the sliding surface of an undisturbed volume of soil in the form of a truncated cone, including the pile, relative to stationary soil;

- sliding surfaces have constant faces close to rectilinear, which are rigidly oriented in space by angles of inclination to the vertical-β_i*, along the length of the trunk, and* α_i*-to the horizontal, at the level of the end of the pile;*

- the maximum normative load on the pile is determined under the condition of the maximum ordinate of the development of the limit equilibrium region $z_{max} = 0.25d$*, which corresponds to the critical draft* $s_{кр} \leq \xi\, s_{u.}$*.*

The proposed theoretical solution can be used to develop methods for assessing the load-bearing capacity and stability of both finished and Packed piles of various types, for which partial coefficients of the working condition are established, depending on their parameters, soil properties and manufacturing method.

Keywords: theory of ultimate stress equilibrium of the soil, particular problem of stability of the pile base, sliding surface, shear zone, ultimate draft.

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РАСЧЕТ НЕСУЩЕЙ СПОСОБНОСТИ И УСТОЙЧИВОСТИ ОСНОВАНИЯ ВЕРТИКАЛЬНО НАГРУЖЕННОЙ ОДИНОЧНОЙ СВАИ ВДАВЛИВАЮЩЕЙ (ВЫДЕРГИВАЮЩЕЙ) СИЛОЙ ПО РЕЗУЛЬТАТАМ ИСПЫТАНИЙ ГРУНТОВ

АННОТАЦИЯ

В статье рассматриваются современные практические подходы к расчету оснований свай, даны фактическая картина их деформирования и потери устойчивости при воздействии вертикальной вдавливающей/выдергивающей нагрузки. Констатируется, что используемые в настоящее время методы оценки несущей способности и устойчивости основания свай (теоретические, инженерно-практические, численные), построенные на упрощенных идеализированных моделях, теории упругости твердого тела, не соответствуют фактическому состоянию, свойствам и поведению дисперсных грунтов под нагрузкой. Это сильно снижает достоверность полученных по ним результатов (разница между рассчитанными и опытными данными достигает 100 %), в связи с чем они требуют уточнения. Поэтому целью работы является повышение точности и надежности расчетных методов посредством учета фактической работы свай в грунте и его свойств. Анализ опытных данных показал, что потеря устойчивости основания свай на стадии исчерпания его несущей способности хорошо соответствует допущениям и принципам, принятым в теории предельного равновесия грунтов. В связи с этим в статье предложено теоретическое решение, развивающее положения теории предельного напряженного состояния грунтов применительно к оценке несущей способности и устойчивости основания свай. Решение получено для условия плоской задачи при нагружении многослойного основания сваи вертикальной вдавливающей силой. В расчетной схеме приняты следующие основные допущения:

- потеря устойчивости основания сваи происходит в результате сдвигов по поверхности скольжения ненарушенного объема грунта в форме усеченного конуса, включающего сваю, относительно неподвижного грунта;

- поверхности скольжения имеют постоянные близкие к прямолинейным грани, жестко ориентированные в пространстве углами наклона к вертикали – β_i, *по длине ствола, и* α_i *– к горизонтали, в уровне конца сваи;*

- предельная нормативная нагрузка на сваю определяется при максимальной ординате развития области предельного равновесия $z_{max} = 0{,}25d$, *что соответствует критической осадке* $s_{кр} \le \xi\, s_u$.

Предложенное теоретическое решение может быть применено для разработки методов оценки несущей способности и устойчивости как готовых, так и набивных свай различных типов, для которых установлены частные коэффициенты условия работы, зависящие от их параметров свойств грунтов и способа изготовления.

Ключевые слова: теория предельного напряженного равновесия грунта, частная задача устойчивости основания сваи, поверхность скольжения, зона сдвигов, предельная осадка.

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INTRODUCTION

Comparative analysis of the piles bearing capacity F_k, obtained on the basis of experimental data, with the results of its calculations by well-known theoretical [1–3, etc.] methods based on solutions of the theory of elasticity of a solid, shows that there is a significant discrepancy between them. More reliable results are provided by engineering and practical calculation methods regulated by the norms [4–6, etc.] and official documents (manuals, recommendations [7–9, etc.]), using tabular values of the calculated soil resistance to indentation / pulling out into / out of its lower end (heel) R_b and trunk R_s of the piles. However, these methods are also not accurate enough, since do not take into account the state and physical and mechanical properties of soils, including in the process of their manufacture and operation.

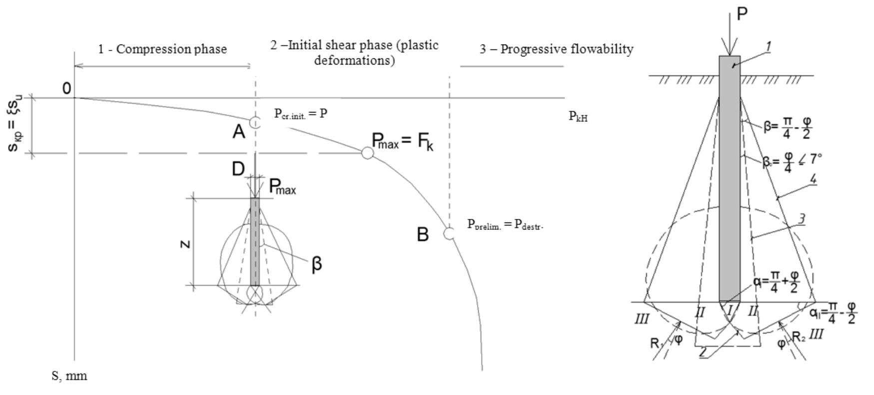
The deviations of the F_k values found from the results of calculations using known existing methods [4–8, etc.] from the experimental data reach 40 % for long piles and up to 100 % or more for short ones, both when they are indented and pulled out. This indicates that the existing methods for determining the bearing capacity of pile foundations (theoretical, empirical, engineering and practical) need to be clarified.

In this regard, the purpose of this work is to increase the accuracy and reliability of the assessment of the bearing capacity (the first group of limiting states) and the stability of vertically loaded pile foundations, by taking into account the experimentally established features of their interaction with the soil, the state and actual physical and mechanical properties of soils based on test results with pressing and pulling.

EXPERIMENTAL AND THEORETICAL PRECONDITIONS FOR CALCULATION

Analysis of experimental studies based on literary sources [7–8, etc.] and our own ones [10, etc.] shows that when acting on the base of the foundation (pile), continuously increasing both pressing and pulling loads, its work at various stages of loading is characterized by three phases: 1 – compaction, 2 – shear from initial to increasing massive (plastic deformations), passing (3rd phase) into progressive soil fluidity (Figure 1).

a b



1 – pile; 2 – the edge of the sliding surface (lift) under the lower end of the pile in the limiting state; 3 – the same along the trunk; 4 – outer boundary of the core of the pile foundation; α and β – angles of inclination of surfaces positions (1–3); R_1 and R_2 – soil resistance to pile indentation; I – rigid core under the end of the pile; II – zone of passive soil shear resistance; III – zones of stationary soil relative to which zones II are shifted.

Figure 1. Stress-strain state of the base of a single pile under loading by pressing force a – averaged graph of the dependence of the draft s on the vertical pressing load P according to the results of tests of experimental piles in soils of medium strength; b – diagram of the limiting equilibrium of a homogeneous base of a single pile at $z \geq 2D$.

On the graph s = f (p) in Figure 1a, three sections can be distinguished: the first is close to a straight line, characterized by an almost linear relationship between the draft (lift) of the pile and the force applied to the base corresponding to the critical load $p_{cr.init.}$ at $s_{cr} = f(p_{cr})$, the second - with a violation of the linear (proportion) dependence of the draft-load and an increase in the initial critical force, $p_{cr.init.}$ up to the maximum critical $p_{max\,cr.}$, the third - by an increase in draft at constant values of the force of the speed of pressing (extraction) of the pile (the fluid state of the foundation soil).

In soil mechanics (the general theory of limiting equilibrium), the strength of the foundation is usually evaluated by the initial critical and limiting pressures on it, at which there is still no loss of soil stability [1, 3, 11, 12].

Initial critical pressure $p_{cr.init.}$ corresponds to the boundary between phases I and II (point A in Figure 1a), to which compaction deformations that are safe for the stability (strength) of soils prevail. Exceeding this pressure causes local loss of soil stability. Therefore, the average pressure of the foundation of buildings and structures of the I-th level of responsibility, obviously, should not exceed the critical $p_{cr.init.}$, taken as the proportionality limit between pressure and soil deformation – $p_{cr.init.} \approx p_{prop.}$.

Ultimate pressure $p_{ult.}$ corresponds to the full use of the resistance of the base to shear (point B in Figure 1a) and the loss of its stability ($p_{ult.} \approx p_{destr.}$).

The position of the critical point A on the pile indentation curve (see Figure 1) determines both the maximum permissible (critical) deformation s_{cr} and the value of soil resistance corresponding to it, expressed by the abscissa of the critical pressure $p_{cr.}$. This resistance in modern design practice is usually called the concept of **the bearing capacity of the soil**, which has a rather vague meaning, since on the pile indentation graph, as a rule, it is not expressed sharply and is set conditionally. To give this concept a physical meaning, the bearing capacity $F_k \leq R_k$ in the normative literature [4, 5, etc.], it is usually customary to assign at a draft $s = \xi \cdot s_u$, where s_u is the maximum draft allowed by the norms [4, etc.], Eurocode 7: TKP EN 1997-2009, Part 1, operating on the territory of the Republic of

Belarus, depending on the structural system of the facility and its purpose.

According to experimental and theoretical studies, the destruction of the pile base by the pressed load occurs along solid wrapped slip planes strictly oriented in space, oriented in space by angles β and α, the formation process of which can be schematically represented in the following sequence (see Figure 1b). As a result of an increase in the load P (the first phase of compaction in Figure 1a), compacted zones begin to form at the base of the pile. Under the lower end (fifth) of the pile, this occurs in the form of the formation of a rigid wedge I, which is its continuation, with faces inclined at an angle α_1 to the horizon $\alpha_1 = \varphi / 4 \ldots (\pi / 4 + \varphi / 2)$; along the trunk in the form of a cone of subsidence (hereinafter referred to as a cone) for each homogeneous layer, with faces inclined at an angle to the vertical $\beta \cong \pi / 4 – \varphi / 2$ (see Figure 1b).

When the load reaches the value corresponding to phase 2 (shears) according to Figure 1a, the cone along the barrel begins to move downward along the enveloping sliding surfaces, and the lateral planes of the wedge under the heel (pos. I in Figure 1a) begin to actively push the lateral volumes of the soil to the sides and upward, forming new sliding surfaces (area II) of passive soil shear resistance directed at an angle to the horizon $\alpha_1 = \pi / 4 – \varphi / 2$. The reaction of immobile soil III to the pressure of the passive shear resistance prisms II when the pile heel is pressed is expressed by the forces R_1 and R_2 applied to the shear surfaces. They are deflected from the perpendicular (normal) to them by the angle of internal soil friction φ (see Figure 1b). Adhesion value should be considered separately.

The projections of the forces on the z-axis applied to one of the halves of the deformable subgrade can be reduced to the form (solution of Puzyrevsky, K. Tertsagi, Gersevanov, etc.).

$$R(\sigma)_{max} = Ah + Bb + Dc, \qquad (1)$$

where $R(\sigma)_{max}$ is the load corresponding to the critical limit of the bearing capacity of the foundation;

h is the depth of the foundation base (surcharge q / ρ corresponding to the height of the conditional soil layer);

b, c – respectively the width (diameter) of the foundation; soil adhesion;

A, B, D – constants for a given soil, the coefficients of its bearing capacity, depending on φ and ρ. The version of this decision adopted in the norms of the Republic of Belarus is given in TCP 45-5.01-67-2008. Foundations slab design rules. Performed analysis of the state of the pile foundation at the stage of its destruction showed that for the study of their stability and calculation of the bearing capacity, it is quite acceptable to apply the provisions of the theory of the limiting equilibrium of soils. In this case, a decisive role in the process of destruction of the base is played by violations of its internal equilibrium in the form of shears along the sliding surface arising in separate volumes of soil along the length of the trunk and under the lower end of the pressed (pulled) pile.

CALCULATION OF THE BEARING CAPACITY AND STABILITY OF THE BASE OF A PRESSED (PULLED-OUT) PILE BASED ON THE RESULTS OF SOIL TESTING

In modern design practice, the calculation of the bearing capacity of the foundation (1st limit state) is carried out according to a 2-component scheme. It is assumed that the ultimate load on the base is equal to the sum of the resistances (reactions) of the soil to the indentation of the heel and the pile trunk into it, corresponding to the exhaustion of strength and loss of stability of the soil - and defined as its bearing capacity.

The characteristic, experimentally established, signs of loss of stability of the foundation when pressing and pulling out a pile are (see Figure 1b):

the block character of the uplift of the shear volumes of the soil without its destruction inside the massif;

the constancy of the curvature of the lateral surface and the volume of the subsidence / thrust blocks, in the form of a truncated cone;

a pronounced sliding surface, directed downward (when pressed) or upward (when pulled out) at a certain angle of inclination to the vertical – for the trunk (separating the bearing “bulging cone of subsidence” from the stationary surrounding soil mass) and angle α to the horizontals – for the heel;

relatively geometrically correct shape of the sliding surface of small curvature close to a straight line;

displacement along the sliding surface of an undisturbed block of soil in the form of a truncated cone, separated from the bulk of the soil with a smaller base at the top, when pressed, and at the bottom, when pulled, equal, approximately, to the diameter of the piles d, with a large diameter D_k at the bottom / top of the base of the cone (on the surface priming when pulling out);

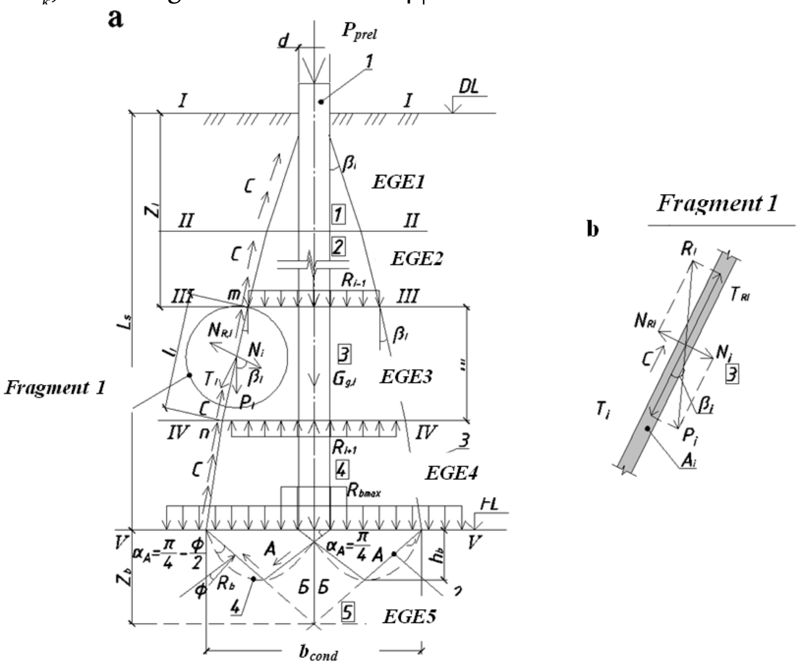
the nature of the sliding surface is determined not so much by the stressed state of the pile base as by the distance z from its heel to the levelling surface, the diameter and properties of the soil (density ρ, moisture ω, angle of internal friction φ, adhesion c).

Thus, to assess the degree of stability and study the conditions for the occurrence of shears in the base of the pile, in this and a number of other cases considered in the works of B. V. Bakholdin, V. G. Berezantsev, A. A. Grigoryan, V. N. Kravtsov, F. K. Lapshin, V. A. Florin [11–13 and others], the possibility of applying the general theory of the limiting equilibrium of soils is quite obvious. The decision on the stability of the pile foundation in this case can be attributed to its elementary particular problem, which consists in determining the equilibrium condition for a fixed sliding surface formed as a result of the shift of a non-deformable block of soil under / over the fifth pile relative to the stationary surrounding massif.

Let us consider the conditions of the limiting equilibrium (stability) of the pile foundation under the influence of the limiting indentation force P / pulling out P applied to it along the axis of its symmetry (the solution for the pulled out pile is given in [13]).

On the basis of experimental studies [10-12, etc.], we assume that when the pile is exposed to the pressing force P_{cr} (see Figure 1a), shear stresses are concentrated along its trunk and under the slab, forming a subsidence body in the form of one truncated cone (see Figure 1b) in homogeneous soil or from several cones (compartments) - in a multilayer one (Figure 2a), with an upper smaller base equal to approximately the area of the pile and a side surface inclined downward at an angle to the vertical. The thickness of the compartments in a multi-layer base is taken to be equal to the thickness of the layers cut by the piles, as a rule, no more than 2 m (see Figure 2a). The area A of the lateral shear envelope surface of each truncated cone of subsidence is essentially a fixed surface of ultimate equilibrium (slip). Therefore, according to the theory of limiting equilibrium of soils, the bearing capacity of the pile foundation can be found from the equilibrium equations for each compartment $\boxed{1}$ – $\boxed{5}$ in Figure 2a, by summing the critical loads P_i in each compartment perceived by them at the moment the soil loses stability.

Let us consider the equilibrium conditions of the i-th compartment (subsidence cone), for example, in Figure 2 in a flat setting, bringing the cone in the plane of the drawing to a trapezoid with side faces directed downward from the upper base of size d, to a larger lower size D_k, at an angle to the verticals β_i.



1 – pile; 2 - the edge of the sliding surface (internal discharge) under the lower end of the pile at the stage of ultimate equilibrium; 3 - the same, along the trunk; A – uplift zones (passive soil shear resistance); B – zone of stationary soil relative to which zones A are displaced; - direction of soil displacement in zone A.

Figure 2. Maximum equilibrium diagram of a multilayer pile base a – calculation model for determining the bearing capacity (stability); b – shear circuit at point *i* on an elementary sliding surface

The following external forces are applied to the compartment at extreme equilibrium: additional Pi from external load P, reactions from the upper Ri-1 and lower Ri + 1 sedimentary compartments, and the dead weight of the $\boxed{3}$ compartment G_{gi} with the pile section. We expand the resultant force P_{i} at the point of application to the unit area of the shear plane of the enveloping sliding surface *nm* into its components: normal Ni, and tangent T_i:

$$T_i = P_i \bullet cos\beta_i;\ N_i = P_i \bullet sin\beta_i; \qquad (1)$$

External forces on the shear plane (see Figure 2b) at ultimate equilibrium are counteracted by the following **internal** forces directed in the opposite direction from the shear forces T_i: adhesion c_i, acting along the edge of the sliding surface nm and the resistance of the soil to indentation of the pile R_i, which decomposes into components at the point of application: tangent T_{Ri} (frictional force) and normal N_{Ri} (rebound reaction of the base soil to the face nm):

$$c_i = \sum c_{II;si} \bullet l_i; \qquad (1a)$$

$$N_{Ri} = R_i \bullet \sin\beta = \gamma_{II;si} \bullet z_{\mathrm{i}} \bullet \sin\beta_{\mathrm{i}}, \qquad (1b)$$

$$T_{Ri} = f N_i = \mathrm{tg}\varphi_{\mathrm{II;i}} \bullet N_i, \qquad (1c)$$

where $\gamma_{II;\ si}$ **is** the equivalent (average) specific gravity of soils above the considered compartment, z_i is the thickness of soils from the surface to the middle of the compartment h_i).

We compose the general equations of the limiting equilibrium for the h_i compartment. We project all the forces on the inclined face of the shear surface nm and on the reaction normal to it N_{Ri}, referring them to the unit area of the shear plane A_{si}.

$$T_i = T_{Ri} + c_i \bullet A_{si} + \sum R_i = f_i \bullet N_{Ri} \bullet c_i \bullet i_i + R_{i+1} \cos\beta_{\mathrm{i}} - (R_{i-1} + G_{gi}) \cos\beta_{\mathrm{i}}; \quad (2)$$

$$N_{Ri} = N_i + \sum R_i = N_i + R_{i+1} \sin\beta_{\mathrm{i}} - (R_{i-1} + G_{gi}) \sin\beta_{\mathrm{i}}. \qquad (3)$$

Substituting expression (3) into (2), we obtain

$$T_i = f_i N_i + c_{II;si} \bullet l_i + R_{i+1}(f_i \bullet \sin\beta_{\mathrm{i}} + \cos\beta_{\mathrm{i}}) - (R_{i-1} + G_{gi})(f_i \sin\beta_{\mathrm{i}} + \cos\beta_{\mathrm{i}}). \quad (4)$$

We transform expression (4), using the formulas for the difference of two angles $\sin(\varphi_i+\beta_i)$ and $f_i=\text{tg}\varphi_{II;i}=\frac{\sin\varphi_{II;i}}{\cos\varphi_{II;i}}$, solving it with respect to R_{i-1}, leading to a common denominator, taking into account the total area of the enveloping surface slip A_{si}.

$$R_{s,i-1}=\frac{A_{si}(T_i-\text{tg}\,\varphi_{II;i}\cdot N_i-c_{II;i}\cdot l_i)\cdot\cos\varphi_{II;i}}{\cos(\varphi_{II;i}+\beta_i)}+R_{i+1}+G_{gi},\ \text{MN.} \quad (5)$$

Formula (5) makes it possible to determine the maximum bearing capacity of the soil of the pile foundation (shear resistance) at any section hialong the length of its trunk. To calculate the total value Rs;i of the pile trunk, the calculation must start from the upper compartment $\boxed{1}$ according to Figure 2, for which $R_{s;i+1}=0$.

The total bearing capacity of the pile base is determined by calculation as the sum of the soil resistance to immersion of the heel and the pile shaft into it.

$$R_{k;cal}=\gamma_{t1}\cdot\gamma_{t2}\cdot(\gamma_b\cdot R_{b;k;cal}\cdot A_b+\gamma_{si}\cdot\sum R_{si;k;cal}),\ \text{MN,} \quad (6)$$

where t_1 and t_2 are partial coefficients of working conditions, depending on the type of piles (finished, rammed), designs and manufacturing technologies given in the latest edition of the norms (SP) of the Republic of Belarus for finished and rammed piles of different types;

γ_b and γ_{si} – partial safety factors on the ground, respectively, at the heel level and average along the length of the trunk, given in the edition of the norms (SP) of the Republic of Belarus;

$R_{b;k;cal}$ is the maximum standard resistance of the base soil to the indentation of the lower end of the pile (heel) into it according to formula (1) as amended by N.M. Gersevanov and norms (TNLA) of the Republic of Belarus (ТКП 45-5.01-67-2008), MPa;

A_b = 0.785 $D^2_{усл}$ – is the lower effective area of transfer of the load to the soil at the level of the end of the pile with the nominal diameter $D_{cond.}$, equal at $L_s < 6$ м – $D_{cond.} = d + 2L_s \bullet tg\beta_i$; at $L_s \geq 6$ м – $D_{cond.} = d + 1.1$, m² (here $\beta_i = 6°$);

$\sum R_{si-1}$ – the sum of the maximum soil resistance (bearing capacity) to the pile trunk being pressed into it until the limiting equilibrium is violated according to the formula (5), MN.

The design (design) load allowed on the piles $F_{d;cal}$ should not exceed the design (design) resistance of the pile base soils $R_{d;cal}$ and is determined by the formula of the norms [4]

$$F_{d;cal} \leq R_{d;cal} = \frac{R_{k;cal}}{\gamma_k}, \text{ MN}, \qquad (7)$$

where k = 1.4 is the coefficient of accuracy of the method is determined by $R_{d;cal}$.

The obtained theoretical solution to the particular problem of assessing the bearing capacity and stability of multilayer pile foundations is currently used to develop a universal method for calculating finished and rammed piles of prismatic and cylindrical shapes of a longitudinal profile with a length (2 to 35) m with a cross section (20 to 80) see, included in the latest edition of the joint venture for the design of bases and foundations of the Republic of Belarus.

CONCLUSION

1. A particular theoretical solution has been obtained that develops the provisions of the theory of the limiting stress equilibrium of soils in relation to the assessment of the bearing capacity and stability of the pile foundations for the conditions of the plane problem, proceeding from the assumptions that the loss of their stability occurs along rectilinear shear planes with angles of inclination strictly fixed in space at the maximum ordinate development of the region of

limiting equilibrium $z_{max} = 0.25d$, which corresponds to the critical draft $s_{cr.} \leq \xi \bullet s_u$ on the graph in Figure 1a.

2. This theoretical solution can be used to develop various types of piles by introducing partial safety factors and working conditions into the calculation formulas, taking into account the specifics of its interaction with the soil and manufacturing methods. At present, the obtained theoretical solution is used in the development of a universal method for calculating finished and rammed piles of prismatic and cylindrical shapes, which was included in the latest edition of the Resolutions of the Council of the Republic of Belarus for the design of bases and foundations.

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