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ABOUT THE METHODS FOR CALCULATION OF THE STABILITY OF SLOPES OF GROUND MASSIVES

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Abstract

Methods for calculating the stability of slopes and models of soil deformation are considered, modern methods for calculating the stability of slopes (including SNC) are based on a simple model of an ultimate stressed environment. which does not consider the deformed state of the soil massif. The shape of the fracture (sliding) surface is taken as the main feature in the classifications for calculating c. Classification of methods for determining slope stability parameters.

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INTRODUCTION

Soil dams, dams, embankments and natural slopes act as soil massifs when they or the adjacent territories are used for economic activity. The use in design practice of a large number of calculation methods for assessing stability indicates the complexity of the problem and the incompleteness of its solution.

RESEARCH METHODS

The stability of slopes of soil massifs (dam slopes, subgrade and side slopes of rocks) is determined mainly by two factors: 1) the stressed state of the massif; 2) physical and mechanical properties of the soil mass. In this case, the stress state of the array determines the acting loads, and the physical and mechanical properties determine the strength of the array along the weakest surface.

In the study of the stress-strain state of the slopes of the soil massif or the edge massif of open pits, two models of soil deformation are used:

1) linearly deformable (Hooke's law) and

2) limit-stressed environment.

Using a linearly deformable model of the medium to calculate the stability of slopes, various methods have been developed in [1,2,12] and other works, however, the application of the Hooke 's law of deformation gives clearly overestimated (by 2-3 times) values of the limiting parameters of the slopes, besides the calculated the parameters determined using linear deformation are not confirmed by practical data [1-2]. This is due to the fact that to describe the stress-strain state of soils and rocks, more complex models are required - elastoplastic, viscoelastic, rheological, etc. The construction of rigorous mathematical solutions based on these models is a complex task associated with the solution of systems of differential equations, the fulfillment of the boundary conditions imposed in the formulation of the original problem. Determining the stress-strain state of soil massifs, taking into account the non-linear deformable model, is the goal in the next stages of this project.

All modern methods for calculating the stability of slopes (including SNC) are based on a simple model of an ultimate stressed environment, in which the deformed state of the soil mass is not considered. Calculations without taking into account deformations, considering the equilibrium of rigid elements, under the assumption that the ultimate stress state is reached along the collapse surface, allows obtaining relatively simple solutions for various design schemes, which have received good practical confirmation [1].

Many methods, methods and techniques for calculating the stability of slopes have led to the need to classify them according to one or another feature. As the main feature in the classifications for calculation in [4-6], the shape of the fracture (slip) surface is taken. According to this classification, four classes of methods for determining slope stability parameters were identified in [5]:

1) construction of the slope contour, at all points of which the limit equilibrium condition is satisfied (in this case, the system of differential equilibrium equations is solved together with the limit state condition, the foundations of which are laid down in [7]);

2) construction of a slope contour along which the condition of equality of the angle of inclination of the tangent to the angle of shear resistance is satisfied (the basis of this method is given in [8]);

3) construction of a sliding surface in the slope mass, along which the limit equilibrium condition is satisfied (calculation methods of this class are the most numerous and are included in the KMC, they are based on the adoption in the calculation schemes of one form or another of the sliding surface: flat, round- cylindrical , in the form of a logarithmic spiral, complex curvilinear, broken line, etc.) ;

4) construction of a sliding surface in the slope mass, along which the condition of a special limiting equilibrium is satisfied (for inhomogeneous and anisotropic massifs).

In practice, the method for assessing the stability and seismic resistance of slopes of dams, dams, quarry walls, etc., has been widely used, assuming the fulfillment of the limit equilibrium condition (ultimate stress state) along the inner boundary of a certain area of the slope zone of the soil massif. The boundary of this area is considered to be the expected fracture surface (sliding surface). The methods listed above are fundamentally based on the solution of the Coulomb limit equilibrium equation. Ultimate stress state, i.e. the limit equilibrium equation (Coulomb's law) has

$\tau = \sigma \, \mathrm{tg} \varphi + C$, (1.1)

where σ, τ is the normal and shear stress acting on the sliding surface areas, C is the cohesion of the soil mass, φ is the angle of internal friction.

The initial provisions on which the calculation methods for stability are based are as follows [1-3,9]:

- deformation of dam slopes, dams or quarry slopes occurs in the form of slumping or collapse along the sliding surface in the near- contour soil or mountain range;
- in the absence of an unfavorable arrangement of weakening surfaces in the slopes of the massif, the sliding surface is close in shape to round- cylindrical ;
- if there are unfavorably oriented weakening surfaces in the slope, the sliding surface coincides with them.

The calculation of the stability of the slopes of the dam, dams or sides of the open pit consists in determining the minimum stability coefficient for the accepted shape of the transverse profile of the soil mass and is carried out for the most unfavorable cross sections of the characteristic sections of the mass. The minimum slope stability factor is taken depending on the class of structure (dams, dams, embankments, etc.) according to Table 1.1 [1,9]:

Table 1.1 - Permissible stability factors

Combination of loads and actions	Safety factors for different classes			
Main	1.3	1.2	1.15	1.1
Special	1.1	1.2	1.06	1.05

Currently, in the design of earth dams and quarrying, slope stability is calculated using a method based on circular sliding surfaces (CSS). This method was first proposed in 1916 by the Swedish engineers Peterson and Gyultin based on studies of the collapse of clay soils. They found that the surfaces of clay soil slope collapse are curvilinear and can be taken as cylindrical in cross section [9]. The CSS method is notable for its significant simplicity and compliance with the results of field observations. According to the CSS method, the loss of stability of soil structures occurs along round- cylindrical sliding surfaces (collapse).

It is recommended in the SHNC to calculate the stability of slopes using the methods of round- cylindrical sliding surfaces (the method of NII VODGEO) and it consists in finding such radii and positions of the centers of slip curves at which the stability coefficient will be the smallest. There are various approaches and methods for determining the geometric center of the CSS [9-14].

The simplest and most widely used method is given in [9-14]: the calculation for the dam is carried out for a number of points of the centers of the slip curves, selected in the so-called region of the centers of the most dangerous curves (Figure 1.1). This area is located between two straight lines restored from the center of the slope at an angle of 85º and perpendicular to the base of the dam. Between these straight lines, two arcs of a circle are drawn from the center of the slope with radii depending on the magnitude of the slopes and the height of the dam.

In this area, a number of points of centers are taken, consistently approaching the most dangerous area. From each point, a sliding circle is drawn with such a radius that it passes through the crest of the dam and captures part of the base to a depth $H/2$ of H is the height of the dam. Within the slip curve, the slope and the base of the dam are divided into a number of compartments (columns) of the same width, depending on the radius of the slip curve. On the vertical, lowered from the center of the sliding curve, is the middle line of the initial compartment (column). The numbering of the midlines up the slope is positive, down the slope is negative. The initial middle line has the number "0" (Figure 1.2).

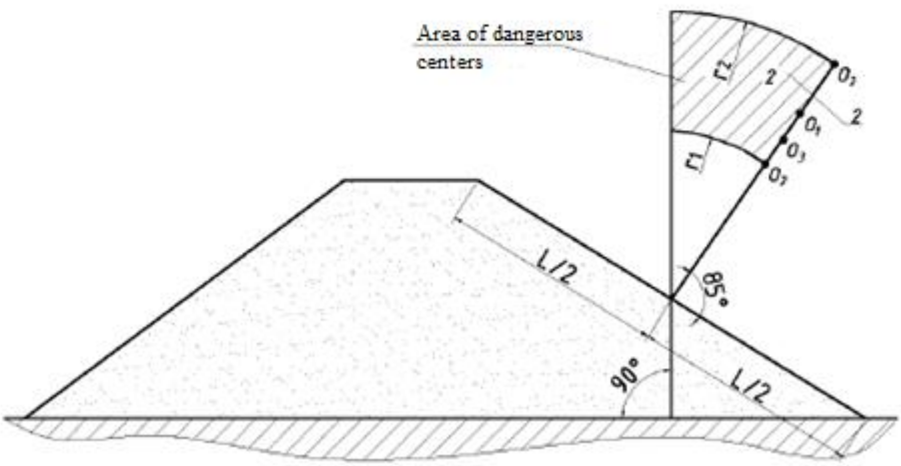

Figure 1.1 - Scheme for determining the area of hazardous centers of the circular sliding surfaces

For each compartment, the weight of the compartment and all forces acting on the compartment are determined.

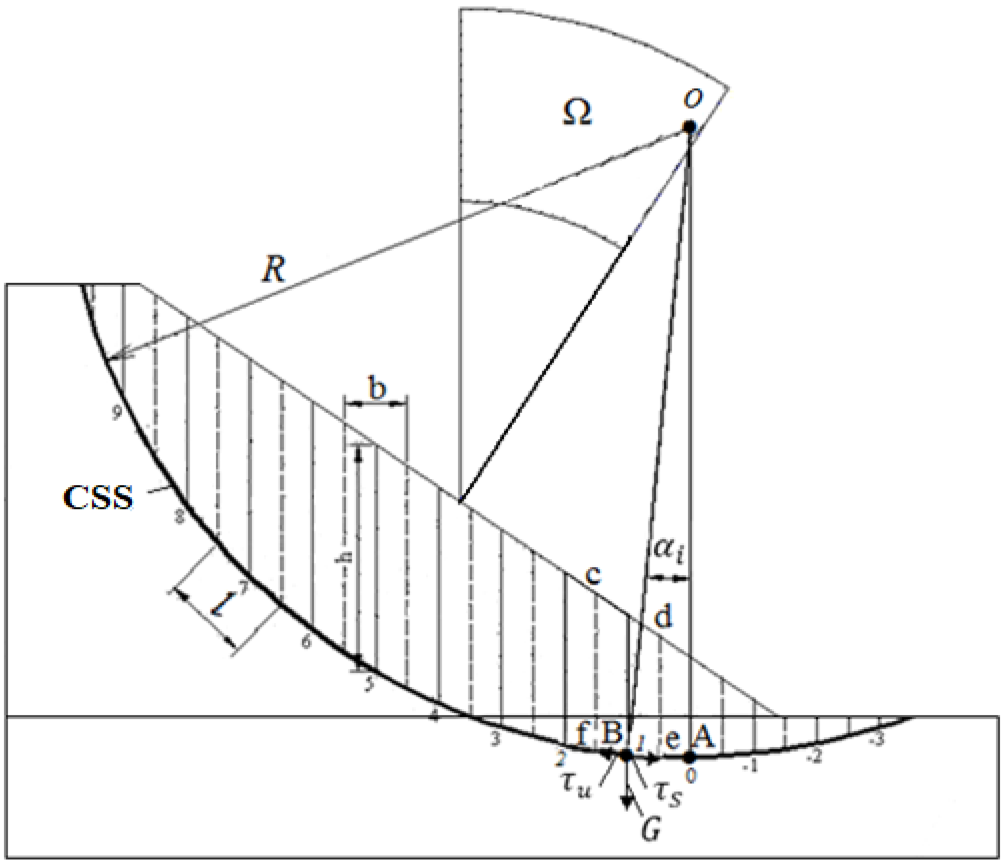

Figure 1.2 - Scheme for calculating the stability of the slope using the CSS method

The angle α between the vertical dropped from the center of the slip curve and the line drawn from this center to the center of the base of each compartment will be determined by the sine b of R the n angle $\sin\alpha = \frac{bn}{R}$:

The stability coefficient is calculated by the formula

$$K_{ust} = \frac{F_{uder}}{F_{cdvig}}, \qquad (1.2)$$

where F_{uder} is the sum of all holding forces, F_{cdvig} is the sum of all shearing forces [9].

Verification of slope stability in [10] and other works is reduced to determining the stability coefficient, which is equal (except for the forces of lateral pressure) to the ratio of the moment of holding forces (friction and adhesion) to the moment of shearing forces:

$$K_{ust} = \frac{\sum M_{uder}}{\sum M_{cdvig}}. \qquad (1.3)$$

The stability coefficients defined in terms of (1.2) and (1.3) are practically equivalent. In the limit-stress state of the soil mass, formula (1.3) proposed by C. Terzaghi [9] takes the form:

$$K_{ust} = \frac{\sum M_{pr}}{\sum M_{cdvig}} = \frac{\int_l R\tau_{pr} dl}{\int_l R\tau_{akt} dl} = \frac{\sum \tau_{pr} \Delta l}{\sum \tau_{akt} \Delta l}, \qquad (1.4)$$

where τ_{pr} is the limit value of the shear stress, it is determined using the limit stress state (1.1); τ_{akt} - shear stress, actively acting along the proposed curve (surface) of the collapse. using (1.1), the stability coefficient according to the Terzaghi method has the form [9]:

$$K_{ust} = \frac{\sum \left[(G_i - P_b b_i) \cos \alpha_i tg\varphi + Cb_i / \cos \alpha_i \right]}{\sum G_i \sin \alpha_i}, \qquad (1.5)$$

and when seismic loads are taken into account, in this case seismic forces are taken into account by setting the vertical and horizontal components, formula (1.5) will be written as

$$K_{ust} = \frac{\sum \left[(G_i - P_b b_i - S_{Vi}) \cos \alpha_i - S_{gi} \sin \alpha_i \right] tg\varphi + Cl_i}{\sum (G_i \sin \alpha_i + S_{gi} \cos \alpha_i) - S_{Vi} \sin \alpha_i}. \qquad (1.6)$$

In formulas (1.5)-(1.6) $G_i = \gamma_i h_i b_i$- the weight of the collapse prism (compartment); γ_i- specific gravity of the soil; $h_i$is the height, a $b_i$is the width of the compartment, $P_b$is the pore pressure along the bottom of the compartment, $P_b = h_0 \gamma_0$; α_i- the angle between the lines from the center of the sole of the compartment to the center of the RCC and the vertical line from the RCC (between the lines OA и OB, Figure 1.2); $\varphi_i$is the angle of internal friction of the soil; c_i– soil cohesion coefficient; $l_i = b_i / cos\alpha_i$; $S_{gi}$is the horizontal component of the seismic force; $S_{vi}$is the vertical component of the seismic force.

It is obvious that the soil massif, as well as their collapse compartments, is considered to be an absolutely rigid body according to the CSS method. Consequently, the CSS method cannot take into account the deformed state of the soil mass arising from static and dynamic loads. Despite all the shortcomings, the CSS method is currently the main method that is used in engineering calculations for the design and construction of soil structures and is recommended in the SHNC [13,14].

CONCLUSION

1 The use in engineering practice of a large number of calculation methods for assessing the stability of soil massifs indicates the complexity of the problem and the incompleteness of the process of finding its solution that would satisfy researchers and designers.

2 Using existing methods, finding the most dangerous round-cylindrical sliding surface using a selected area does not guarantee finding the minimum value of the stability coefficient, since several local minima may exist in other areas.

3 The variety of interpretations of the stability coefficient indicates that the stability coefficient gives, in fact, only a relative assessment of stability, revealing the measure of stability within the framework of the calculated assumptions of the method used, so the development of a universal method for quantitative assessment of stability remains an unresolved problem.

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