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THE ROLE OF STRUCTURAL COHESION IN CLAY SOILS IN THE EVALUATION OF SOIL BASES BEARING CAPACITY

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Abstract. The article examines the causes of bearing-capacity failures in foundations of buildings and structures situated on sloping areas in Republic of Moldova, focusing on the response of Sarmatian clay foundation soils, which exhibit time-dependent strength reduction and variations of shear resistance when subjected to prolonged loading. The experiments included tests on undisturbed monolithic samples using a direct shear apparatus at various levels of normal stress, as well as an evaluation of residual and long-term strength in accordance with the principles of the physico-technical theory of creep. The obtained data indicate that the investigated clays are characterized by a significant structural cohesion (average value of approximately 60kPa) and a relatively low internal friction angle of 7-12° (average around 9°). It was established that the application of long-term loads leads to a pronounced reduction in shear strength parameters: the long-term strength reaches approximately 40kPa, while the residual strength is on the order of 20kPa. Calculations of ultimate bearing capacity showed that, depending on the applied stress level, the permissible bearing capacity of the foundation may decrease by 2.5-3 times compared to the initially determined values. These findings highlight the need to account for time-dependent strength degradation when designing foundations on Sarmatian clays and emphasize the importance of accurately determining soil strength parameters, particularly the structural cohesion.

Keywords: *clay soils, stability, strength, structural cohesion, ultimate load.*

Rezumat. În articol sunt analizate cauzele pierderii capacității portante a terenurilor de fundare ale construcțiilor amplasate pe versanți sau zonele în pantă din Republica Moldova, cu un accent pe comportarea argilelor sarmatiene utilizate ca teren de fundare, care prezintă o reducere și o variație în timp a rezistenței la forfecare sub acțiunea sarcinilor permanente. Partea experimentală a inclus încercări pe probe monolitice neperturbate (stare naturală), utilizând aparatul de forfecare directă pentru diferite valori ale tensiunii normale, precum și evaluarea rezistenței reziduale și cea de lungă durată în conformitate cu principiile teoriei fizico-tehnice a fluajului. Datele obținute indică faptul că argilele studiate sunt caracterizate printr-o coeziune structurală ridicată (valoare medie de aproximativ 60 kPa) și un unghi de frecare internă relativ redus, cuprins între 7–12° (în medie circa 9°). S-a constatat că aplicarea sarcinilor de lungă durată conduce la o diminuare pronunțată a parametrilor de rezistență la forfecare: rezistența pe termen lung ajungând la aproximativ 40 kPa, iar rezistența reziduală

poate ajunge până la 20 kPa. Calculul capacității portante la starea limită ultimă a arătat că, în funcție de nivelul solicitării aplicate, capacitatea portantă admisibilă a fundației poate scădea de 2,5–3 ori în comparație cu valorile determinate inițial. Aceste rezultate subliniază necesitatea luării în considerare a degradării în timp a rezistenței pământului la proiectarea fundațiilor pe argile sarmatiene și evidențiază importanța determinării cât mai corecte a parametrilor de rezistență, în special a coeziunii structurale.

Cuvinte-cheie: *pământuri argiloase, stabilitate, rezistență, coeziune structurală, sarcină limită ultimă.*

1. Introduction

When designing buildings and engineering structures, a number of issues arise related to the quantitative assessment of overall stability and deformation of foundation soils.

The bearing capacity of the foundation is defined as the ultimate pressure exerted on the soil beneath the foundation at which shearing initiates and a failure/slip zone develops. Any further load increase leads to a loss of stability and, consequently, to catastrophic deformations (Figure 1).



Figure 1. Loss of stability of a residential building in Seoul, South Korea, 2014

Source: image taken from the website [1].

Terzaghi K. is rightfully regarded as the founder of theoretical research in this field [2, 3]. The theory developed by Terzaghi K. was later modified to take into account the shape and embedment depth of foundations, the magnitude of the applied load, and the eccentricity of loading [4-7].

Among the extensive body of scientific works in this area, notable contributions include those of Bartolomey [8], Berezantsev [9], Bogomolov [10], Vyalov [11], Goldshtein [12], Gorbunov-Posadov [13], Maslov [14], Ter-Martirosyan [15], Florin [16], Tsytoovich [17], as well as Akai [18], Lundgren. and Mortensen [19], Meyerhof [20], Prandtl [21] and many others.

In particular, the studies conducted by Berezantsev [9], Bugrov [22], Malyshev [23,24], and other researchers have played a crucial role in ensuring the reliability and long-term performance of structures founded under complex engineering–geological conditions. Their contributions are grounded in systematic investigations and analytical developments within the fields of solid mechanics, as well as geology and hydrogeology.

A substantial contribution to the theory and practice of evaluating foundation bearing capacity and slope stability was made by the works of Maslov N.N. In particular, reference [14] presents the author's perspective on the potential loss of stability in foundations resulting from time-dependent reduction of soil strength.

With the development of an extensive experimental database and a robust mechanico-mathematical framework, it has become possible to address complex problems related to the interaction between structures and soil masses.

The studies of Kalaev [25], Ter-Martirosyan [15], Dobrov [26], Bogomolov [10], Karaulov [27], Dyba [28], and others are dedicated to modern numerical methods for analyzing the stress–strain state (SSS) of soil masses.

Within the framework of the present study, it is particularly important that these works not only enable the solution of specific practical problems but also make it possible to formulate and analyze mathematical experiments that account for the deformation and failure characteristics of the Sarmatian clays under investigation.

Shakirzyanov proposes employing a methodology for SSS analysis based on determining the ultimate load using the principles of limit equilibrium theory [29]. The method developed for assessing the bearing capacity of the “structure–soil” system incorporates a range of influencing factors, including the time-dependent reduction of soil strength parameters.

Based on an analysis of various factors affecting the stress state and bearing capacity of a two-layer foundation, Vaingolts [30] developed an engineering method that allows the determination of the design bearing resistance of foundations for the most commonly encountered geological conditions. The author's findings and the proposed method may be applied in Republic of Moldova, as most foundation soils in the region are represented by sandy–clayey sequences.

This brief review of the literature indicates that the established principles of soil mechanics inevitably introduce certain simplifications and idealizations of soil properties, whereas the actual behavior of soils in a foundation under structural loading is considerably more complex.

As a classical example of foundation instability, the case most frequently cited is the leaning tower in Pisa, Italy [31, 32]. The main causes of the excessive settlements and differential deformations are attributed to the presence of weak clay layers beneath the foundation and a high groundwater level. Stabilization was achieved by controlled soil excavation on the northern side of the structure.

However, analysis of the literature sources has shown that, in addition to this well-known case from Italy, several other examples convincingly demonstrate the necessity of a timely and properly executed assessment of foundation bearing capacity.

This can be illustrated by the construction of a high-rise building in San Francisco, USA [33, 34]. Since its completion in 2009, the Millennium Tower has settled more than 40cm. The deformations developed non-uniformly, resulting in a lateral tilt of approximately 35cm. The tower has consequently been nicknamed the “Leaning Tower of San Francisco.” Unfortunately, even after repair and remediation works amounting to 100 million USD, the deformation process has continued.

The reasons for the observed behavior of the tower can be attributed to the following factors:

1. **Geological conditions classifying the foundation as “weak”.** The tower was built in an area underlain by former marine deposits known as Bay Mud - soft, water-saturated clays and silts capable of undergoing significant deformation under long-term loading (in this case, from the building's own weight).

2. **Incorrectly selected foundation type.** Instead of pile foundations resting on rock formations and functioning as end-bearing piles, the designers employed friction piles (“floating piles”) resting on sandy-clayey soils at depths of 20–30 m. Although this choice reduced construction costs, but made the building vulnerable to slow, weakly attenuating deformations of the underlying soft sediments.

3. **The exceptionally large loads transmitted to the “weak foundation”.** The tower consists of 58 stories, with a height of nearly 200 m and a total weight reaching approximately 700,000 t. Naturally, such loading induced consolidation of the water-saturated clays, resulting in excessive settlement.

4. **The influence of the nearby foundations of the Salesforce Transit Center.** During its construction, deep excavations and groundwater pumping were carried out. The sharp change of hydrogeological conditions accelerated the settlement of the tower and contributed to its lateral tilting.

5. **Improperly selected foundation strengthening the foundation and foundation soils.** In 2020–2022, a project was implemented to install additional piles designed to transfer loads directly to bedrock. However, during the drilling operations and installation of these additional piles the foundation soils were compacted again. The settlement continued and, according to the most recent data, has reached approximately 45 cm.

As a result, increasing the number of piles, by installing additional ones bearing on the rock, did not stop the settlement process. Currently, engineers are developing another project that should stop the settlement of the existing complex foundation and contribute to the equalization of the pressure transmitted to the foundation soils.

In the given examples, plastic clays served as the foundation soils. As shown by the analysis of the literature sources and by the results of previously performed studies by the authors of the present paper [35–38], most slopes in the central and northern parts of Republic of Moldova are composed of Sarmatian clays in a stiff to semi-stiff state. Despite this circumstance, it often creates difficulties for designers when selecting the design strength parameters for evaluating the bearing capacity of the foundation soils. For example, during construction on clayey soils, creep processes and strength reduction under long-term loading are very often not taken into account.

In most cases, a certain schematization of soil behavior under loading, in accordance with the applicable design standards, provides quite acceptable results. However, there are numerous examples where ignoring the mentioned soil-specific characteristics in calculations has led to irreversible consequences. Some of these include the deformations of several private buildings in the Codru town, Chişinău municipality; the deformation of an embankment section on the Ialoveni bypass; the loss of stability of a wind turbine foundation in the Varzăreşti village; and the overturning of a retaining wall near the “Valea Morilor” lake in Chişinău.

There are also other examples which, along with those mentioned above, indicate the need for closer attention to studying the nature of strength of clay soils and, in particular, the role of structural cohesion in assessing long-term strength and the bearing capacity of the foundation soils.

2. Theoretical Principles

The methodological framework of the theoretical investigation is formed by contemporary principles of soil mechanics for determining stresses in soil masses, as well as in conceptual approaches addressing the potential reduction of soil strength over time.

In solving various geotechnical and structural engineering tasks, it is essential to determine the magnitude of load that can be safely transmitted to the foundation soil. The applied bearing pressure depends on the loaded area. If the soil is unable to sustain the required load, structural safety must be ensured by increasing the bearing area of the foundation. Naturally, the size of the foundation should be increased within reasonable limits. The ultimate objective of the designer is to specify the minimum feasible foundation dimensions while simultaneously ensuring the strength and stability of the foundation soil and limiting potential settlements.

Upon application of an external load, a complex stress state develops almost instantaneously within the soil mass. Under the action of normal stress P_n a compaction process is initiated, which results in the settlement of the structure (Figures 2 and 3). This process has an attenuation nature.

As the applied load increases to value P_2 further soil compaction and settlement occur, but at a lower intensity (the soil progressively compacts).

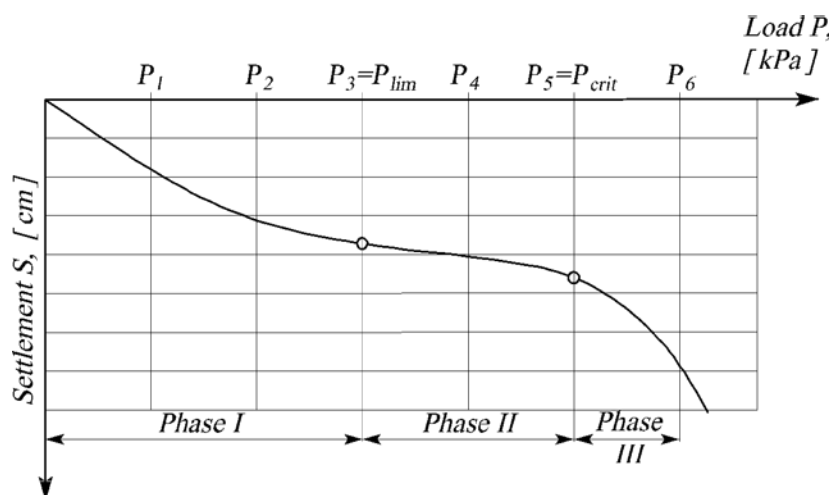


Figure 2. Three phases of loading and deformation of a structure's foundation [26, p. 120].

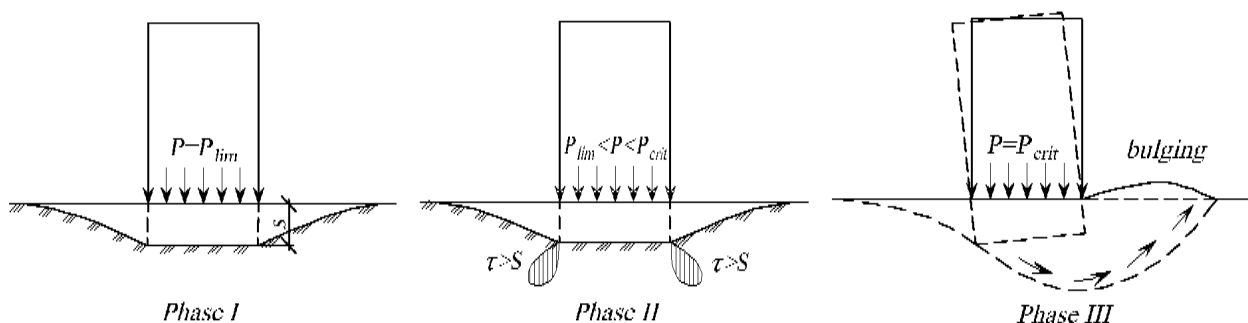


Figure 3. Three phases of soil behavior in a foundation: compaction, local shear, and deepbulging [26, p. 120].

Within the pressure interval $0 < P < P_{lim}$, the contribution of shear stresses is almost reduced to zero. These conditions define the first phase of soil behavior at the base of the structure. In this phase – commonly referred to as the soil compaction (or consolidation)

phase under structural loading – the soil strength and the stability of the foundation system are fully ensured.

With a further increase in load ($P > P_3$) progressive settlement of the structure begins, associated not only with soil compaction but also with local strength failures. Zones appear where $\tau > S$ (shear stress τ exceeds the soil strength S). The load corresponding to the initial stage of development of these local shear zones is called the limit load ($P_3 = P_{lim}$). The corresponding soil behavior conditions define the Phase II, known as the strength failure phase or local shearing phase. Such soil conditions in the building foundation are not acceptable in all cases. The resulting settlement is limited to certain limits that ensure the normal operational performance of the structure ($S < S_u$).

With a further increase in load, the progressive development of local failure zones (shear zones) occurs. These zones extend to increasing depths and, finally, at $P_5 = P_{crit}$, a usually sudden and catastrophic failure of overall foundation stability takes place, associated with soil uplift. This load is called critical load. The process corresponds to the Phase III of soil behavior in the foundation, called phase of general failure of the foundation or the phase of global shear and soil uplift. It is evident that Phase III must be excluded under all circumstances.

Returning to Phase II, under certain conditions, local shear zones develop in the soil. Their growth must be limited, which is achieved by reducing the pressure transmitted to the foundation soil. In foundation design, it is assumed that the depth of local shear zones should not exceed 1/4 of the foundation width ($h \leq b/4$). The pressure at which the local failure zones extend to a depth $h \leq b/4$ is called the design bearing capacity of the foundation soil R . The fundamental inequality that must be satisfied in calculations is therefore:

$$P \leq R, \quad (1)$$

where: P – average pressure under the foundation base, kPa;

R – design bearing capacity of the foundation soil, kPa.

The design bearing capacity R is determined based on the equations proposed by Berezantsev, according to cl. 2.41 [39]:

$$R = \frac{\gamma_{c1} \cdot \gamma_{c2}}{k} (M_\gamma \cdot k_z \cdot b \cdot \gamma_{II} + M_q \cdot d \cdot \gamma'_{II} + M_c \cdot C_{II}), \quad (2)$$

where: γ_{c1}, γ_{c2} – coefficients of working conditions;

M_γ, M_q, M_c – coefficients depending on the value of the angle of internal friction φ of the soil under foundation base;

k_z – coefficient taken as $k_z = 1$ for $b < 10$ m, and $k_z = 8/b + 0.2$ for $b \geq 10$ m;

b – width of the foundation base, m;

γ_{II} – design value of the unit weight of the soil lying below the foundation base (considering the buoyant effect of groundwater, if present), kN/m³;

γ'_{II} – average unit weight of the soil layers above the foundation base, kN/m³;

d – foundation depth embedment from the ground surface, m;

C_{II} – design value of the cohesion of the soil directly below the foundation base, kPa;

k – coefficient equal to 1.0 if the soil parameters φ_{II} and C_{II} are determined by direct testing, and $k = 1.1$ if they are adopted approximately from reference tables.

As seen from Equation (2), the design bearing capacity depends on a number of factors: the structural system of the building, the foundation embedment depth, the foundation base width, the unit weight of the soil, and its strength parameters.

A preliminary analysis showed that, in most cases, the role of cohesion is decisive.

It should be emphasized that incorrect estimation of this strength parameter has often led to loss of bearing capacity and excessive deformations of buildings and structures.

The main design error consisted in underestimating the potential decrease of cohesion (and overall strength) over time due to various factors such as:

- changes in the moisture regime;
- creep deformations;
- the presence of weakened zones within the soil thickness, etc.

In light of the above, this study investigates the role of soil cohesion in determining the safe, permissible, and critical load transmitted to the foundation soil.

For this purpose, analytical relationships given in sources [15,25,40] were applied.

The safe load P_{safe} , determined with a significant margin of safety and therefore unquestionably acceptable for the structure, was established for the case of a uniformly distributed load according to the following formula:

$$P_{safe} \approx \pi \cdot C_w \approx 3C_w. \quad (3)$$

The permissible load P_{perm} was limited by the inequality:

$$P_{perm} \leq R, \quad (4)$$

where: R – is the design bearing capacity of the foundation soil, kPa.

To determine the critical load P_{crit} , approximate relationships were used as proposed by Prandtl, Terzaghi, Shieldt and Pauker, and presented by Maslov [40], which can be expressed as follows:

$$P_{crit} = (\pi + 2.0) \cdot C = 5.14 \cdot C, \quad (5)$$

where $C = \Sigma \omega$ – hydro-colloidal cohesion, kPa.

$$P_{crit} = 5.7 \cdot C, \quad (6)$$

$$P_{crit} = (5.14 + b/l) \cdot C, \quad (7)$$

where b – half the width of the loaded area, m; l – length, m.

$$P_{crit} = \gamma \cdot (d + h_c) \cdot tg^4 \left(45 + \frac{\varphi}{2} \right), \quad (8)$$

where γ – weight density of soil, kN/m³; d – foundation embedment depth, m; φ – angle of shearing resistance, degrees; C – cohesion intercept, kPa.

$$h_c = \frac{C}{\gamma \cdot tg \varphi}. \quad (9)$$

The assessment of soil strength was carried out based on the premise that, depending on the operating conditions of the foundation, their strength parameters may vary. At the same time, shear stresses in all cases adversely affect the working conditions of the soils.

For the calculations, the constitutive framework of the Physico-Technical Theory of Creep (PTTC) [11] was used.

The shear strength was determined using the following expression:

$$S_{pw} = P_n \cdot tg \varphi_w + C_w, \quad (10)$$

where: P_n – normal stress, kPa; φ_w – angle of shearing resistance corresponding to the “density–moisture” state; C_w – total cohesion intercept, kPa.

$$C_w = C_c + \Sigma_w, \quad (11)$$

where: C_c – rigid structural cohesion, kPa; Σ_w – hydro-colloidal soil cohesion, kPa.

The strength of clay soils may vary under the influence of various factors. To identify the role of structural cohesion and moisture content, a series of tests were performed: direct shear tests on samples in their natural state, with a prepared and water-moistened failure surface, as well as on samples pre-moistened in consolidation rings.

The tests conducted simulated the behavior of the soil under conditions of existing cracks and slip surfaces within the soil mass, as well as potential reductions of cohesion due to the degradation of structural cohesion C_c and hydro-colloidal cohesion Σ_w caused by changes in moisture content within the shear zone.

The long-term strength of quasi-plastic clays was determined from the following expression:

$$S_\infty = \sigma \cdot \operatorname{tg} \varphi_w + \Sigma_w. \quad (12)$$

The residual strength values were determined from shear tests performed on samples with a prepared and moistened shear surface:

$$S_{res} = \sigma \cdot \operatorname{tg} \varphi_{w,res} + \Sigma_{w,res}. \quad (13)$$

3. Results and Discussion

The soil property investigations were carried out in the “Ingeotech-Grup” laboratory. Within the objectives of the study, the primary task consisted in determining the strength parameters of clay soils and demonstrating the possibility of their reduction due to the degradation of structural cohesion. The direct shear tests were performed using standard direct shear apparatus under various normal consolidation pressures. Undisturbed block samples (monoliths) were collected from a site located in Codru town.

Analysis of the obtained results showed the following:

1. The strength of the investigated samples in their natural state is characterized by high values:

- total cohesion $C_w=70-90$ kPa, with an average value - $C_{w,avg}=83$ kPa;
- angle of shearing resistance $\varphi=7-12^\circ$, with an average value - $\varphi_{avg}=9^\circ$.

2. The average value of structural cohesion is $C_c=60$ kPa; the hydro-colloidal cohesion component $\Sigma_w=23$ kPa.

3. Under conditions where structural cohesion may be reduced due to creep-induced deformations on the slope, the long-term shear strength can be described by the following relationship:

$$S_\infty = \sigma \cdot \operatorname{tg} 9^\circ + 23, \text{ kPa} \quad (14)$$

4. Further reduction is possible due to a significant increase in moisture content within the shear zone, along the slip surface. The residual strength equation is expressed as:

$$S_\infty = \sigma \cdot \operatorname{tg} 5^\circ + 10, \text{ kPa} \quad (15)$$

At the design pressure under the foundation base specified by the designers, $\sigma = 100$ kPa (while in practice the contact pressure commonly ranges between 150–250 kPa), using

the equations presented above, was obtained that the average strength of $S_{avg}=100$ kPa. At values of shear stress exceeding the creep threshold (in this case $\tau_{lim}=76$ kPa), and under conditions of complete destruction of structural cohesion, the long-term shear strength becomes $S_{\infty}=39$ kPa, and the minimum residual strength is $S_{res}=19$ kPa.

The obtained strength values for the investigated clays characterize the foundation soil as unreliable.

The conducted investigations convincingly confirm that the total cohesion in clay soils, in this case Sarmatian clays, is represented by a rigid, irreversible component of structural cohesion C_c and hydro-colloidal cohesion Σ_w . Structural cohesion may be partially or completely destroyed under the action of shear stresses, while hydro-colloidal cohesion may decrease to residual values as a result of increased moisture.

The possible strength reduction inevitably affects the permissible bearing pressures considered when determining the dimensions of foundation footings. Tables 1-4 present the final results of determining the safe, permissible and critical bearing pressures corresponding to different cohesion values.

The calculations were performed using the following input parameters:

- foundation footing width: $B=2b=2.0$ m;
- foundation embedment depth: $d=2.5$ m;
- average contact pressure under the footing: $P=250$ kPa;
- angle of shearing resistance: $\varphi=9^\circ$;
- cohesion: $C=10-100$ kPa.

Table 1

Nr.		Method (formula)	Values of the critical load for cohesion C, kPa					
			100	80	60	40	20	10
1*	Prandtl-Tsytovic method (plane strain problem) for a purely cohesive soil $P_{crit} = (\pi + 2.0) \cdot C + \gamma_w \cdot d = 5.14C + \gamma_w \cdot d$	563	460	357	255	152	100	
2*	Terzaghi method (plane strain problem) taking into account the roughness of the foundation base $P_{crit} = 5.7C + \gamma_w \cdot d$	619	505	391	277	163	106	
3*	Ishlinsky method (for foundations with a square footing) $P_{crit} = 5.71C + \gamma_w \cdot d$	620	506	392	277	163	106	
4*	Hencky method (under a rigid circular stamp) – “generalized form” $P_{crit} = 5.64C + \gamma_w \cdot d$	613	500	387	275	162	105	
5*	Schildt method (for rectangular foundations) and $l \rightarrow \infty$ (plane strain problem) $P_{crit} = \left(5.14C + \frac{b}{l}\right) \cdot C + \gamma_w \cdot d$	563	460	357	255	152	100	
6*	Prandtl method (plane strain problem) for soils with $\phi \neq 0$ $P_{crit} = \frac{C}{tg\varphi} \left(tg^2 \left(45^\circ + \frac{\varphi}{2} \right) \cdot e^{\pi \cdot tg\varphi} - 1 \right) + \gamma_w \cdot d$	841	683	524	366	207	128	

Continuation Table 1

7	Pauker–Maslov method		1279	1041	804	567	329	211
		$P_{crit} = \gamma_w \left(d + \frac{C}{\gamma_w \cdot tg\varphi} \right) \cdot tg^4 \left(45^\circ + \frac{\varphi}{2} \right)$						
8**	Terzaghi K. method		1037	855	674	492	310	219
		$P_{crit} = \gamma_w \cdot b \cdot N_\gamma + \gamma_w \cdot d \cdot N_p + C \cdot N_c$						

Note: The formulas presented in the Table can be found in [40, pp. 340–342, 350, 355]; * – changes made: Since the formulas presented for determining the critical load were developed for the case of direct surface loading ($d=0$), the authors considered it necessary, in accordance with the recommendations of TsytovichN.A., to introduce corrections to these expressions by including the pressure due to the self-weight of the soil located above the foundation base; ** – the values of the coefficients N_γ , N_p , and N_c were selected from [41] for $\varphi=9^\circ$.

Table 2

Results of Determining the Design Bearing Resistance of the Foundation Soil R

Nr.	Method (formula)	Values of the design bearing resistance of the foundation soil for cohesion C , kPa					
		100	80	60	40	20	10
1	According to [30]						
	$R = \frac{\gamma_{c1} \cdot \gamma_{c2}}{k} (M_\gamma \cdot k_z \cdot 2b \cdot \gamma_{II} + M_q \cdot d \cdot \gamma'_{II} + M_c \cdot C_{II})$	615	513	412	311	210	159

Note: The formula used in the calculations is given in cl. 2.41 [39]; for $l_c < 0.25$, the values of the coefficients are: $\gamma_{c1}=1.25$ and $\gamma_{c2}=1.0$, for $\varphi=9^\circ$ the values of the coefficients are: $M_\gamma=0.16$, $M_q=1.64$, $M_c=4.05$.

Table 3

Results of Determining the Safe Load P_{safe}

Nr.	Method (formula)	Values of the safe load for cohesion C , kPa					
		100	80	60	40	20	10
1*	Case of uniformly distributed load						
	$P_{safe} = \pi C_w + \gamma_w d = 3.14 C_w + \gamma_w d$	349	289	229	169	109	79
2*	Case of triangular distributed load						
	$P_{safe} = 4 C_w + \gamma_w d$	449	369	289	209	129	89
3*	Puzыrevsky formula						
	$P_{safe} = \frac{\pi \gamma_w \left(d + \frac{C}{\gamma_w \cdot tg\varphi} \right)}{ctg\varphi + \varphi - \frac{\pi}{2}} + \gamma_w d$	485	404	323	242	161	121

Note: The formulas presented in the table are given in [40, pp. 336, 341]. * – changes made: pressure from the weight of soil above the foundation base has been included.

Table 4

Results of Determining the Permissible Load P_{perm}

Nr.	Method (formula)	Values of the permissible load for cohesion C , kPa					
		100	80	60	40	20	10
1	Maslov method						
	$P_{perm} = \frac{\pi \cdot \gamma_w \cdot \left(2b \cdot tg\varphi + d + \frac{C}{\gamma_w \cdot tg\varphi} \right)}{ctg\varphi + \varphi - \frac{\pi}{2}} + \gamma_w d$	489	408	327	246	165	125

Continuation Table 4

2	Pauker–Maslov method	$P_{perm} = \frac{1}{k_{safe}} \cdot \gamma_w \left(d + \frac{C}{\gamma_w \cdot tg\varphi} \right) \cdot tg^4 \left(45^\circ + \frac{\varphi}{2} \right)$	639	521	402	283	165	105
		for $k_{safe} = 2.0 \in (1.5 \dots 2.0)$						
3	Terzaghi method	$P_{perm} = \frac{1}{k_{safe}} (\gamma_w \cdot b \cdot N_\gamma + \gamma_w \cdot d \cdot N_p + C \cdot N_c)$	519	428	337	246	155	110
		for $k_{safe} = 2.0 \in (2.0 \dots 3.0)$						

Note: The 1st and the 2nd formulas presented in the table are given in [40, p. 336, 350], the 3rd is determined by dividing the 8th formula from Table 1 by the safety coefficient k_{safe} , according to provides from [40, p. 356];

Comparison of the obtained values (see Tables 3 and 4) shows that determining the design soil resistance R according to clause 2.41 [39] allows an increase of the safe load calculated using Puzyrevsky's formula by a factor of 1.25.

It should be recalled that the behavior of the foundation is considered in Phase II.

It is noteworthy that the value of the design resistance corresponding to the long-term strength of the soil is approximately 225 kPa, which is 2.3 times lower than the value obtained at the average cohesion in the natural state ($C_{w,avg}=83$ kPa, $R=528$ kPa).

Thus, the commonly adopted value for the average pressure under the foundation base, equal to 200kPa, can be considered fully justified.

For preliminary calculations of foundation base dimensions, ensuring the bearing capacity of the soil, Puzyrevsky's formula may be used with cohesion values corresponding to the long-term strength of the soil (in presented case, $\Sigma_w=23$ kPa)

Analysis of the results showed that the pressure transmitted to the foundation, taking into account the reduction of soil strength over time, should be reduced by a factor of 2.5–3.0 relative to the value obtained at the average strength at the design stage.

This circumstance indicates the necessity, when calculating foundation dimensions and evaluating the bearing capacity of the soil, to account for the presence of weakened zones with disrupted structural cohesion within the Sarmatian clay layer.

4. Conclusions

1. In the design of buildings and structures located on sloped terrains in Republic of Moldova, the calculation of the ultimate load transmitted from foundations to the soil is a fundamental task. The reliability of this value determines both the safe operation of the structures and the economic efficiency of the adopted design solutions.

2. In engineering practice, methods based on the theory of ultimate soil equilibrium are still widely used to calculate bearing capacity. Calculations performed using this theory rely on strength characteristics such as structural cohesion, hydro-colloidal cohesion component, and internal friction forces.

3. When determining the ultimate load, it is necessary to account for the potential reduction in the strength of clay soils (Sarmatian clays) due to the disruption of structural cohesion under creep deformation and the decrease in hydro-colloidal cohesion caused by additional soil moisture.

4. To identify the causes of structural cohesion reduction (manifestation of seismo-gravitational processes, development of creep deformations on slopes, excessive vertical loads, etc.), specialized studies should be conducted at the stage of technical justification.

Due to the anticipated increase in loads on the soil caused by the construction of high-rise buildings (more than 20 floors), it is advisable in the future to consider determining soil strength as a function of the actual external load acting on the foundation base.

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