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CLAY SLOPE STABILITY COMPUTATIONS

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Abstract. This paper presents new ideas about the possibilities to increase the reliability of clay slope stability computation method. Many known methods are proved to give controversial results. The reason for this discrepancy is explained to be the result of assumptions and simplifications introduced into the methods, insufficiently precise estimation of porous water pressure, and the dependency of clay cohesion on a sudden change of compression as well. The method of a modified Boussinesq equation and slices is suggested to compute slope stability in terms of clay cohesion and internal friction determined under unconsolidated-undrained conditions.

Keywords: clay slope, stability, porous pressure, slice method, cohesion, consolidation.

1. Introduction

Picturesque views and landscape visible from the top of a slope makes the site an attractive location for dwellings and hotels. Slope stability is usually investigated before designing and constructing such buildings nearby. Nevertheless, some of the stable (according to computations) slopes suddenly slip causing a great loss. Such cases were investigated by Skempton, Henkel, and Maslov (Маслов 1977; Tavenas and Lerouil 1987).

Long lasting observations of the clay slope, which was seriously damaged during a slip and restored afterwards, were made by Skempton (Skempton 1995). The slope was stable for a long time, yet after 17 years its new slip happened. Specialists concluded that the clay slopes were dangerous and unpredictable due to the complexity of clay properties and its ability to change mechanical properties within a broad range of limits (Amšiejus 2000). In the mentioned case, slope stability decreased by 61% prior to the second slip. Each slope slip causes a property loss and sometimes human victims. There are cases when a slope slip resulted in a great number of victims: 2.600 in Italy, in 1963; 23.000 in Columbia, in 1985 (ICG 2008).

Clayey grounds are prevalent in Lithuania. They form about 70% of the earth surface here. The territory of Lithuania is rather wavy with a dense network of rivers, hills and slopes. Probably, it is the reason why frequent slope slips happen.

Kaunas town is located at the confluence of two largest Lithuanian rivers the Nemunas and the Neris with their deep valleys and a great number of tributaries. The slopes prone to slip are distributed along the territory of Kaunas and its suburbs uniformly (Fig. 1). The number of slope slips increases in the region annually, which may be seen from slip statistics data presented in Table 1.

Table 1. Dynamics of slope slips in Kaunas

Year	Slip cases
1996	42
2000	53
2004	64
2006	68

An increasing intensity of slope slip in Kaunas and elsewhere in Lithuania is a separate topic, which was not included in our investigations. Although it may be explained here that the phenomenon of a constant increment in clay slope slip may be caused by the global climatic changes and by intense human activity, more often intensive storms create more severe hydro geological conditions, which are more favourable for slope slip. Compact building constructions in towns force to approach slopes closer, which increases the danger of slope slip.

The capital of Lithuania Vilnius located at the confluence of the Neris and the Vilnele is under similar conditions. A large scale slip (about 50.000 m^3) occurred in Dvarčionys, the suburb of Vilnius in 2000, when two storehouses of the ceramics plant were swept from the earth surface (Fig. 2). The loss exceeded 5 million litas (Stelmokaitis 2003). The recent slip of this year has damaged the Gediminas mound in the centre of Vilnius (Fig. 3).

In Kaunas, a great variety of natural and artificial slope deformations and slips is caused by both natural causes and human activities. Slips of excavation or embankment slopes are rather unexpected. They were designed and their stability was verified; nevertheless, they lost stability. Such cases are not unique.

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Fig. 1. Scheme of Kaunas with location of active slopes

Fig. 2. Dvarčionys slope slip

Fig. 3. Slip of the Gediminas mound slope (Photo by Vaitiekūnas)

For example, the slopes of Via-Baltic highway between Kaunas and Kedainiai are damaged by slips in almost each tenth of kilometres of its length, especially in the territory of Kaunas (Ždankus and Stelmokaitis 2007). The reason seems to unrelated to design mistakes or construction faults. The gaps in the methods used to compute slope design and stability may be the only cause for this phenomenon.

The reason for an unexpected failure in clay slope stability has been the aim of our investigations, the results of which are described in this article.

2. Comparison and analysis of known methods

There are over 30 methods applied to design a slope and to compute their stability (Abramson 1995; Zhu *et al.* 2005). The majority of them include slope stability coefficient computation procedure. Generally, the slope stability coefficient expresses the ratio of reaction and action torques and may be written as

$$k_s = \frac{\sum T_r}{\sum T_a}. \tag{1}$$

The sum of torques of forces resisting to the slope slip is expressed as

$$\sum T_r = T_f + T_c, \tag{2}$$

whereas the sum of torques of forces initiating the slip consists of 3 terms

$$\sum T_a = T_{g\tau} + T_s + T_p. \tag{3}$$

Here T_f, T_c, T_g, T_s and T_p are torques of friction, cohesion, gravity, seepage and pore pressure and forces F_f, F_c, $F_{g\tau}$, F_s and F_p, respectively (Fig. 4). Thus the formula (1) may be written in an expanded format as follows

$$k_s = \frac{T_f + T_c}{T_{g\tau} + T_s + T_p}. \tag{4}$$

This equation was used for multi-factor analysis and their influence on slope stability to determine factor priority rank.

The known methods differ in the shape of rated slip surfaces, the ways to determine forces, the technology of computation; however, the principle of all the methods is the same. It seems that the results of computations using different methods should be very close. On the contrary, they differ greatly. Computed using 8 methods (Chugaev's, Terzagi's, Maslov's, Bishop's, Felny's, M-P, Spenser's and Jambu), the stability coefficients of 6 clay slopes of 7–31 m height, 19°–44° inclination, 1.7–1.92 kN/m^3 specific weight, 16–125 kPa cohesion, 11°–23° internal friction angle varied within the limits 1.00–4.35. For dry slopes the maximal coefficient exceeded the minimal one up to 3 times, for saturated clay – up to 2 times (Table 2).

The Chugaev's method definitely gives maximal stability coefficients, whereas the Maslov's method gives the minimal ones. The difference between maximal and minimal coefficients computed using 6 methods reaches 24% from average magnitude of the coefficient. Even such a discrepancy is unacceptable when considering the methods thought to be reliable enough and when applying them to practical computations. To determine the reasons for these differences, all these methods were analyzed focusing on simplifications and their possible impact on the results of computation.

Chugaev's method is based on the following simplification: force pressing the moving mass of slope to slip surface F_{gn} is substituted by gravity force of the mass F_g (Fig. 4), which definitely increases the stability coefficient artificially. Considered as the most reliable, a widely used Terzagi's method does not use such a rough assumption.

Each method has its own discrepancy either in the shape of slip surface profile or in neglecting the side shear stress or other assumptions, hence introducing definite errors which finally cause the declination of computation results from the actual situation and reduce their reliability.

In all the methods enumerated above, the ground water pressure is estimated in terms of one force for the whole sliding mass, i.e. the force to be applied to the gravity centre of the mass. Pore pressure force is usually not taken into account when considering the consolidation process to be completed.

This and other simplifications may lead to considerable errors subject to the degree of importance of a neglected factor to the slope slip phenomenon.

3. Priority of factors influencing slope stability

Slope stability depends on many factors like height H and inclination angle α, cohesion c, internal friction angle φ of ground, and the location of groundwater free surface. To determine the influence of enumerated factors, the computations of a stability coefficient were performed for a number of clay slopes using software GEOSLOPE. The following parameters were taken as nominal: the slope of 20 m height and 40° inclination angle, 20 kPa cohesion, 20° internal friction angle and 20° angle inclination plane depression surface (Fig. 4).

The slope with such parameters is typical of Lithuanian conditions. Dependency on parameters H, α, c, φ, and β was investigated while changing one parameter, say H, keeping the remaining α, c, φ, and β nominal magnitudes constant, and computing the stability coefficient k for each magnitude of a variable parameter. The computations were performed with 5 magnitudes of the variable: minimal, nominal and maximal and two intermediate magnitudes. While drawing $k - H / H_0$ graphs (Fig. 5), qualitative and quantitative characters of the dependency were investigated later.

Numerical results of the above described investigation are given in Table 3. The ratio of maximal and minimal slope stability coefficients does not allow drawing sound conclusions so far, when the limits of parameter ranges were selected freely, while the priority of factors is evident. The ground internal friction angle and slope inclination angle factors are prevailing in the analyzed relationships.

Table 2. Slope parameters and stability coefficients computed by different methods

Parameters		Dry slopes			Saturated slopes		
		1	2	3	4	5	6
Slope and ground parameters	Slope height H, m	31	15.5	7.5	18.2	9	7
	Slope inclination angle α, °	43	44	29	32	25	19
	Specific gravity γ, kN/m^3	1.88	1.8	1.7	2	1.92	1.92
	Ground cohesion c, kPa	50	125	60	78	100	16
	Ground internal friction angle φ, °	22	11	23	22	13	20
Authors of methods and slope stability coefficient, *k*	Chugaev's	3.05	3.18	4.35	2.02	1.42	1.86
	Terzagi	2.89	3.12	4.22	1.98	1.39	1.74
	Maslov's	1	1.02	1.76	1.04	0.94	1.05
	Bishop's	2.88	3.1	4.2	1.91	1.45	1.85
	Felnius	2.69	2.86	3.4	1.41	1.21	1.55
	M-P	2.85	2.9	4.1	1.85	1.41	1.84
	Spencer's	2.77	2.9	4.0	1.89	1.44	1.82
	Jambu	2.74	2.79	3.8	1.79	1.38	1.77

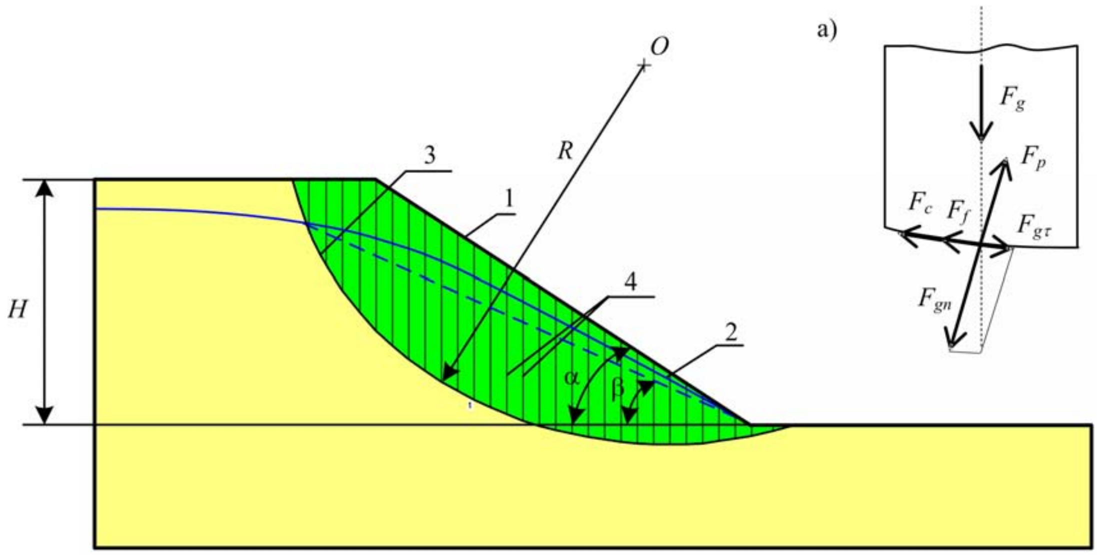

Fig. 4. Scheme of slope dimensions and force vectors for computation of slope stability by slice method: *a* is slice bottom; 1 is slope profile; 2 is ground water depression surface profile; 3 is supposed slip surface profile; 4 is the limits of a single slice

It should be explained, that the cohesion resists a slip only when $\varphi = 0$, therefore $k_{\max} / k_{\min}$ obtains an extremely large magnitude in this case and distorts the phenomenon under investigation. It was determined that $k - H$, $k - \alpha$, $k - c$, $k - \varphi$, and $k - \beta$, graphs were homogeneous, all curves were smooth and flat, which indicates the absence of critical conditions, at which slip may happen unpredictably.

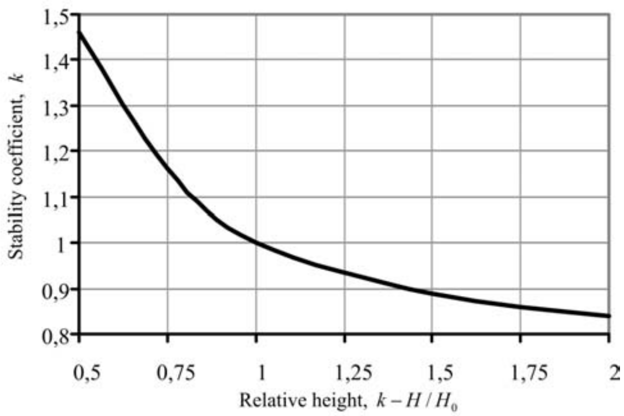

Fig. 5. Example of $k - H / H_0$ relationship graph

To imagine the influence of percentage magnitudes of ground water pressure forces in (1), (2) and (3) relationship terms, they were computed for some typical Lithuanian conditions using GEOSLOPE: $H = 20$ m; $\alpha = 35°$; $c = 20$ kPa; $\varphi = 31°$; $\beta = 30°$; density of ground $\rho = 1850$ kg/m^3 and zero pressure of water in pores T_f =1622 kNm; T_c =1079 kNm; T_τ =1545 kNm; T_s =781 kNm; T_p =0. The influence of these terms on the magnitude of slope stability coefficient

$$k = \frac{1622 + 1079}{1545 + 781 + 0} = 1.161, \tag{5}$$

where friction and cohesion forces contain 60% and 40% of a numerator respectively, while tangential and seepage forces have 66% and 34% of a denominator respectively in the expression of the slope stability coefficient. Pore pressure forces may be of the same order or even 2–3 times higher than seepage pressure forces. Neglecting them may change the situation radically and distort the computation results significantly.

Table 3. Results of clay strength testing in unconsolidated-undrained conditions

Parameter	Direct cut test result		Odometeric test result	
	Consolidated	Unconsolidated	Consolidated	Unconsolidated
Internal friction angle φ ,°	13.4	13.6	2.0	11.2
Cohesion c , kPa	36.2	20.1	34.8	17.3

4. Pore pressure forces

These forces appear due to the increment in compression stress. The compression may increase because of the external load of a slope top. The increase in load may by caused by the storage of building materials and mechanisms on the top of the slope, as well as snow, storm water, etc (Rahardjo *et al.* 2001).

The increment in ground compression stress deforms its skeleton and causes the recession of water from pores. This phenomenon called consolidation is rather rapid in sandy grounds yet slow in clayey ones (Alikonis 2001). If the permeability coefficient of clay is smaller

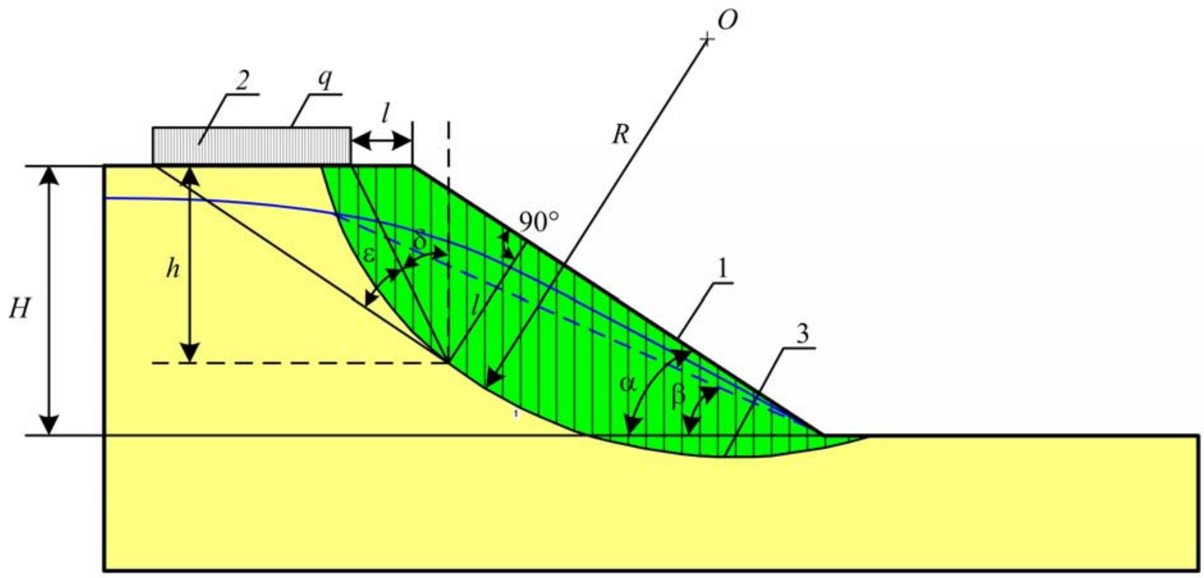

Fig. 6. Rated scheme for the estimation of slope top loading and porous pressure influence:1 – slope profile; 2 – load; 3 – slip surface profile

that n×10^{-10} m/s, where n=1, 2…9, its consolidation may last for months. In the long period of consolidation, the compression stress may reach the critical level and slope slip may happen.

An additional load of 50–100 mm of storm water layer may provoke slope slip, if it was near a critical state. Therefore, the top loading of a slope should be taken into account, when the clay slope stability is estimated. Pore pressure p_p depends on load q and depth h. At the level of supposed slip surface, pore pressure may be computed using modified Businesq formula

$$p_p = \frac{q}{\pi}\left[\alpha + \sin\alpha\cos\left(\alpha + 2\beta\right)\right]\left(\frac{h}{l_s}\right), \qquad (6)$$

where q is the intensity of load; α and β are the angles between straight lines drawn through the point under consideration and through the points of distributed load limit in the vertical cross section of the slope; h – distance to the ground surface; l_s – distance to the slope (Fig. 6).

5. Some characteristics of clay mechanical properties

It is known (Craig 1997; El-Ramly *et al.* 2002) that cohesion and internal friction of clay increase during consolidation.

To verify this phenomenon, testing of clay samples were performed and this information was approved. The results of the test are in Table 3. The difference in results between the test under consolidated and that under unconsolidated conditions is evident.

This dependency should be taken into account in clay slope stability computations. For this reason, ground samples should be investigated under laboratory conditions. Their testing should be under unconsolidated-undrained conditions only.

More reliable ground characteristics may be received from the results of field investigation. It is rather problematic to arrange field investigations at the moment of a sudden increment in slope top load and a sudden increment in pore pressure; therefore, it was decided to apply the direct cut method for field investigations.

Clay slope excavation of Via Baltic highway was selected as the site for field tests. A dynamic cut of clay by GEONOR type vane brought cohesion magnitude 12 kPa to 43 kPa. At approximately the same depth (2 m for one cross-section and 2.5 m for another one) the $c-h$ graph for all verticals clearly expressed minimum. When going deeper, the cohesion increased steadily (Figs 7 and 8). A similar picture may be seen on the graphs of $\varphi-h$ relationship which indicate the same depth of 2 m and 2.5 m. It was assumed that at the indicated depths clay structure was already damaged and the slope would slip along the surface located at this depth in the near future (Cortellazzo 2002). Actually, the slip happened on the site in the following spring after our investigations.

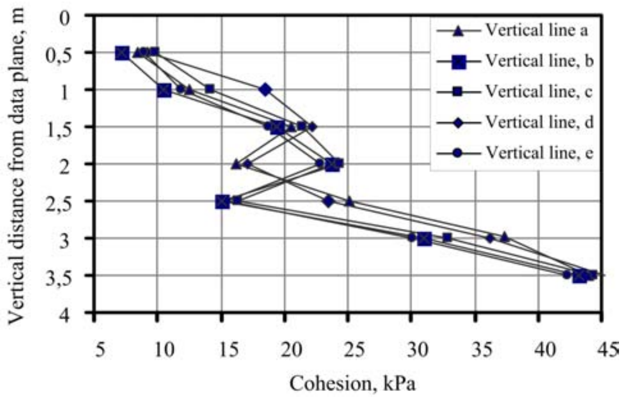

Fig. 7. Graph of $c-h$ relationship according to field data

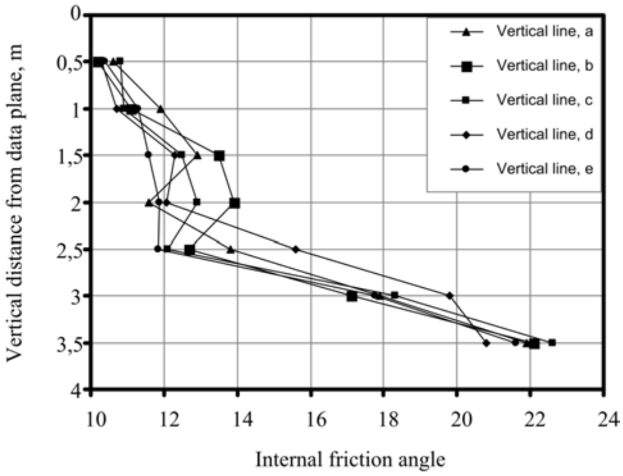

Fig. 8. Graph of $\varphi - h$ relationship according to field data

6. Conclusions

1. The magnitudes of the clay slope stability coefficient obtained by Chugaev's method are increased, those obtained by Maslov's method are reduced, and the values obtained by Terzagi are similar.
2. The factor of water pressure forces (seepage and pore pressure) contains a significant part (30% and greater) of rest factors influence initiating the slip of clay slopes.
3. Loading of a slope top (in addition to a direct increment in forces initiating slip) enhances pore pressure and reduces clay cohesion, thus reducing slope stability significantly.
4. Clay slope stability should be computed by Terzagi method taking into account pore pressure and using ground properties determined in unconsolidated-undrained conditions.
5. Clay slope field investigations by direct cut may help to judge about the stability of the slope and a possible slip of it in the nearest future.

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MOLIO ŠLAITO STABILUMO SKAIČIAVIMAS

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Santrauka

Moliui būdinga filtracijos koeficiento priklausomybė nuo slėgio gradiento ir nulinis vandens laidumas, kai gradientas mažesnis už pradinį. Dėl šios priežasties depresijos kreivė molio šlaite yra trumpa ir stati, dažnai ji sutampa su šlaitu. Staiga apkrovus šlaitą ir padidėjus gniuždymo įtempimams iki dviejų kartų sumažėja molio sankiba. Skaičiuojant šlaito stabilumą, neįvertinus šių svarbių aplinkybių, gaunami klaidingi, per daug optimistiniai rezultatai. Norint išvengti tokių klaidų, molio šlaito stabilumo skaičiavimuose depresijos kreivę reikėtų tapatinti su šlaito profiliu, o sankibą ir vidaus trinties kampą nustatyti nedrenuoto nekonsoliduoto nesuardytos struktūros molio bandymo lauko sąlygose metodu. Hipotetinį šliaužimo kūną apskaičiavimuose reikėtų dalyti į vertikalias prizmes ir vandens slėgio jėgas skaičiuoti kiekvienai prizmei atskirai, o ne visam šliaužimo kūnui iš karto, kaip siūloma kai kuriuose metoduose.

Reikšminiai žodžiai: šlaitų stabilumas, sankiba, vidinės trinties kampas, molio konsolidacija, depresijos kreivė.

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