

Impact of the method of determining soil strength parameters on the results of slope stability analyses

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Abstract: The article discusses the assessment of the impact of changes in mechanical parameters based on the example of the stability of a slope in Czerna, southern Poland. Stability analyses were performed on the basis of field and laboratory tests as well as numerical simulations. The article refers to studies conducted in 2010 after landslide movements following intense rainfall, and in 2018 after slope stabilisation. The results of the study show that in the case of landslide stability analysis, the use of an individual approach to determining the strength parameters of the soil, based on the mapping of soil conditions, enables the researcher to obtain reliable test results. They showed that the testing methodology for cohesion and angle of internal friction according to Eurocode 7, which is based on the determination of effective strength parameters, may return falsely high values and thus result in excessive values of the factor of safety F_s .

Keywords: slope stability analysis, cohesion, angle of internal friction, FEM modelling

INTRODUCTION

Landslides are one of the most significant natural hazards in Poland. Over 90% of them occur in the Flysch Carpathian area (Chowaniec et al. 2012, Bednarczyk 2018). Many years of research within the framework of the SOPO, the Landslide Counteracting System – LCS (SOPO n.d.), allowed them to be accurately identified (Grabowski et al. 2008).

Outside the Carpathian area, landslides are still a significant problem. In many cases, the factors influencing the initiation of mass land movements in the area of the Carpathian foredeep differ from those described in the flysch (Bednarczyk 2018, Rączkowski & Mrozek 2002). These mass movements often take place in inhabited areas with developed infrastructure. In such conditions, the main problem is to provide reliable values of slope

stability indices, allowing for rational risk assessment. A reliable assessment of stability conditions is the basis for the design, construction, and use of buildings located within landslide hazard zones.

The reasons for the loss of stability of slopes are complex. The change in the distribution of stresses in the soil is determined by a number of factors of natural and anthropogenic origin, leading to the initiation of landslide movements. The most significant factor of natural origin affecting the triggering of mass movements is intense precipitation (Rączkowski & Mrozek 2002, Sahin 2004, Gil & Długosz 2006, Filho & Fernandes 2019, Nguyen & Likitlersuang 2019, Peranić et al. 2020) and the strong influence of the intensity and duration (Rahimi et al. 2011, Cotecchia et al. 2015, Lollino et al. 2016, Pedone et al. 2022, Peranić et al. 2022). Other natural factors include the degree of complexity of the geological structure, the existence of structural discontinuities in the form of contact surfaces and tectonic disturbance surfaces, the shape and dimensions of the slope, the inclined deposition of ground layers in accordance with the slope inclination, the type of soils in the area and their physical and mechanical parameters, the degree of plasticity and susceptibility of soils to plasticisation after saturation, the existence of old landslide surfaces in the rock mass, water pressure on the upper poorly permeable ground layers causing a reduction in shear strength, the formation of seepage pressure forces in the slope, etc. (Bizjak & Zupančič 2009, Cha & Kim 2011, Federico et al. 2012, Cano & Tomas 2013, Pasierb et al. 2019, Fernández et al. 2020, Peranić & Arbanas 2020, Wistuba et al. 2021).

The experience gained by the authors from studying approximately 100 landslides in the Carpathian Flysch area indicates that the triggering of landslides in the area of buildings and infrastructure was also influenced by anthropogenic factors such as construction works in the area of a landslide, undercutting of the slope base by an excavation, loading of the slope by an embankment, improper drainage, directing rainwater to the landslide, vibrations and tremors caused by road traffic etc. (Kaczmarczyk et al. 2012).

The theoretical solution of the slope stability problem requires the selection and application of appropriate mathematical solutions describing

stress and deformation tensors in the soil medium (Abramson et al. 2002). In such cases, numerical methods are a helpful tool for assessing stability (Lane & Griffiths 1997, Griffiths & Lane 1999, Kundu et al. 2018).

Continuous development of advanced software for slope stability analysis, including numerical modelling, enables better mapping of the complexity of geological and engineering conditions to be undertaken. The ability to predict changes in the state of stress and deformation in the rock mass significantly facilitates the safe and economically justified design of landslide protection measures. However, for engineering design in landslide hazard zones, it should be remembered that any reliable assessment of slope stability should be based not only on accurate studies of the soil structure, the identification of factors affecting slope stability, and the selection of appropriate computational tools, but also on the use of applicable and current values of geotechnical parameters in the analyses (Savely 1990, Shephard et al. 2019, Wu & Tung 2020). This is particularly important when a loss of stability is observed in a slope already developed by humans (Kaczmarczyk et al. 2012).

The selection of the best method for determining the strength parameters of soil samples collected from landslide hazard zones is not always obvious, as soil strength parameters are not a material constant. This means that the values of the angle of internal friction and cohesion depend not only on the physical properties of the samples and the genesis and lithology of the soils, but also on the methodology used to determine these parameters. This may lead to a situation where soil strength parameters determined according to different standards (or using different test methods) may differ significantly, greatly affecting the results of engineering calculations (Tan 2006, Wu 2013, Choo et al. 2019, Jin et al. 2020, Xu et al. 2020). This problem is particularly relevant in relation to the results of slope stability analyses. In practice, differences in shear strength parameters as small as several percent may have far-reaching consequences, e.g. on decisions regarding the necessity to reinforce the slope or resettle inhabitants from the landslide hazard zone.

The main objective of the research and analysis presented in this article is to determine how the

results of numerical analyses of slope stability are influenced by the methodology of determining the strength parameters (cohesion and angle of internal friction) of soils using different testing procedures. The decision was made to achieve this objective by verifying whether the method of testing soil strength parameters significantly affects the results of their determinations. The implementation of the test results in the numerical slope model enabled an assessment of the possibility of using the effective strength parameters recommended in Eurocode 7 (Polski Komitet Normalizacyjny 2008, 2009a) for slope stability analysis.

MATERIALS AND METHODS

This section presents the theoretical basis and methodological framework used to evaluate how different procedures for determining soil strength parameters affect the results of slope stability analyses. Particular attention is given to the selection of laboratory testing conditions that adequately represent the stress, drainage, and hydraulic conditions occurring in the analysed slopes.

Theoretical introduction

By Coulomb's (1776) assumption, shear strength (τ) is a function of normal stress (σ_n), angle of internal friction (φ), and cohesion (c).

The effective stress principle, formulated 150 years later by Terzaghi (1923, 1936), specifies that shear strength is not a function of total normal stress but of the difference between total normal stress and pore-water pressure in the soil (u).

Thus, in order to determine the effective strength parameters, a testing method should be selected in which no change in pore-water pressure occurs within the sample during the test, or in which changes in pore-water pressure are measured during testing.

Triaxial compression or direct shear apparatus are the techniques most commonly used to determine strength parameters. In the triaxial compression apparatus, depending on the consolidation conditions of the samples, the drainage conditions, the rate and direction of changes in principal stress, and the changes in pore water pressure, several different ways of determining strength parameters can be used when loading the samples.

The three most commonly used methods are: unconsolidated undrained (UU), consolidated undrained (CU) or consolidated drained (CD).

The methodology of soil strength testing based on Eurocode 7 guidelines (Polski Komitet Normalizacyjny 2009a) assumes that the method of consolidation and loading of samples should be selected so as to obtain effective values of the angle of internal friction and cohesion. According to these guidelines, tests in the direct shear apparatus should be performed under 'drained conditions.' Negative or positive pore-water pressures due to shear should be avoided during testing as they cannot be measured and taken into account in the interpretation of the test results. In order to maintain the conditions with drainage, the shear rate should be low enough for pore pressure dissipation to occur.

The use of effective strength parameters in calculations is justified by the fact that the water contained in the soil pores has the ability to carry loads (stresses) but does not have shear strength. Wherever there is a possibility of pore pressure dissipating as the load changes, the use of effective values of the angle of internal friction and cohesion for calculations is fully justified. When permeable soils are loaded, water drains out very quickly, and the pressure increase caused by the loading is negligible $\Delta u \approx 0$. In this case, the effective stress (σ') is nearly equal to the total stress $\sigma' = \sigma_n$. In less permeable soils, the time required for the pore pressure to dissipate is longer, but with the slow load increment occurring during the construction, it is usually sufficient for almost the entire load increment to be transferred to the soil skeleton.

With the rapidly increasing ground load, water pressure increases in proportion to the change in stress $\Delta u \approx \Delta \sigma$. In this case, despite the increase in stress, the shear strength does not increase, and as a result, the value of the angle of internal friction for total stress is lower than the value of the angle of internal friction for effective stress. According to the authors, such a situation occurs in the colluvium of landslides built of Flysch formations during the triggering of mass movements, when the rapid movement of poorly permeable soil masses causes a sudden increase in pore pressure that is unable to dissipate. The occurrence of high

pore-water pressures results in a decrease in the effective shear strength relative to the strength determined for total parameters. This is because the effective stress, which is the difference between the total stress and the pore pressure ($\sigma' = \sigma - u$) in the absence of the possibility of excess (incremental) water pressure dissipating, is significantly less than the total stress. Consequently, despite high values of the angle of internal friction, the effective strength τ' also decreases (Polski Komitet Normalizacyjny 2008, 2018a). According to the interpretation of Eurocode 7 (Polski Komitet Normalizacyjny 2009a), testing without consolidation and drainage (in the UU-unconsolidated, undrained triaxial compression test) will only permit the determination of undrained shear strength (c_u). In this type of test, the standard assumes that under 'undrained' conditions, normal stresses acting on the shear plane do not affect the shear strength of the soil. In theory, this means the complete disappearance of internal frictional resistance in the soil ($\phi' = 0$). In practical terms, even under fully saturated conditions ($S_r = 1$), such a state is unlikely to occur. In practice, even with a very rapid build-up of shear stress, the value of the angle of internal friction is always greater than zero, but usually significantly less than the effective value of this parameter.

The authors of the Eurocode 7 standards (Polski Komitet Normalizacyjny 2008, 2009a) emphasise that 'one should only consider undertaking strength tests of soils completely saturated with water or dry soils.' Since it is not possible to precisely measure the pore-water pressure of unsaturated soils, the testing methodology proposed in the standard is valid from both a theoretical and mathematical point of view; however, the test results obtained from this type of testing in some situations do not reflect the behaviour of the soil under real conditions. This is especially true for the poorly permeable soils found in landslide colluvium, which are unconsolidated and unsaturated as a result of displacement.

The authors of Eurocode 7 (Polski Komitet Normalizacyjny 2009a) point out that 'uniaxial compression tests and undrained, unconsolidated triaxial compression tests do not necessarily represent (reflect) the *in situ* shear strength of the soil.'

In practical solutions, i.e. stability analyses of actual landslides, the use of both effective strength parameters and undrained shear strength parameters (c_u) is questionable due to the extremely different results.

According to the authors, the choice of testing method for strength parameters should be adapted to the specific engineering task.

The methodology for determining the cohesion values and the angle of internal friction of soil samples defined in older literature (Lambe & Whitman 1977, Kaczyński 1984, Glazer 1985, Wiłun 2013) recommends the maximum possible adjustment of the method of loading model samples to the conditions of stress changes occurring in real conditions.

The authors' experience shows that apart from classical methods of parameter determination, the methods based on determining residual parameters (especially in consolidated clays and clay shales) and methods after saturation of the cut surface of samples, e.g. in silty soils sensitive to changes in water content due to saturation, are highly effective (Borecka & Kaczmarczyk 2007, Kolano & Cała 2011).

In the analysis of slope stability, the methodology of laboratory tests should be selected so as to best represent the processes occurring in the soil. The most reasonable course of action, therefore, is to use such methods that provide maximum similarity between model tests and the states and processes occurring both now and in the history of the natural ground base. The selection of appropriate methods for testing soil strength parameters should therefore take into account the spatio-temporal analysis of the stress distribution in the soil substrate.

Description of the research area

The research area was a landslide in Czerna, located in southern Poland (Fig. 1).

The geological-engineering profile is composed of Quaternary loess formations with a thickness of up to 15 m and Triassic and Carboniferous clays and clay shales. The study area is subject to complex hydrogeological conditions. This is due to the lithofacial diversity of the formations, strong tectonic involvement and varied morphology (Płonczyński & Łopusiński 1993).

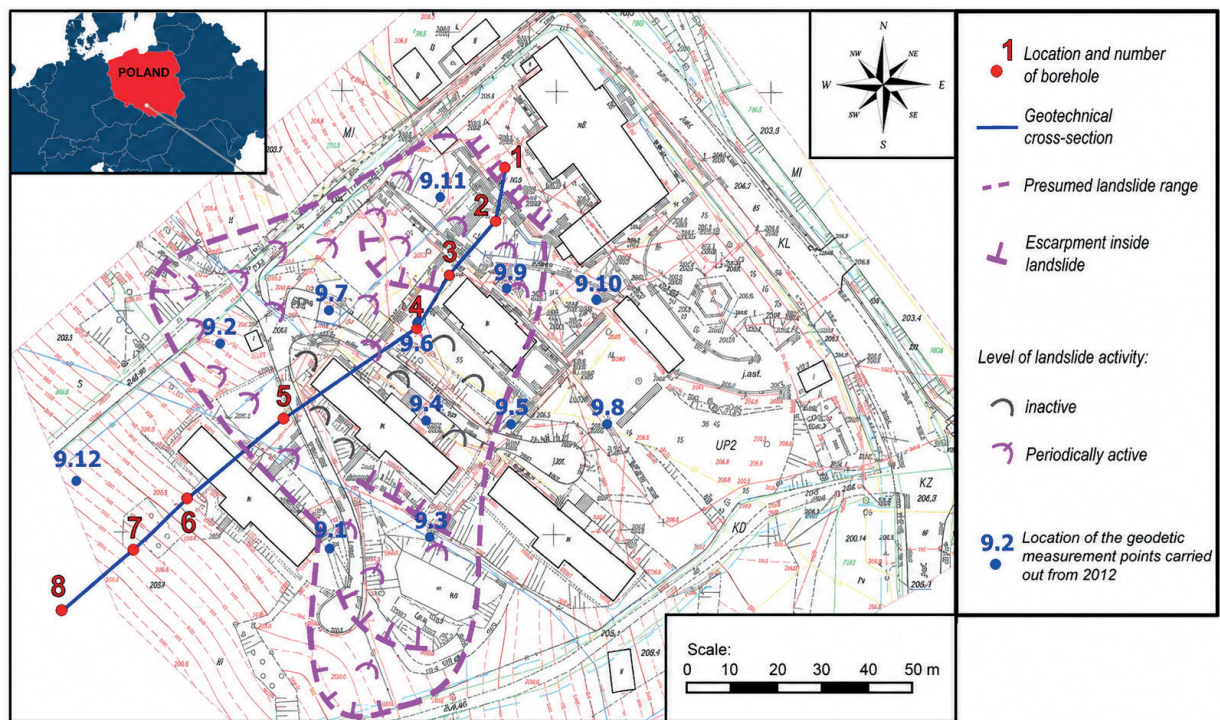


Fig. 1. Layout of the research area in Czerna (southern Poland) with the location of geological works and landslide forms, based on a site visit and archival data

The research area slopes in a northeasterly direction at an angle of about 15° towards the Czerna River valley. The elevations of the research area are between 308.0 and 324.0 m a.s.l. Groundwater is supplied by infiltrating precipitation. Groundwater runoff is in a northeast direction. The landslide area is 1.7 ha, its length is 175 m and its width is 130 m (Chowaniec et al. 2012). The landslide is a consistent translational landslide. Fieldwork revealed numerous seeps within Quaternary loess soils and Triassic clay deposits.

Atmospheric precipitation had been occurring since the beginning of May 2010. In May, the total number of days with precipitation was 28, while in June, it was 15. The amount of rainfall in May 2010 exceeded 350% (locally over 400%) of the long-term monthly norm from 1971 to 2000, while in June, it reached approximately 170% of the norm for that month. Particularly intense rainfall occurred in the second half of May. The first stage of mass landslide occurrences resulted from an uninterrupted three-day rainfall period from the evening of May 15 to the evening of May 18,

when more than 200 mm of rain fell on already saturated ground. The second stage of intensified mass movements was caused by continuous rainfall from May 30 to June 4, 2010, with total precipitation ranging from approximately 118.8 mm to 192.3 mm. The rainfall not only led to a significant increase in soil moisture but also to the creation of seepage pressure in the subsoil. The accumulation of water in surface layers and deeper zones weakened the strength parameters of the soil and rocks, which resulted in numerous, sometimes rapid, mass movements. Although the spring rains were the most intense, total precipitation during the summer also significantly exceeded long-term norms (in August 2010, over 162 mm of rain fell, compared to an average of 80 mm). This led to another phase of landslides in September 2010. These were the result of both accumulated soil moisture from previous months and additional rainfall that further increased the already high level of ground saturation. Landslides were triggered in areas where the substrate structure had previously weakened, leading to further slope degradation (Fig. 2).

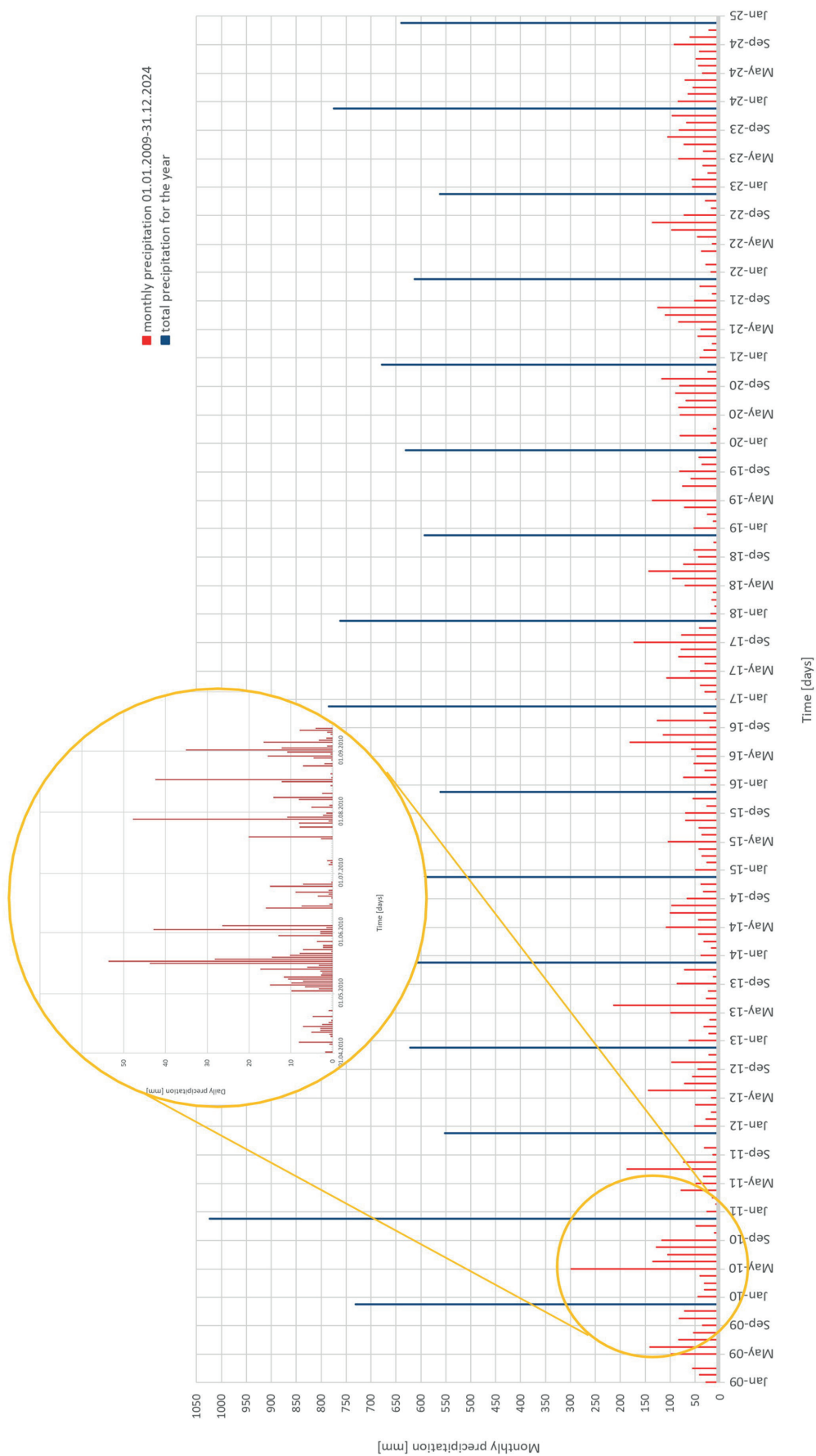


Fig. 2. Amount and intensity of rainfall between 2009 and 2024 indicating the moments of development of landslide processes

Numerous tensile fractures, deformations of infrastructure components, and leanings of retaining structures were observed during the inventory work. The effects of mass movements are particularly evident at the bottom of the slope near retaining walls and a pond. The morphological forms typical of landslides are poorly visible. However, the susceptibility of the research area to the development of landslide processes is evidenced by the disturbed layering in the lower part of the slope. The silt layer was covered by older clay deposits of the colluvium (Lenduszko 2010). This process occurred prior to the development of the slope. Strong transformations of an anthropogenic nature – construction of public utility buildings and a road – have obliterated the effects of the landslide.

The choice of the Czerna landslide as the subject of this study was due to two primary factors. The first was the relatively low complexity of the geological setting that allowed us to eliminate most of the usual factors affecting the results of

stability analyses. The second factor involved the fact that the landslide is fairly well researched, based on the results of studies conducted directly after the landslide was triggered in 2010 (Lenduszko 2010) as well as detailed geological and engineering studies carried out by the authors of this article in 2018 (Kaczmarczyk et al. 2018).

Before the occurrence of landslide movements and damages observed in 2010, no geotechnical monitoring was carried out on the slope. A geodetic monitoring network was only built to register displacements after the damage occurred and the slope was stabilised in 2012 (Fig. 1). The monitoring carried out in 2012–2021 showed displacements of up to a maximum of 9 mm in points 9.1, 9.2 and 9.3 in terms of height ordinates, which could only result from the local consolidation of subsurface soils.

Stability analyses were performed on the cross-section running in the central part of the landslide, where the greatest evidence of its activity was observed (Fig. 3).

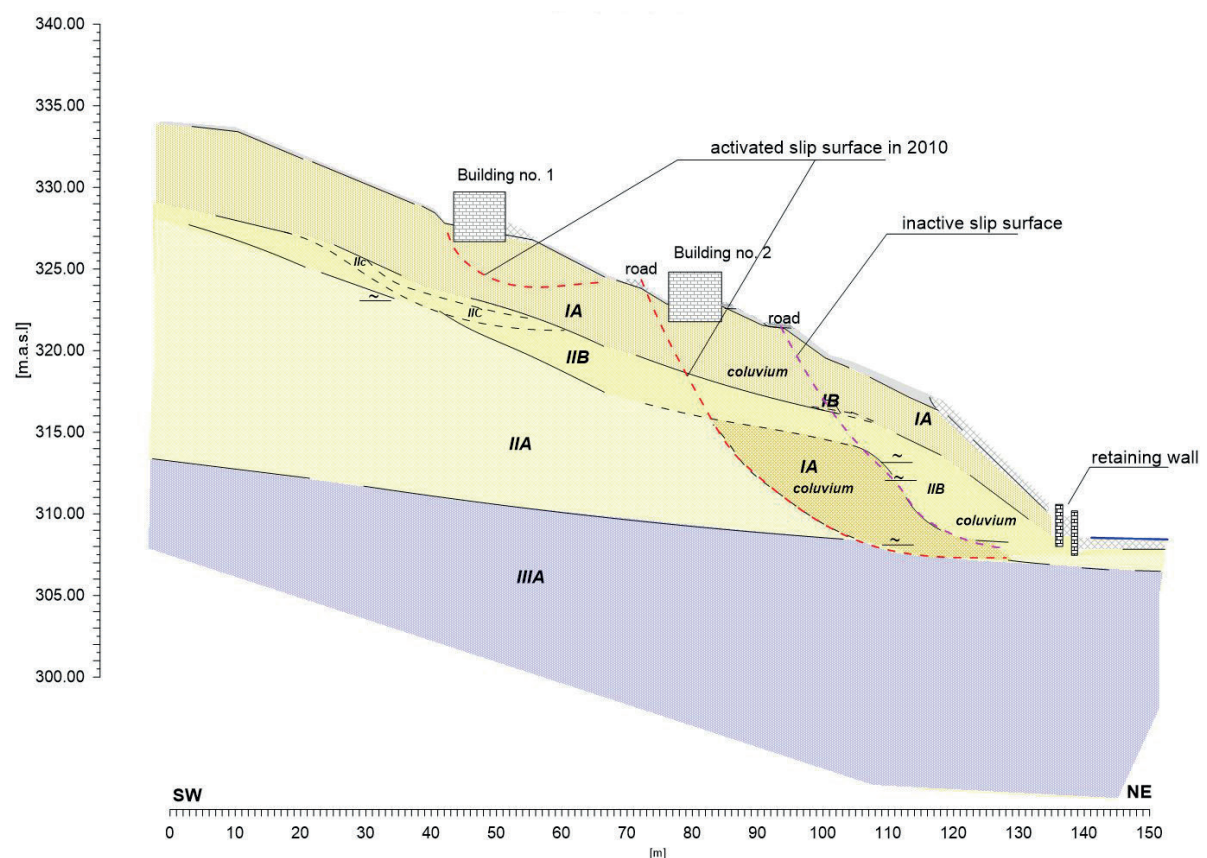


Fig. 3. Geotechnical cross-section

Three geological and engineering layers were distinguished:

- Layer I. Consists of Quaternary loess and loess-like clay loams in semi-compact (layer IA) and plastic (layer IB) states. The thickness of this layer increases in the upper parts of the slope and varies from 1.8 to 5.3 m. The IB layer was found locally in the loess and clay contact zone. The layer has a thickness of approximately 0.1–0.2 m.
- Layer II. Consists of Triassic residual clays in semi-compact (layer IIA), hard plastic (layer IIB) and plastic (layer IIC) states. Layer IIC occurs locally in the contact zone of older soil with silts. In the lower part of the slope, the spatial arrangement of the layers is disturbed. This demonstrates the occurrence of landslide processes in historical times.
- Layer IIIA. Consists of Carboniferous clay shales interbedded with sandstones. This layer occurs at a depth of some 15–16 m below ground level.

The geological and engineering survey was conducted in 2018 in accordance with Eurocode 7 (Polski Komitet Normalizacyjny 2008, 2009a). In the course of the fieldwork, eight boreholes were drilled, of which two were made with a double-core drill and six by window sampling. The boreholes were drilled to depths ranging from 5.0 to 24.5 m below the surface. A total of 104 m of boreholes were drilled.

The water table was not found during the fieldwork. Archival documentation (Lenduszek 2010) shows that in 2010, during the period of intense rainfall, there were short-term exudates in the substrate. This was due to improper drainage of rainwater within the slope, especially in the contact zone of medium-permeable silty soils and very poorly permeable clay. This phenomenon stopped immediately after the renovation of the rainwater drainage system.

The retaining walls shown in Figure 3 are structures built prior to the formation of the landslide as a support for the lower part of the slope to create parking spaces. Due to their size and shallow foundation, they did not constitute a retaining structure. In the model, they were taken into account as a concrete block anchored at the depth of the actual foundation with a volumetric weight appropriate for the material from which they were built. Buildings, visible in the cross-section in Figure 3, were taken into account in a similar way.

Methodology for determining geotechnical parameters

For stability analyses, the authors made use of the results of archival research from 2010 and the results of research performed by the authors of this article in 2018 after the landslide protection works. Several variants of geotechnical parameters were used to meet the assumptions of the study.

The results of the archival research include parameters determined in 2010 during a period of triggering of mass movement when water conditions were not regulated (Lenduszek 2010). The water content, soil bulk density, and plasticity limits were determined according to PKN-CEN ISO/TS 17892-1:2009 (Polski Komitet Normalizacyjny 2009b), PKN-CEN ISO/TS 17892-2:2009 (Polski Komitet Normalizacyjny 2009c), and PKN-CEN ISO/TS 17892-12:2009 (Polski Komitet Normalizacyjny 2009f) (Table 1). The values of the total strength parameters were determined according to ISO 17892-10 (Polski Komitet Normalizacyjny 2009d) (Table 2).

The results of the research performed by the authors of this article in 2018 included:

- a) Determinations of water content, soil bulk density, plasticity state and filtration coefficient carried out in accordance with PN-EN ISO 17892-1:2015 (Polski Komitet Normalizacyjny 2015a), PN-EN ISO 17892-2:2015-02 (Polski Komitet Normalizacyjny 2015b), PN-EN ISO 17892-12:2018-08 (Polski Komitet Normalizacyjny 2018b) and PKN-CEN ISO/TS 17892-11:2009 (Polski Komitet Normalizacyjny 2009e) (Table 1).
- b) Determinations of effective strength parameters determined after consolidation of soil samples under slowly increasing shear stress (CD method) carried out in accordance with PN-EN ISO 17892-9:2018-05 (Polski Komitet Normalizacyjny 2018a) in an AUTOTRIAX Automatic Triaxial System, model 28-WT4412, by Wykeham Farrance.
- c) Determinations of total and residual strength parameters, as well as strength parameters after saturation of the shear plane, were carried out using a direct shear apparatus (Wykeham Farrance, model 27-WF25510).
- d) *In situ* determination of undrained shear strength using a SLVT conical vane probe.

The direct shear tests referred to in point c) were performed without prior consolidation, at a shear rate of 1.0 mm/min for the determination of total strength parameters and 0.1 mm/min for the determination of residual strength parameters. The total strength parameters were determined using the UU method at normal stresses of 50, 100, 150, and 200 kPa. Before shearing, the samples were subjected only to a short stabilisation period to equalise the pore-water pressure within the sample.

Due to its box-type design, the direct shear apparatus does not prevent pore-pressure dissipation; however, at sufficiently high shear rates, this process is significantly limited. At a shear rate of 1.0 mm/min, the load increment is sufficiently rapid that water does not have enough time to drain from the sample before failure occurs. The purpose of this approach was to reproduce the shear strength of the samples at moisture conditions corresponding to their natural state. The procedure was intended to simulate UU conditions, i.e. unconsolidated and undrained conditions, while varying only the vertical stress. In the total-stress tests, the shearing time was selected in each case to prevent complete dissipation of pore-water pressure and to avoid consolidation of the sample during shearing.

For the determination of residual values of the angle of internal friction and cohesion, the tests were continued until a deformation of approximately 20–25% was reached. The carriage of the apparatus was then returned to its initial position, and the sample was sheared again. This procedure was repeated several times until a constant minimum value of shear resistance was obtained.

Strength parameters were also determined after saturation of the shear surface. Before testing, the shear box containing the soil sample was flooded with water and left for at least 12 hours to saturate the shear surface. After saturation, the shear strength was determined at four successively increasing normal stress levels of 50, 100, 150, and 200 kPa.

According to the authors, this testing scheme provides the best possible representation of soil behaviour under natural conditions and facilitates

a comprehensive analysis of the results. The determination of residual strength was intended to identify changes in strength parameters occurring along disturbed and eroded landslide slip surfaces as the soil moves along the discontinuity surface. Tests conducted after saturation of the shear surface were intended to represent changes in the physical and mechanical properties of the soil resulting from an increase in water content at the slip surface.

All laboratory tests were performed on samples retrieved from borehole cores (Category A samples according to Eurocode 7 (Polski Komitet Normalizacyjny 2009a)).

Modelling of slope stability

Calculations of the stability of the study landslide were performed using the GEO5 ver. 2021 Slope Stability program, which can be used to check slope stability for both circular and polygonal slip surfaces (Fine Ltd. 2018). The computational methodology took into account various approaches using boundary equilibrium methods. For the purpose of this article, the slope stability was tested by the Bishop and Morgenstern–Price methods. Calculations were performed separately for the upper and lower parts of the slope. The division was made due to two main slip planes having been defined. The upper part of the slope covers a smaller area and lies on a shallower (approx. 2 m b.g.l.) slip plane located between buildings 1 and 2. The analysis of the stability of the lower part of the slope was carried out for the area located below building 2, where there is a larger slip plane located at a depth of about 10 m b.g.l.

RESULTS

This section presents the results of laboratory tests used to determine the strength parameters of the investigated soils and evaluates the influence of the adopted testing methodology on the obtained values. The resulting parameter sets were subsequently used in slope stability analyses to assess their effect on the calculated stability conditions.

Table 1
Summary of geotechnical parameters

Geotechnical layer	Soil type according to PN-EN ISO 14688-1:2018-05 (Polski Komitet Normalizacyjny 2018c)	2010			2018			
		Natural moisture content w_n [%]	Bulk density of soil ρ [Mg/m ³]	Liquidity index I_L [-]	Natural moisture content w_n [%]	Bulk density of soil ρ [Mg/m ³]	Liquidity index I_L [-]	Filtration coefficient k [m/s]
IA	Si, cSi	22.5	2.07	0.15	22.1	2.00	0.10	6.2×10^{-7}
IB	Si, cSi	25.4	1.93	0.47	23.6	1.92	0.36	
IIA	Cl	27.5	2.08	0.05	18.4	2.15	-0.20	5.5×10^{-11}
IIB	Cl	30.2	1.98	0.23	24.5	2.00	0.03	
IIC	Cl	33.7	1.92	0.48-0.60	30.1	1.90	0.28	
IIIA	SH//SA	28.1	2.10	0.07	17.4	2.20	-0.20	-

Table 2
Results of measurements of strength parameters

Geotechnical layer	2010		2018				
	Angle of internal friction φ [°]	Cohesion c [kPa]	Effective angle of internal friction (CD) φ' [°]	Effective cohesion (CD) c' [kPa]	Angle of internal friction (UU) φ_u [°]	Cohesion (UU) c_u [kPa]	Undrained shear strength s_u [kPa]
IA	15.0	18.0	27.0	41.0	21.5 ^a 14.0 ^b 7.3 ^c	20.9 ^a 14.0 ^b 11.8 ^c	52.0
IB	10.0	9.0	20.7	34.0	14.8 ^a 8.7 ^b 6.7 ^c	16.5 ^a 12.0 ^b 10.7 ^c	47.0
IIA	12.0	57.0	26.7	74.0	12.0 ^a 13.0 ^b 7.0 ^c	57.0 ^a 26.0 ^b 13.0 ^c	104.0
IIB	9.5	46.0	18.0	55.0	17.4 ^a 9.5 ^b 5.2 ^c	34.0 ^a 19.5 ^b 8.5 ^c	65.0
IIC	5.0	16.0	15.0	33.0	12.0 ^a 10.0 ^b	27.0 ^a 20.0 ^b	43.0
IIIA	-	-	-	-	18.0-23.0 ^a 14.0 ^b 9.4 ^c	118.0-160.0 ^a 26.0 ^b 18.2 ^c	-

^atotal strength parameters;

^bresidual strength parameters;

^cstrength parameters obtained after saturating the shear plane.

Results of laboratory tests

Table 1 provides a summary of the archival results of studies of geotechnical parameters (Lendusko 2010) and the results of studies conducted by the authors of this article.

The results of the strength parameters are presented in Table 2. The values were determined directly in laboratory tests.

The results of the study conducted by the authors confirm that the methodology of the laboratory tests has a very significant effect on the values of the shear strength parameters of soils. The values of effective strength parameters determined after consolidation with drainage (CD method) are significantly higher than the total strength parameters determined without consolidation (UU method). In the case of the

samples considered, the effective values of the angle of internal friction are on average about 50% higher than the values of the total strength parameters determined in 2018, i.e. after protective works, and about twice as high as the total strength parameters.

Modelling results

Calculations were performed using the Bishop method for a circular surface and the Morgenstern–Price method for a polygonal slip surface after optimisation, i.e. for a surface with the most unfavourable ratio of stabilising forces to sliding forces. The strength parameters for the six variants presented in Table 3 were used in the calculations.

The results of the model calculations are presented in Figures 4–9 and in Table 4.

Table 3
Design variants

Variant I	Variant II	Variant III	Variant IV	Variant V	Variant VI
Total strength parameters determined in 2010	Effective strength parameters determined in 2018	Total strength parameters determined in 2018	Residual strength parameters determined in 2018	Strength parameters obtained by shear plane saturation as determined in 2018	Shear strength without drainage determined in situ in 2018

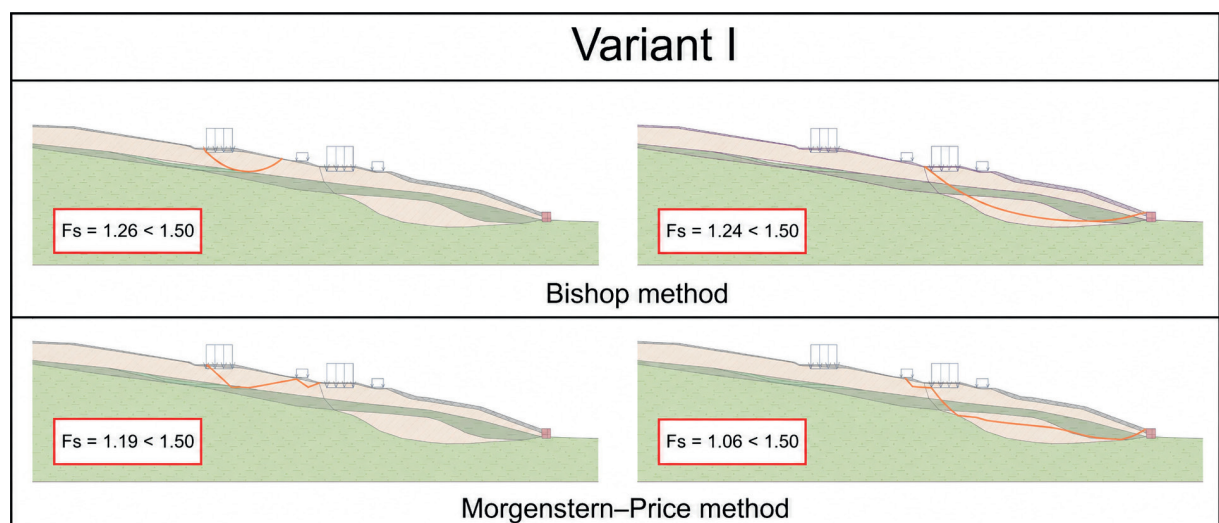


Fig. 4. Graphical representation of the modelling results for variant I

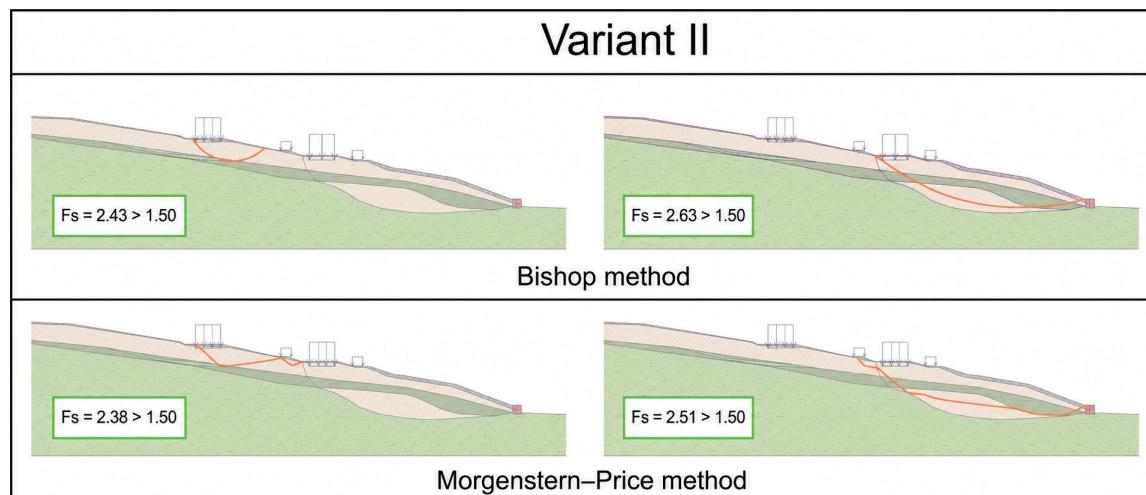


Fig. 5. Graphical representation of the modelling results for variant II

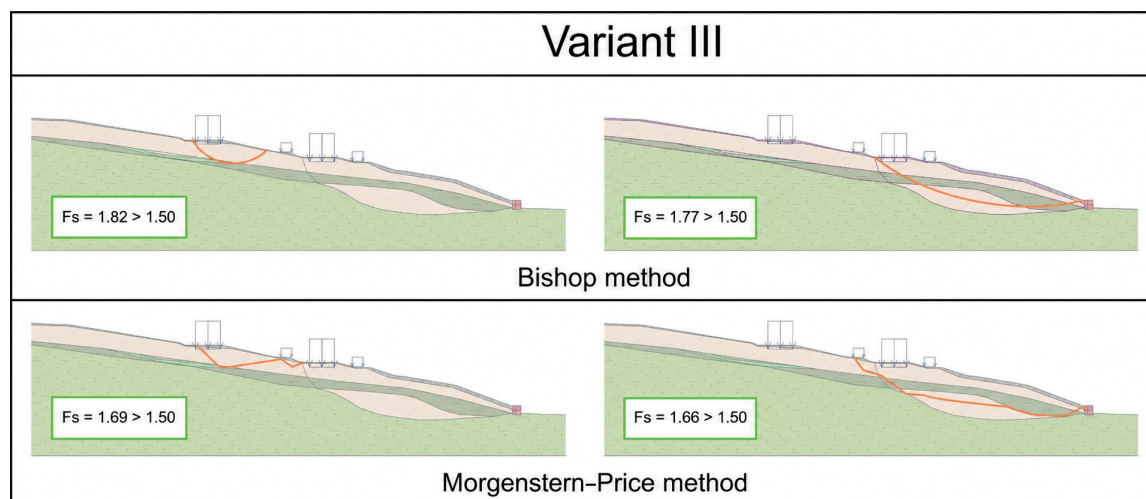


Fig. 6. Graphical representation of the modelling results for variant III

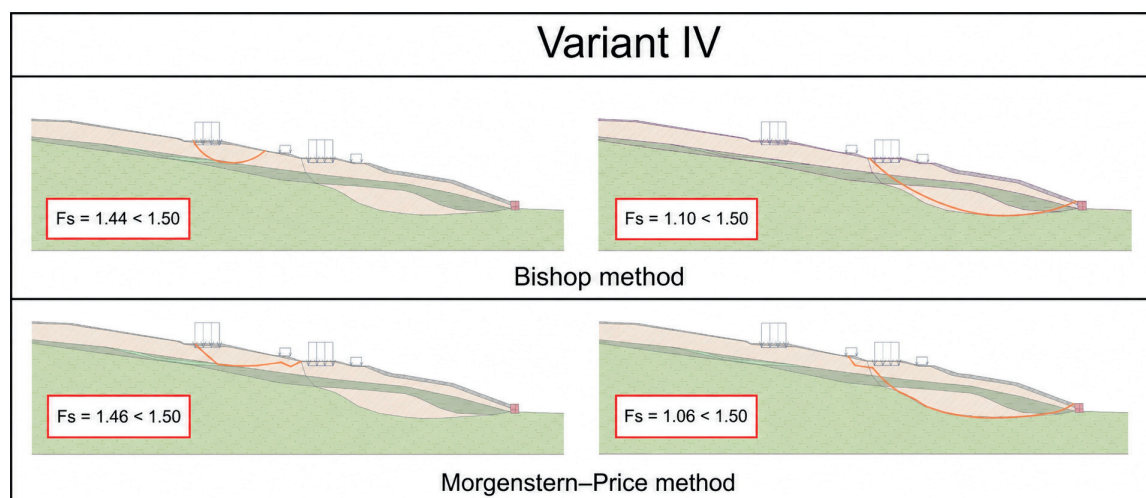


Fig. 7. Graphical representation of the modelling results for variant IV

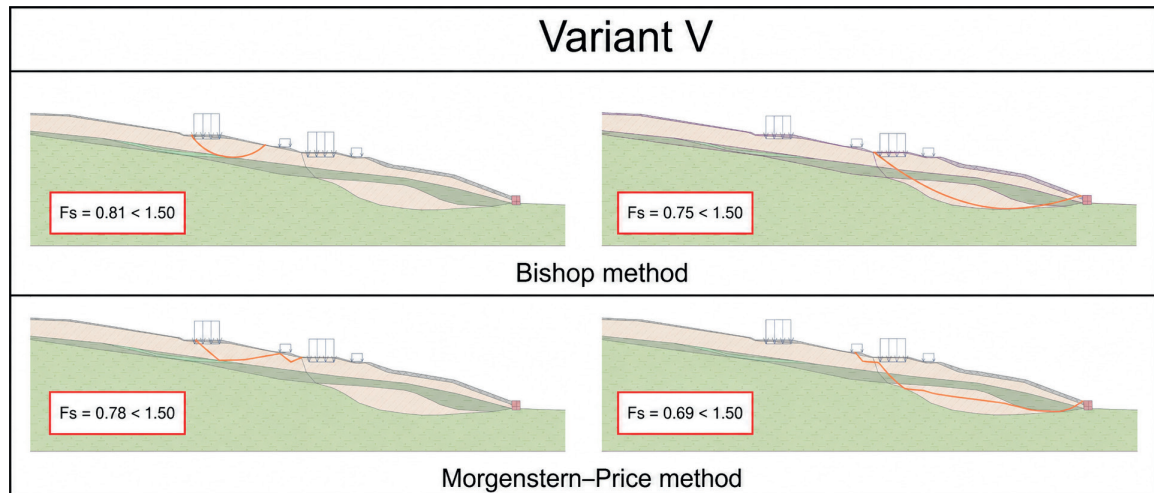


Fig. 8. Graphical representation of the modelling results for variant V

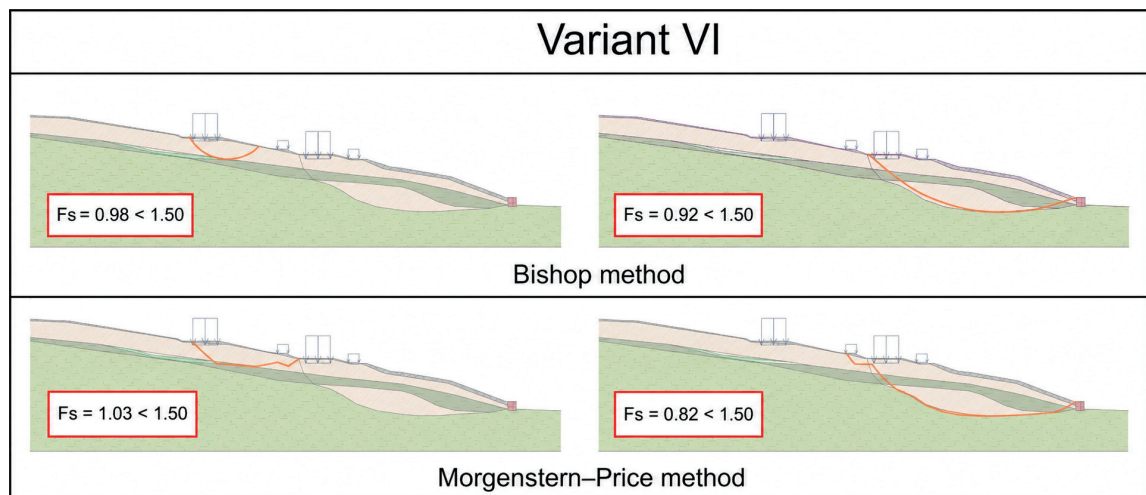


Fig. 9. Graphical representation of the modelling results for variant VI

Table 4
Results of stability analyses

Variant	Calculation method	Factor of safety	
		Bottom of the slope	Top of the slope
I	Bishop method	1.24	1.26
	Morgenstern-Price method	1.06	1.19
II	Bishop method	2.63	2.43
	Morgenstern-Price method	2.51	2.38
III	Bishop method	1.77	1.82
	Morgenstern-Price method	1.66	1.69
IV	Bishop method	1.10	1.44
	Morgenstern-Price method	1.06	1.46
V	Bishop method	0.75	0.81
	Morgenstern-Price method	0.69	0.78
VI	Bishop method	0.92	0.98
	Morgenstern-Price method	0.82	1.03

DISCUSSION

The large differences observed in the values of the strength parameters for soils with poor load-bearing characteristics may be the reason for the controversy concerning the selection of the proper methodology for their determination. Depending on the method of determination, different values of the angle of internal friction and cohesion can be obtained for the same type of soils in the same layer. However, the variation in the strength parameters has a significant impact on the results of stability analyses.

For the landslide described here, the differences in the modelling results are very significant. For variant II, the factor of safety values ranged from 2.38 to 2.63, which means that the sum of the stabilising forces is approx. 2.5 times higher than the sum of the sliding forces. The factor of safety values for variant III are significantly lower. Calculations made for variant I (a few weeks after the landslide became active) indicate a state of near limit equilibrium – the factor of safety values vary from 1.06 to 1.26. Calculations for variant III, i.e. after the protective works had been carried out (capture and discharge of rainwater), indicate that the factor of safety values increased by ca. 50% in relation to the parameters considered in variant I.

After the landslide was triggered in 2010, a system of subsurface and surface drainage was designed and constructed across the entire slope. The system conveyed rainwater and groundwater to the river flowing through the valley at the slope toe, thereby promoting the dissipation of excess pore water pressure within the soil mass.

The results of analyses for variant IV are similar to those for variant I. The results of analyses for variant V indicate a loss of slope stability in the situation of accumulation of infiltrating rainwater in the slip plane area. The results of the analyses for variant VI suggest that the slope, which in reality maintains stability, is unstable.

The results of this case study indicate that using only effective parameters of shear strength for slope stability analyses is unreliable. Adopting the values of effective parameters (c' , ϕ') for the calculations comes with risk, as the results of the slope stability analysis even for active landslides

indicate that the slope is stable. Adopting the parameters without consolidation does not rule out error; it will, however, be on the safe side.

The effective values of the angle of internal friction are relatively high, often several times greater than the total values. In cohesive soils tested using the CD and CU methods, when strength parameters are calculated using Terzaghi's formula (1923, 1936), the differences in these values increase with the increasing degree of plasticity, i.e. in the compact state, the differences are insignificant, while in the plastic and soft-plastic states, they are excessively large. In stability analysis cases where the values of the angle of internal friction exceed the angle of slope collapse, high values of the factor of safety are obtained. The results of the stability analyses of the study landslide showed that when carrying out the calculations, adopting the effective values of the shear strength parameters obtained for the samples after consolidation and shearing at a low rate of shear stress build-up indicates a large factor of safety.

Analytical results obtained for the same shear samples without consolidation at rapid load increments show a much lower F_s .

In practice, a small (3–5%) increase in natural moisture content produces a significant, approximately twofold, decrease in the strength parameters of loess-like formations (silts) (Chowaniec et al. 2012, Kaczmarczyk et al. 2012). Adopting effective strength parameters for calculations appears to be valid only for slopes that are well-drained or composed of freely permeable formations where there is potential for the rapid dissipation of increases in pore water pressure. In the case of the landslide analysed in Czerna, the residual clays (layer IIIA) occurring below loess (layer IIA) are very poorly permeable formations. As a result, rainwater infiltrating through the moderately permeable near-surface formations accumulates in the layer contact zone, significantly affecting slope stability.

It is also worth noting that even the most advanced software for stability analysis is only a tool, and its results cannot be accepted uncritically. The results of the numerical modelling conducted show how big the scattering of factor of safety F_s values can be depending on the method of

strength parameter determination applied, taking into account the fact that the same slope model is used in the calculations and to define the same models of slopes.

CONCLUSION

Many elements contribute to the differences in the results of stability analyses. These include the approach of the geologist or geotechnical engineer, the calculation factors selected, and the type of calculation method selected. The geotechnical design parameters of the soil layers adopted for the model are, however, of the greatest importance. In the case described, these differences are particularly evident from the perspective of different approaches to determining strength parameters.

In laboratory tests with consolidation and drainage, high values of the effective angle of internal friction are obtained in clay soils regardless of the consistency state. The strength parameters determined without consolidation, and thus without the time required for pore pressure to dissipate, have significantly lower values.

According to the authors, in the analysis of the stability of a slope made of poorly permeable formations, the optimal method for determining the strength parameters is to determine the total parameters in the direct shear apparatus without consolidation of the sample. This method allows the test to be performed under conditions similar to the stress distribution in the ground substrate on a slope where relatively rapid landslide movement prevents the pore pressure from dissipating. They show that in case of intensive precipitation, a lack of effective drainage and accumulation of water in soil pores at the interface between layers with different filtration coefficients causes the slope F_s to decrease significantly.

The analyses conducted also indicate the possibility of using the study of residual strength parameters to determine the boundary equilibrium state for a given slope. The results of the analyses for variant IV coincide with those obtained for variant I, whereas the investigated slope actually lost stability. This may indicate that the shear surfaces were water-saturated, although not to the extent represented by the full saturation conditions simulated in variant V.

The results of numerical modelling for the parameters of variant VI proved the unsuitability of using the undrained shear strength (c_u) value, determined in situ using the SLVT probe, in stability analyses. The model also assumes an angle of internal friction of 0° a priori, which results in a falsely underestimated factor of safety.

These case studies show that using only values of effective parameters of shear strength for slope stability analysis is not fully reliable. Depending on the methodology of the laboratory tests, different values of the shear strength parameters (c , c' , φ , φ') can be obtained for the same soil in the same layer and from the same depth interval.

Many elements contribute to the differences in the results of the stability analyses. However, the calculation parameters of the soil layers adopted for the model are of the greatest importance. In the described case, these differences are particularly visible from the perspective of different approaches for determining the strength parameters of the soil.

The results of the analyses obtained for the same shear samples without consolidation with rapid load increase indicate a much lower factor of safety, which much better reflects the stability conditions in the period of triggering of the landslide.

The use of effective parameters in calculations appears to be justified only for well-drained slopes, where increases in pore water pressure can dissipate rapidly. This may apply, for example, to landslides in which the slip surface is defined as a plastic displacement zone from which water has been drained as a result of stabilisation works. These parameters are not suitable for old inactive landslides, where pore fissures may provide pathways for water to reach old slip surfaces and induce conditions intermediate between those represented by variant IV and variant V. For not-well-drained slopes, especially landslides, whose activity occurs after prolonged, intense rainfall or snowmelt, the authors recommend using the total parameters for analysing slope stability.

In landslides where the presence of reflecting surfaces has been found on drilling cores, it is advisable to use residual parameters for the analysis, and in the case of slip surface water logging, the residual parameters after water failure. The comparison of the research results and their influence on the stability analysis indicates that, in the case

of landslides, an individual approach and selection of an appropriate research method are necessary. Therefore, it is important that each stability analysis is verified by the experience of the person responsible for the analysis.

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