Studia Geotechnica et Mechanica, Vol. XXVII, No. 1–2, 2005

INFLUENCE OF STRUCTURAL SURFACES ON THE NUMERICAL ESTIMATION OF SLOPE STABILITY

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Abstract: In the present paper, two major types of soil weakness are considered: they can occur either due to structural damage in the material or due to interlayer contact. The influence of soil weakness on the results of stability calculations was investigated using a real example – a slope with stability-loss hazard. The strength of particular soil layers was examined by strain-controlled undrained triaxial compression tests with pore pressure measurement, whereas the shear resistance for the brown coal–clay contact was determined in a direct shear apparatus. Numerical analysis of slope stability was performed using the modified Janbu method. It has been found that an adequate assessment of soil-structure disturbance and the knowledge on the shear resistance in the contact zone are of paramount importance to the stability analysis of a slope which consists of hard soil series.

1. INTRODUCTION

Research and observations have shown that landslides occur primarily along the weakest structural surfaces such as those between different soil series, especially those between cohesionless saturated soil and cohesive soil. That is why this type of landslide is referred to in the literature as structural landslides.

At the stage of designing the inclination of the slopes, the strength parameters of the surface of weakness are generally unknown, because the tests on the soil specimens from boreholes provide neither a reliable distinction between the weakened zones nor the assessment of their strength parameters. Investigations have revealed that in the course of open-cast mining operations, the shear strength parameters of the weakened structural surfaces take much lower values than those obtained in borehole tests. It is therefore of importance to take into consideration the strength parameters of the weakened surfaces when analysing the slope stability.

In this paper, the problem is exemplified by a slope exposed to stability loss hazard. The hazard region under study includes the western long-term slope of a deep brown-coal opencast. The slope also acts as a protecting pillar for a river.

The age of the slope in the hazard region was over twenty years and a total depth of the excavation approached 100 m. The slope stability loss commenced after cutting a successive bench, the level +125, and providing a 6 m deep dewatering ditch. The potentiality for stability failure motivated us to initiate geotechnical investigations in this region. Since the conventional approach to the problem had not revealed any failure hazard, we decided to analyse the factors affecting the results of stability calculations in more detail than usual. Hence, our analysis included the evaluation of the design parameters, knowledge of the strength properties of the structural surfaces, as well as the choice of the scheme of calculations. The effect of the calculating method was neglected because under determined geological conditions the choice of one or another calculating technique seems to be without any significance.

2. GENERAL CHARACTERISATION OF GEOLOGICAL CONDITIONS

A lithostratigraphical profile is built of crystalline rocks made of granites and granite gneiss (G); Tertiary deposits subdividing into residual soils (R); underlying clays (UC); the first seam of brown coal (BC I); interseam clays (IC); the second seam of brown coal (BC II) and Quaternary (Q) formation: sands and gravels (figure 1). The principal structural surfaces formed by the roof of residual soils as well as by the roofs of underlying clay and interseam clays are inclined with respect to slope excavation. Of the sedimentation contacts, surfaces *A* and *C* are the most active as shown by the landslides in the openpit and by inclinometric observations on the openpit slope. The character and position of the structural surfaces should be attributed primarily to the geological history of the Żytawa-Zittau Brown Coal Basin.

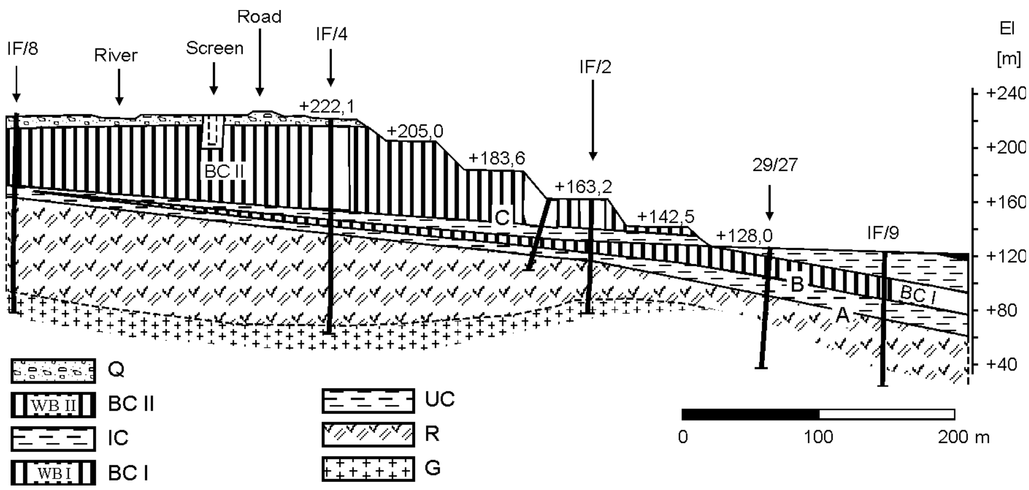

Fig. 1. Typical geological cross-section

The region has been cut by two large gravitaty faults and a number of smaller ones. The latter can be detected only after mining operations and the concomitant decompression processes. The weakness of the contacts between seam II and interseam clays as well as between seam I and underlying clays owes its origin primarily to a greater saturation of the clays in the contact zone and to the decompression processes. The effects induced by decompression take the form of cracks and fissures on the slopes of the excavation. They facilitate infiltration of water into the soil mass, specifically into the coal–clay contact zone. A greater deal of moisture is observed in the clayey zone which is in direct contact with the coal layer.

Summing up, a geological structure creates favourable conditions for the development of structural landslides.

3. RESULTS OF GEOTECHNICAL INVESTIGATIONS

Our considerations will focus on the soil series which are of importance to the stability analysis of the protecting pillar.

Oedometer tests have shown that interseam clay and underlying clays are overconsolidated soils because the overconsolidation ratio (*OCR*) is higher than 2 [1]. According to the definition in the Polish code PN-B-02481: 1998, soils are overconsolidated if the *OCR* is higher than 1 unity. Thus,

$$OCR = \frac{\sigma'_p}{\sigma'_v}, \tag{1}$$

where σ'_p is preconsolidation pressure and σ'_v is actual effective vertical stress.

Shear strength was examined in unconsolidated and undrained triaxial compression tests with pore pressure measurement, at a cell pressure of 0.0 to 0.6 MPa. The triaxial compression tests involved undisturbed specimens collected during drilling operations for the installation of inclinometers. More details can be found in Ref. [4].

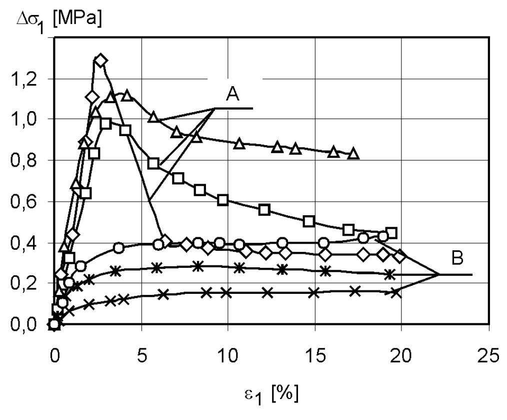

Fig. 2. Typical stress–strain curves

Figure 2 depicts the behaviour of the soil during the tests. It is interesting to note how differently the soil may behave within the same series (interseam clays). The plots representing specimens (A) characterise brittle failure observed in preconsolidated soil of a solid consistence and undisturbed structure. For these curves the maximal value of shear resistance is adopted as the standard strength τ_f. The plots representing fissured specimens (B) describe plastic failure observed in preconsolidated stiff soil with structural disturbance. Such soils display a considerably lower strength referred to as residual strength τ_r.

The strength characterisation presented here was confined to the analysis of total stresses, because the soil mass had been dewatered and the differences in the values between c_u and c' as well as between φ_u and φ' were very small. Adequate estimation of the values of material parameters (i.e. those of the design parameters) is of prime importance to reliable stability evaluation. In our study, the values of the design parameters were established by linear regression with a confidence coefficient of 0.95 and by anticipating that

$$c_d = \frac{\bar{c} + c_{\min}}{2}, \qquad \varphi_d = \frac{\bar{\varphi} + \varphi_{\min}}{2}, \tag{2}$$

where $\bar{c}$ and $\bar{\varphi}$ are average values, and $c_{\min}$ and $\varphi_{\min}$ are minimal values of the parameters. But if the minimal values are smaller than zero, they should be adopted as equal to zero.

In order to assess numerically a decrease in the shear strength during the transition from the peak strength τ_f to the residual strength τ_r, use can be made of Bishop's brittleness index I_B, Haefeli's residual coefficient λ_R or Skempton's residual factor R. The values of I_B, λ_R and R are given by the following relations:

$$I_B = \frac{\tau_f - \tau_r}{\tau_f}, \quad \lambda_R = \frac{\tau_r}{\tau_f} = 1 - I_B, \quad R = \frac{\tau_f - \bar{\tau}}{\tau_f - \tau_r}, \tag{3}$$

where $\bar{\tau}$ is an average shear stress along the surface of potential failure. Index I_B has the advantage of expressing directly the maximal percentage of strength reduction which may result from the progressive failure of overconsolidated clay (figure 3).

The method of determining the soil shear strength of the bedding zone τ_{fc} is suggested by the Polish mining standard BN-82/0403-02. In this method, use is made of the formula:

$$\tau_{fc} = A\,[\sigma(\tan\varphi_1 + \tan\varphi_2) + (c_1 + c_2)], \tag{4}$$

where A is an empirical coefficient, and $\varphi_1, \varphi_2, c_1, c_2$ are the strength parameters of the contacting materials. As for the clay–brown coal contact, the BN-82/0403-02 standard recommends that the A-values of 0.30 be adopted.

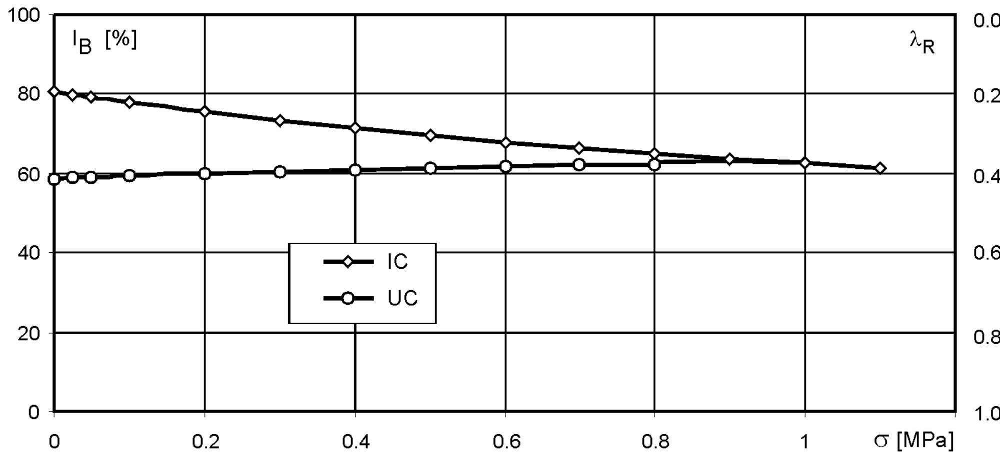

Fig. 3. Variation in the I_B and λ_R values related to normal stress

The contact zone under study is described in [5] which reports on laboratory tests where the strength of the brown coal–clay contact was examined in a direct shear apparatus. In order to reduce the influence of soil desiccation on the samples, they were collected during formation of the working slopes. During the tests the following parameters were determined:

- standard strength τ_f equivalent to the shear resistance for intact contact of brown coal–clay (*f*),
- residual strength τ_r occurring at the contacts characterized by disturbances due to tectonic pressure and decompression (*r*),
- strength after saturation of the slip surface occurring under conditions of both disturbed and saturated contacts (*s*).

In the contact zone, the clays in a direct contact with coal were predominantly in a stiff or firm state; those at a greater depth occurred in a hard state. Table 1 shows the average values of some major physical properties and strength parameters obtained by detailed identification [4], by tests on coal–clay contact zones [5], and by the back analysis of slope stability [6].

The results of laboratory tests have shown that:

- for clays with structural damage, both average and design shear resistances are smaller than the values obtained by back analysis;
- the brown coal–clay contact displays noticeably lower strength parameters than brown coal or clay;
- the structural damage in the area of instability hazard owes its origin to geological history and recent deformations.

Table 1

Properties of soil mass

Soil series		State of clay	Refs	Physical properties			Strength parameters			
							Average		Design	
				w	ρ	ρ_s	c	ϕ	c	ϕ
				[%]	[t/m^3]	[t/m^3]	[kPa]	[°]	[kPa]	[°]
IC		hard	[4]	32.7	1.86	2.69	332	12.1	243	6.0
IC_r				34.0	1.97	2.69	64	8.0	50	6.3
UC				28.2	1.96	2.67	281	9.5	223	4.7
UC_r				28.1	1.92	2.67	71	7.5	61	5.2
BC_r				94.2	1.30	1.63			0	18.0
Coal–clay contact	*f*	hard	[5]	30.2			116	22.3	74	16.1
		stiff		38.4			21	16.7	18	10.3
		firm		49.7			20	13.7	15	9.8
	r	hard					22	19.8	15	14.9
		stiff					13	11.0	12	8.0
		firm					16	9.5	10	6.2
	s	hard					22	16	11	10.5
		stiff					7	10.2	4	7.1
		firm					13	6.5	6	5.3
IC			[6]		1.93				69	14
UC					1.95				80	15.3
WB				94.2	1.30	1.63	68	11.5	68	11.5

4. NUMERICAL ANALYSIS OF SLOPE STABILITY

In the analysis of stability, it is essential to adopt an appropriate scheme of calculations that would include the geological structure of the soil mass, which is generally inhomogeneous and shows structural disturbance. In our study, use was made of slide lines as depicted in figure 4. Two potential slip surfaces were considered (denoted as 1–2–3 and 1–2–4–5–6, respectively). The shear resistance on the contact surfaces of the two layers were adopted as for clay [4] or for the coal–clay contact [5]. Our analysis included the results obtained for undisturbed and disturbed specimens. The disturbed specimens were assumed to display residual strength (table 1, index *r*).

The strength of the brown coal seams, which was equivalent to the intact conditions, did not indicate any stability failure hazard whatsoever. That is why the strength of brown coal was described either in terms of the strength parameters established by back analysis of failure [6], or in terms of the parameters assigned to residual strength [4]. Calculations were carried out for average and design strength parameters. Numerical analysis of slope stability was performed using our own programme, referred to as FILAR, which is based on the slightly modified Janbu method [3]. Thus, with

FILAR, the iterative process automatically corrects the pressure line position, and it is possible to obtain a convergent solution.

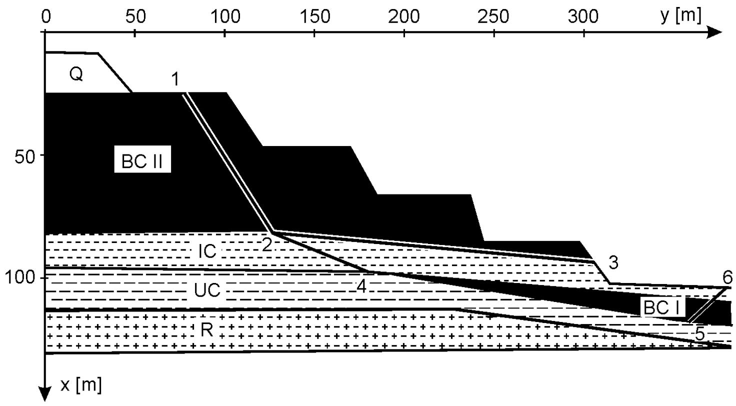

Fig. 4. Scheme of calculations

Table 2

Values of safety factor F

Slide line	Conditions at contact	Coal strength	State of clay	Strength parameters			
				Average	Design		
					f	r	s
1–2–3	2–3 IC [4]	c = 68 kPa ϕ = 11.5°	hard	5.84	4.21	1.60	
		c = 0 kPa ϕ = 18°	hard	5.71	3.91	1.25	
	2–3 coal–clay [5]	c = 68 kPa ϕ = 11.5°	hard	3.30	2.39	1.46	1.27
			stiff	1.79	1.35	1.13	0.97
			firm	1.58	1.28	0.99	0.88
		c = 0 kPa ϕ = 18°	hard	3.07	2.16	1.17	1.03
			stiff	1.56	1.10	0.88	0.72
			firm	1.34	1.03	0.74	0.62
1–2–4–5–6	4–5 UC [4]	c = 68 kPa ϕ = 11.5°	hard	3.67	2.73	1.24	
		c = 0 kPa ϕ = 18°	hard	3.51	2.58	1.08	
	4–5 coal–clay [5]	c = 68 kPa ϕ = 11.5°	hard	3.19	2.26	1.42	1.15
			stiff	2.18	1.54	1.02	0.91
			firm	2.01	1.49	0.92	0.84
		c = 0 kPa ϕ = 18°	hard	3.04	2.11	1.27	0.98
			stiff	2.02	1.38	0.86	0.75
			firm	1.84	1.33	0.75	0.68

The calculated results of stability analysis are listed in table 2. As shown by these data, the values of the safety factor differ considerably. Another major finding is that the hazard of stability loss will be detected only if we take into account the strength parameters of the coal–clay contact and if the clay has a liquidity index higher than zero.

In stability predictions for overconsolidated soils, errors are propagated due to the neglect of the inhomogeneous effort of the soil in potential slip surfaces [2]. The indicator of the material effort E is defined as

$$E = \frac{1}{F} = \frac{\tau}{\tau_f}, \tag{5}$$

where τ is the shear stress on the slip surface, and τ_f is the shear strength.

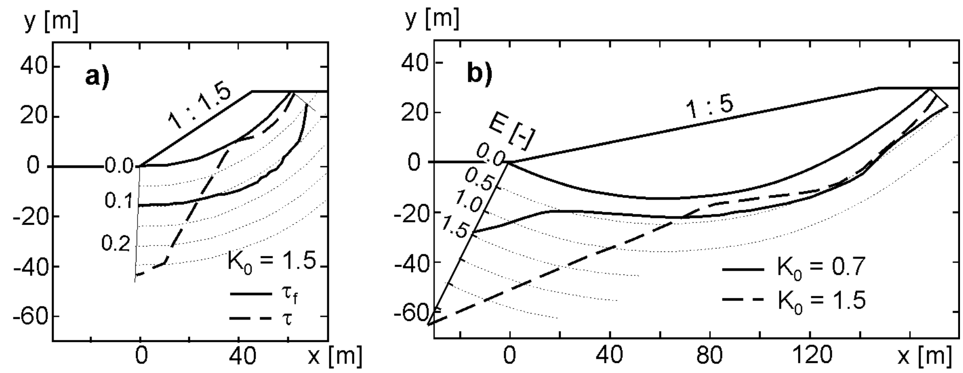

Fig. 5. Stresses and material effort on the slip surface

The plots in figure 5 illustrate the material effort during unloading. The distribution of the material effort on the slip surface is notably nonhomogeneous and increases with an increase in overconsolidation. If the effort distribution is known, we can assess the zones where standard strength τ_f is likely to decrease to the level of residual strength τ_r.

5. CONCLUSION

The results of laboratory tests and numerical calculations enabled the following conclusions to be drawn:

• In overconsolidated cohesive deposits with structural damage, even average shear resistance takes smaller values than those obtained in back analysis of slope stability.

• In openpit mining, the prediction of slope stability is influenced by the structural disturbance in the soil series and weakness of the contact layers.

• The contacts of the brown coal seam with the underlying clay series are the main contributors to the landslide hazard.

• Overconsolidated clays are primarily sensitive to moisture variations and decompression.

• The failure of overconsolidated Tertiary clays in the hard state is of a progressive character.

• The recognition of the strength properties of both brown coal and contact zone is insufficient, even though their knowledge is of paramount importance to stability analysis.

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