Heavy preloading and consolidation of soft soil masses with the use of reinforced embankment and vertical drains

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ABSTRACT: A large part of field "B" in the open cast coal mine "Kolubara", in the Republic of Serbia, was covered by soil masses slid from the disposal area to the active coal excavation area in March-April 2006. A landslide with a length of approximately 2 km and the width of more than 400 m stopped the coal excavation in the field "B". The already exposed coal stratum was covered by 30 m – 40 m thick very soft soil deposits. An improvement of soft soil deposits at the front part of the field "B" and subsequent open excavation to the roof of the covered coal layer seemed to be the best solution. A heavy pre-loading consisting of a 9.5 m high embankment and an accelerated consolidation by the use of vertical drains was selected. A high tensile strength geosynthetic reinforcement placed at the base of the embankment allowed for a shorter construction time. The pre-loading stage was monitored. After 8 months of consolidation 25 samples were taken away from different locations and depths and a reasonably improvement of the shear strength could be confirmed. Due to the performed consolidation the preload could be removed and a sloped excavation to the coal layer started. After completion of the 1:6 inclined slope down to the coal layer, the coal exploitation re-commenced at the start of the End of 2008. To date (July 2009) more than 1.2 Million. tons of coal have been excavated and exploitation is continuing without instability problems. The paper describes the design procedure as well as execution details and monitoring results.

1 INTRODUCTION

Brown Coal Mine Kolubara is located in the west part of Serbia, ca. 80 km from Belgrade. The coal exploitation in the field "B" started in 1951. From 1951 and up to March 2006 approx. 75 Million tons of brown coal were excavated. Simultaneously, more than 176 Million cubic meters of removing cover soil were deposited behind the steadily moved front of the excavation area. In spring 2006 the first signs of a landslide process could be observed. Due to the very wet winter and spring an acceleration of the movement of deposited soft soil fill was noticed. In March 2006 the velocity of the landslide was estimated with 2 m/day. The movement was observed on the length of 2 km and on the width of more than 450 m. Practically 80 Million m³ moved slowly towards the front of the coal excavation. Due to this extreme landslip risk in April 2006 the whole equipment was removed and the exploitation of coal in the field "B" was stopped. The whole area of the recently opened coal field was completely covered by soft soil masses. A part of the landslide overtopped the unexcavated coal in a form of a tongue with the thickness of 12 m - 15 m. This situation is schematically presented in cross-section B-B', in Figure 1.

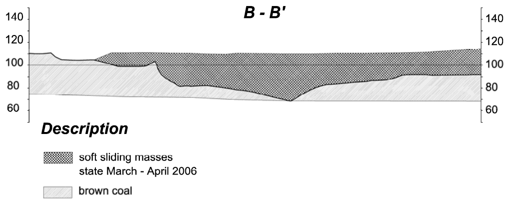

Fig. 1 Cross-section B-B', March-April 2006, state after of the landslide of soft soil masses

Thick soft soil masses (30 m - 40 m) covered the bottom of the excavation and after overtopping of the coal roof their movement slowed (a quasi limit equilibrium was reached). After October 2006 creep and a gradual movement at the front of the landslide were further observed. The overtopped cover soil showed signs of cracks fissures and was very uneven. On the surface of the landslide many small lakes or ponds formed. This infiltration water further destabilized the front slope of the landslide. In addition to surface observations of the landslide a few boreholes were undertaken to remove soil samples for the first laboratory investigations in October 2006. It was clear, that without knowledge of properties of sliding masses no remedial solution for a reopening could be developed.

2 PROPERTIES OF SOFT SOIL MASSES

An investigation of soil samples showed a very low shear strength and a low water permeability. With a clay fraction 30-50% the disposed material was essentially a heavy clay with an addition of sand, silt and only sporadic gravel. The residual angle of internal friction was estimated with phi' = 8°. In the remolded and quasi full water saturated state no cohesion could be identified for the sliding masses. Taking into account a full water saturation the theoretical stable slope angle of an excavation in sliding masses was estimated to be not greater than 3° - 4°, i. e. a safe slope should have an inclination not steeper than 1:15 – 1:20. From practical point of view, an excavation without ground improvement would be endangered by the existence of creep and further activity of inertial forces, which could be activated or accelerated after flattening of the landslide tongue. An additional restriction to ground improvement was the very low permeability of the soil, which was estimated with $k = 8.7 \times 10^{-11}$ m/s. The characteristic value of the oedometric modulus for the stress range of σ = 0-500 kN/m² was equal to $Mo_k = 5.07$ MPa and of the unit weight was assumed with $\gamma_k = 17.87$ kN/m³. For the design purposes shear tests on consolidated as well on consolidated and preloaded samples were conducted taking into account the suggestions given in Bjerrum (1973). The increase of shear strength due to consolidation stress σ = 0-500 kN/m² or preloading with $\Delta\sigma$ = 200 kN/m² can be seen in table 1.

Table 1. Shear parameters of normally consolidated and consolidated + preloaded samples of soft soil masses, $\Delta\sigma$ = 200 kN/m²

Type of test	Friction angle [°]	Cohesion [kN/m²]
Normally consolidated	12 15	10 8
Preloaded	16	12
Untreated soft soil	8	-

3 DESIGN OF SOIL IMPROVEMENT

It was assumed, that soft soil masses will be improved by pre-loading with a working platform H = 2.0 m and embankment h = 9.5 m. The pre-loading stress for such height 2.0 + 9.5 = 11.5 m was estimated with $\Delta\sigma$ = 200 kN/m². The preload area at the base of working platform had a length of 378 m and the width of 116 m. The embankment had at the base 360 m x 88 m. The stability and settlement analysis were performed in four cross-sections (A-A'…D-D', see table 2), which were quasi symmetrically located along the base of working platform. The analyzed cross-sections served as a reference for the monitoring during the construction works, too. The first step of the design was concentrated on consolidation and settlement analyses for a planned schedule of earth works. It was assumed, that the preload will be realized by: working platform H = 2.0 m: in one month, embankment: h = 0 – 9.5 m in two months, consolidation under the full embankment height: 3 months, removal of the embankment and working platform: 2 months. For the acceleration of consolidation prefabricated vertical drains (PVD) with the discharge capacity of $q_w = 5.5 \cdot 10^{-6}$ m³/s in triangle spacing of 1.25 m were chosen. The design of vertical drains based on recommendations given in Hansbo (1994). Due to a low water permeability of sand used for the working platform ($k = 2{,}2 \cdot 10^{-5}$ m/s), horizontally installed PVD's in a spacing of 1.0 m and drainage pipes (diameter 100 mm) in a spacing of 10 m were placed on the base of working platform, additionally. This combined dewatering system of the working platform (2,0 m sand, horizontally installed PVD's and perforated pipes) had a sufficient drainage capacity and transmissivity. The performed consolidation analysis showed out, that the excess of pore pressure can reach the max. value of Δp = 110 kN/m² as the embankment reaches the max. height of 9.5 m. The maximum settlement was estimated as s_{max} = 1.14 m at the centre. It was assumed that soft soil will be consolidated under own weight (normal consolidation) due to a relative large time needed for the installation of PVD's. Hence the stability analysis of the embankment were conducted with the pore over-pressure Δp_o = 110 kN/m² and soil parameters for the normally consolidated state, as per table 1. Generally, a slip circle method described by Bishop (1955) was used for the examination of stability

Table 2. Prognosed values of safety factors for the critical stage of embankment

Section		**Drain spacing 1.25m**	
		soft soil parameters φ'=15°; c'=8kN/m²	soft soil parameters φ' = 12°; c'=10kN/m²
A-A'	left	1.74	1.66
	right	1.31	**1.19**
B-B'	left	**1.19**	**1,05**
	right	1.42	1.29
C-C'	left	1.42	1.36
	right	1.44	1.33
D-D'	left	**1.19**	**1.13**
	right	1.38	1.24

A high tensile strength woven fabric made from polyester with ultimate tensile strength UTS = 1.600 kN/m was designed as a basal reinforcement. Due to the short time of loading up to 1 year , the design tensile strength of this geotextile was estimated as F_d = 830 kN/m. The collection of the achieved results for all sections is presented in table 2. The most critical location was the cross-section B-B' with the safety factor η = 1.05, which was lower than the value of 1.30 required in DIN 4084 for construction works, see Figure 2.

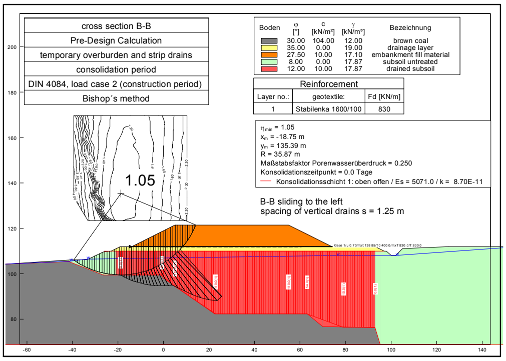

Fig. 2 Cross-section B-B', Stability analysis of embankment, Bishop's method

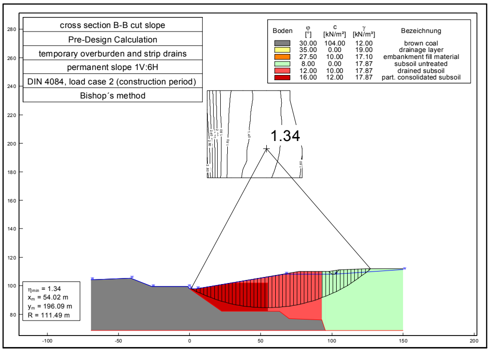

Fig. 3 Cross-section B-B', Stability analysis of excavated slope after pre-loading, Bishiop's method

The next stage of design was to estimate the factor of safety during the sloped excavation in the improved area to the roof of the coal stratum. According to the laboratory investigations the stability analysis was performed with the early predicted shear parameters. Figure 3 presents schematically a cross-section of excavated slope under an inclination of 1:6. The results of the stability analysis for the excavated slope are collected in the table 3. The min. value of safety factor equal to 1.34 were estimated in the cross-section A-A' and B-B'. This value could be accepted for the short time needed for the coal excavation ca. 2 years

Table 3. Predicted values of safety factors after the excavation to the roof of coal stratum

Section	**Drain spacing 1.25 m**	
	soft soil parameters φ'= 15°; c' = 8kN/m²	soft soil parameters φ' = 12°; c' = 10kN/m²
A-A'	1.38	1.34
B-B'	1.40	1.34
C-C'	1.49	1.46
D-D'	1.55	1.54

-in the pre-load zone: φ' = 16°; c' =12kN/m².

4 MONITORING OF CONSTRUCTION WORKS AND SOIL IMPROVEMENT

Due to the low predicted safety of embankment a monitoring system was installed, consisting of: pore pressure devices, hydrostatic piezometers, extensometers, settlement plates installed on the geotextile reinforcement and markers placed on the ground surface for GPS measurements (control of settlements and horizontal movements in the vicinity of working platform) were installed. Typical location of the monitoring devices is shown in Figure 4, i.e. the cross-section B-B' with the lowest predicted stability reserve, η = 1.05.

The embankment was started after the installation of vertical drains and of the geotextile reinforcement in October 2007. The installation of the geotextile reinforcement at the embankment base is shown exemplary in Figure 5. Generally, the area to be loaded and the vicinity of the working platform was very unstable showing cracks and steps. In spite of these difficulties the construction works were continued.

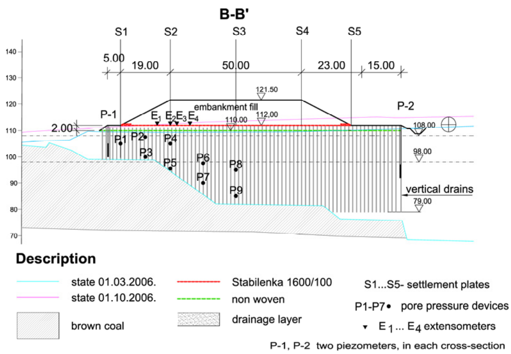

Fig. 4 Cross-section B-B', monitoring plan

In January-March 2008 due to a delay an acceleration of construction works was necessary. This provoked a higher increase in the excess pore water pressure and a rapid increase of the elongation of the geotextlie reinforcement.

Fig. 5. Installation of geotextile reinforcement on the top of working platform, UTS = 1600 kN/m

In April 2008 a rupture at the location of the extensometer E-4 was noticed and a rotational sliding of the front part of the embankment observed, see Figure 6.

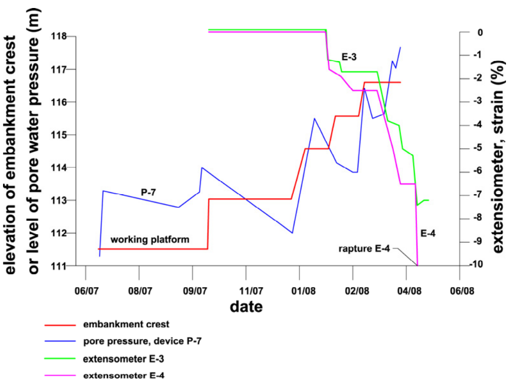

Fig. 6. Monitoring data, cross-section B-B', July 2007 -April 2008

The slip plane was relatively flat due to the partial extrusion of soft soil at the front of the embankment. It was later confirmed that the flat sliding plane did not affect the improved area, which has an importance for the further excavation. After a removal of 25 samples from 12 different located boreholes laboratory tests were performed in the summer 2008, the results showed out a remerkable improvement of the shear strength after pre-loading. The characteristic values of the main parameters were estimated for the confidence level of 95%. Based on 25 individual results the following values were finally estimated:

- angle of friction: $\varphi'_k = 16.2°$
- cohesion: $c'_k = 11.6$ kN/m²
- unit weight: $\gamma_k = 17.7$ kN/m³.

In the unloaded area i.e. under the working platform besides embankment, the predicted values for normally consolidated soil given in the table 1 were confirmed by these shear tests, too. It could be stated, that a very good agreement between the predicted design values and the estimated values after pre-loading was achieved.

In October 2008 the excavation to the coal layer was finished, (see Figure 7) and 2 inclinometers were installed in each of four cross-sections in the central part of slope. A renewed excavation of coal started in November 2008. Initially the slope showed a horizontal movement with a velocity 5 - 20 mm/day. In January 2009 the velocity of the horizontal movement reduced into the range of 0.3 – 0.6 mm/day. Due to the stable situation the monitoring was stopped in February 2009. Up to this day 1.2 Million tons coal were extracted and the excavated slope in the improved area presents a sufficient structural integrity.

Fig. 7 Excavated slope in the improved soft soil masses, slope inclination 1:6, in the front part uncovered roof of coal layer

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