Landslide induced by tunnel excavation

Glissement du terrain causé par le creusement d'un tunnel

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ABSTRACT: The formation of a deep slide induced by the excavation of a highway tunnel in a slope is described and analyzed. The slope is composed of Jurassic carbonate rocks above a subvertical fault, and of Paleozoic clastic rocks beneath the fault. The analyses of surveying data on the tunnel construction site and of geotechnical test results showed that a progressive slope failure was initiated. Regions of plastification and disturbance of the rock mass around the tunnel were determined by numerical modeling of the tunnel excavation using the computer program FLAC. Three-dimensional limit equilibrium stability analysis, taking into account the decrease of shear strength due to disturbance, gave the contours of a possible sliding body. Based on these analyses it was concluded that any continuation of excavation works in the slope might induce a general slope failure.

RESUME: Les auteurs décrivent et analysent le procès de formation d'un glissement du terrain causé par le creusement d'un tunnel routier à travers une pente. La pente est composée des formations carboniques jurassiennes au-dessus d'une faille subverticale et des formations clastiques paléozoiques au-dessous de cette faille. Les résultats des observations topographiques dans le tunnel et à la surface du terrain et des reconnaissances géotechniques ont montré qu'il s'agit du commencement d'une fracture progressive de cette pente. Les zones de plastification et de disturbation du matériel ont été définies par simulation numérique à l'aide du programme FLAC. Les contours de la probable forme finale du corps glissant ont été déterminés par analyse tridimensionnelle en utilisant la méthode d'équilibre limite et en tenant compte de la réduction de résistance du matériel causée par la disturbation. Il a été conclu que chaque nouvelle excavation pourrait provoquer la fracture générale de la pente.

1 INTRODUCTION

The highway connecting Rijeka and Zagreb in Croatia traverses a slope at its thirty-first kilometer. This slope is composed of Jurassic carbonate rocks above a subvertical fault, and of Paleozoic clastic rocks beneath the fault. The highway and the fault cross at. In the clastite region of the slope the retaining structure of maximum height 12m was to be constructed along the highway in the length of 265m. A tunnel was planned for the last 70m of this region. After intersecting the fault, the tunnel was to pass through carbonate rocks. Figure 1 shows the geological map of the area.

The tunnel, with the diameter of about 10m, was excavated and the opening was stabilized in the region of carbonate rocks. But, there was a collapse of the tunnel overburden rock mass, 30m high, when the tunnel was excavated at the contact of carbonate and clastite. This collapse progressed up to the surface of the slope, forming a sink-hole. When the area of collapse was restored, the upper two thirds of the tunnel were excavated in the whole length of the clastite rocks region, as the first phase of the tunnel construction.

An open-cut excavation, about 25m high, with the slope inclination 1:1.5, was made down to the highway level in the region where the retaining structure was to be constructed, except for the last 50m in the vicinity of the tunnel, where the excavation was made only to the level of the retaining structure top. A landslide, with the body about 100m wide, up to 15m deep, was subsequently formed in the deeper part of the open-cut excavation. Cracks occurred first, at the head of the sliding mass, and the main body closed up at its flanks 6 months later, forming definite contours of the landslide, when the slope was further excavated for the wall. The progression of the sliding mass, which reached 10cm at the slide toe, and 100cm at the head, was temporarily prevented by banking up material at the slide toe. The highway construction was consequently stopped for two years.

There was yet another collapse of the tunnel, up to the surface of the slope, when the construction was resumed by excavating the lower third of the tunnel in clastite layers from the contact of carbonate and clastite towards the portal. There was also a crack along the tunnel, at the contact with the fault. By inspecting the tunnel it was determined that a 20cm displacement of the tunnel occurred down the slope.

The present geotechnical analysis was initiated at this stage. In situ investigations (engineering geology mapping, borehole drilling and pressuremeter testing) and laboratory tests were carried out along with slope stability analyses and numerical modeling of the tunnel excavation.

2 STATEMENT OF THE PROBLEM

2.1 *The tunnel*

The stability of the first phase tunnel excavation in the clastite region, i.e. from the collapse site to the tunnel head in the portal region, was estimated by surveying the tunnel cross sections at intervals of 5m, using the AMT 2000 Wild profiler. Visual inspection of the constructed support systems was also made.

The surveying data were compared to those made two years earlier. It was obvious that all tunnel cross sections had moved down the slope. Even though it is not possible to determine the exact displacement vectors, the horizontal displacements can be estimated to 10-25 cm. It was, thus, concluded that the rock mass surrounding the tunnel is quite unstable, which was also confirmed by additional monitoring.

The inspection of the tunnel support systems showed that they were damaged at several locations. There were cracks in the shotcrete, and the anchor faceplates were deformed. The 60cm thick support systems were laterally 5-50cm within the tunnel clearance at every cross section, and the crown support was 10-40cm within the clearance at two cross sections.

Figure 1. Geological map of the area

The overall conclusion was that, due to the instability of the rock mass, all the tunnel support systems constructed during the first phase should be removed along the whole tunnel in this region, and that the excavation should be stabilized once again. The removal of heavy support systems (60cm of shotcrete, 3 steel wire meshes, steel sets, anchors) in unstable and very disturbed rock mass required special operations, which include the construction of a plate above the tunnel crown and the stabilization of the underground excavation under the plate protection, with the condition that the slope is first stabilized.

2.2 *The slope*

The slope stability was estimated based on the measured displacements of points on the slope surface, by the tunnel lining position, and by the visual inspection of cracks. Figure 2 shows the geological map of the landslide area, and Figure 3 shows the characteristic tunnel cross section.

There were 10cm wide cracks in the region from the portal to the carbonate and clastite contact, about 70m long, formed along the fault contact. These cracks appeared at the time when the second collapse of the tunnel occurred. They indicated a general instability of the slope, so that any additional excavation work might induce a landslide.

A similar situation appeared in the region where the retaining construction was to be built. Tension cracks were the first sign that a progressive slope failure was occurring, which became apparent as soon as the first excavation works for the wall were made. The analysis of this landslide was made based on surveying data. Three measurements in two years gave the rate of displacements in the region where the tunnel passes through clastite layers of 0.11-0.18mm/day. This lead to the conclusion that a general slope failure had not occurred yet.

3 DETERMINATION OF GEOTECHNICAL PARAMETERS

The slope stability analyses required the determination of the clastite shear strength in accordance with the Hoek and Brown (1980, 1988) empirical strength criterion.

The clastite rock mass consists of black to gray clayey shale, and gray-brown quartz sandstone in consecutive layers. The sandstone layers (from a few centimeters to 30cm thick) are 8cm thick on the average. The clayey shale layers are predominant, and they come in packages 0.5-3m thick. The package sublayers are 2-36cm thick, 13cm on the average.

The behavior of clastite is, thus, governed by clayey shale. Since it was not possible to obtain undisturbed samples of clayey shale for laboratory testing by drilling, they were obtained by manual extraction from the excavated blocks. The rock mass compressive strength was determined in uniaxial and triaxial tests,

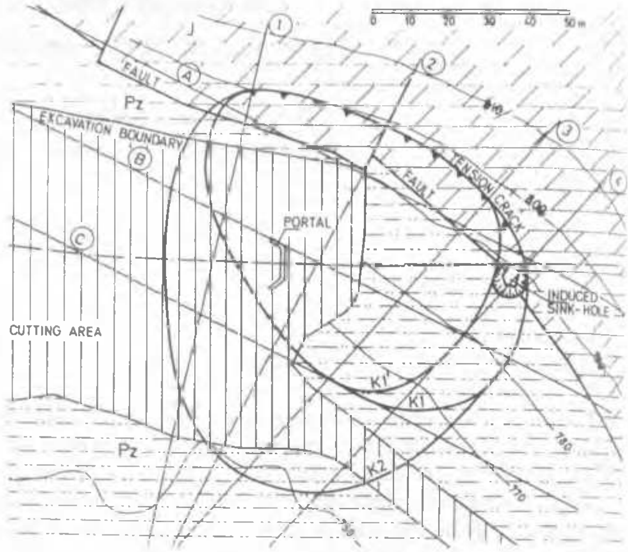

Figure 2. Geological map of the landslide area

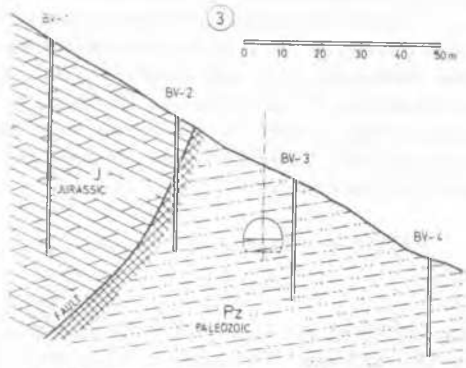

Figure 3. Characteristic cross section

and the shear strength in direct shear tests on such samples. The direct shear tests were also performed on artificially prepared samples of different grain size distribution.

Based on the laboratory tests, the adopted parameters for the Hoek and Brown strength criterion are as follows.

- for the disturbed rock: m=1.6768, s=0.01550, σ_c=300 kPa

- for the undisturbed rock: m=4.0948, s=0.06218, σ_c=300 kPa

where σ_c is the axial compressive strength.

Figure 4 shows the two curves, the upper for the undisturbed (N), and the lower for the disturbed (P) rock mass. It can be seen from Figure 4 that the results of shear tests fit well between the two adopted curves.

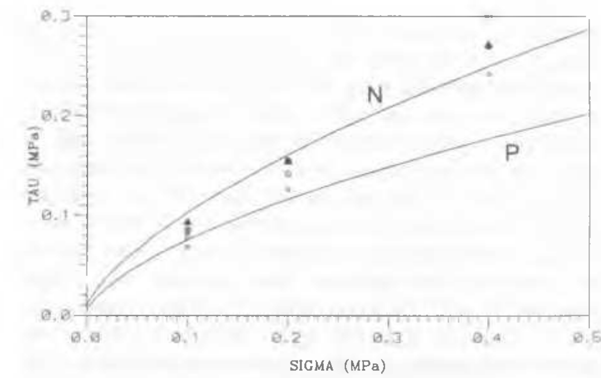

Figure 4. Drained shear strength of clastite

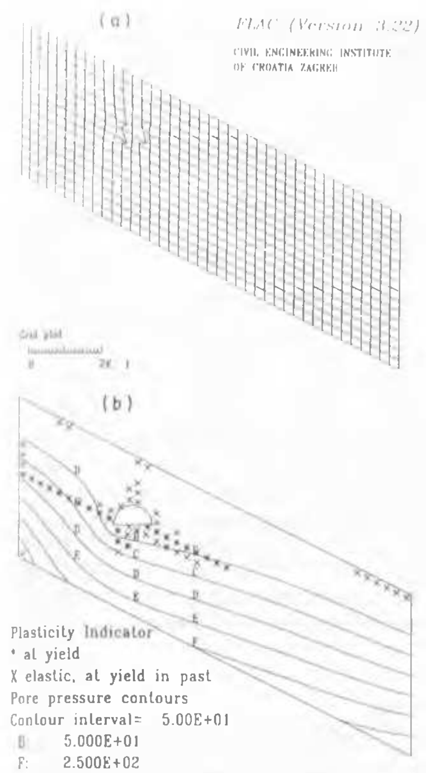

Figure 5. Numerical modeling: (a) finite elements mesh; (b) water pressures and plastification after tunnel excavation

4 NUMERICAL MODELING OF THE TUNNEL

The computer program FLAC (1991) was used for the numerical analysis of the tunnel excavation as a two-dimensional model. The region around the tunnel was set as an infinite slope, and so were defined the initial stresses.

The dimensions of the model are 210m x 40m. The lateral boundaries, which are 100m away from the tunnel, are vertical, whereas the upper and the lower boundaries are at $26{,}6^{\circ}$ to the horizontal. The finite elements mesh is presented in Figure 5a. The influence of finite elements dimensions and of the model boundaries on the initial stresses was analyzed, and the results showed that the model was well defined.

Two values of the axial compressive strength were used. The value of 300 kPa was used for the upper 20m of the model, and the value of 1000 kPa was used for the lower 20m, where the clastite is mostly composed of sandstone.

The slope stability after the tunnel excavation was analyzed for the case of dry rock mass, and for the case of water flow parallel to the slope, with the water lever 5m beneath the slope surface. The results for the second case are shown in Figure 5b. It can be seen that plastification occurred in the region around the tunnel, which indicates that the rock mass is unstable.

5 SLOPE STABILITY ANALYSIS

As the thickness of the overburden rock mass above the tunnel varies considerably from one cross section to another, it was necessary to carry out a three-dimensional slope stability analysis.

5.1 *Three-dimensional analysis*

At present there is no reliable generally accepted method for three-dimensional slope stability computations. Stanić and Mihalinec (1991) have analyzed the existing 3D methods and concluded that for drained slope failures it is more adequate to use complete 2D solutions and take into account the contribution of end effects, rather than completing an incomplete solutions.

The two-dimensional slope stability analysis was carried out using the Spencer method (1967), based on the limit equilibrium, and incorporated into the computer program SBHOEK (Stanić and Kovačević, 1995). This is a version of the program SSTAB (Wright, 1974), modified in order to allow the use of the Hoek and Brown empirical strength criterion.

Mihalinec and Stanić (1991) have presented a 3D solution for the analysis of slope slides of a given shape. The sliding body is divided into elements, each of which is defined by a representative cross section.

It is assumed that the mobilized shear forces between the adjacent elements act together with the resisting forces on the slip surfaces, and that the equilibrium is established on the whole sliding body, which is defined by the 3D safety factor FS_3.

In order to use the proposed 3D method of slope stability analysis, which is completely described by Stanić and Nonveiller (1996), the shear strength at the contacts between the elements has to be defined. The method proposed by Anderson (1985) was adopted for this purpose.

5.2 *Three-dimensional computations of the landslide*

The main purpose of 3D computations of the landslide induced by the tunnel excavation is to establish the validity of input data (pore pressures, shear strength) and assumed cause of sliding, by the back analysis. The following analyses were carried out:

1. For the slope prior to excavation works
2. For the slope after the tunnel and the surface excavation works

The value of 2.2 g/cm^3 was used for the clastite density, and the pore pressures are defined by the pore pressure coefficient $ru = 0.3$.

The 2D slope stability computations were carried out for the representative cross sections 1-4 (Figure 6), whereas the 3D effect was analyzed for 3 possible sliding bodies. The following notations are used for the analyses:

T1 - slope prior to excavation works
T2 - slope with the landslide forming
K1 and K1' - assumed shallow landslides
K2 - assumed deep landslide
N - strength parameters for the undisturbed rock mass
P - strength parameters for the disturbed rock mass

After analyzing the slope stability for cross sections 1-4 in the sliding direction, cross sections A, B and C (Figure 7) perpendicular to the sliding direction were analyzed.

The analysis has shown that shallow slip surfaces K1 can close up into two sliding bodies K1 and K1', as in cross sections A and B. The deep slip surface closes up into the sliding body K2, as in cross sections A, B and C.

3D slope stability analyses were carried out for the defined sliding bodies. The results are presented in Table 1. 3D factors of safety for the undisturbed (N), and the disturbed (P) rock mass are given in Table 2.

6 CONCLUSIONS

1. The regions of rock mass disturbance due to the tunnel excavation, estimated by numerical modeling, indicate that large parts of possible sliding bodies K1 and K1' are within these regions. The lower part of the possible sliding body K2 is beyond the influence of the tunnel excavation.

2. The actual state of shallow landslides K1 and K1' is close to the general failure, i.e. the actual factors of safety are closer to those for the disturbed rock mass (1.002-1.100), than to those for

the undisturbed rock mass (1.384-1.472).

3. The actual state of deep landslide K2 is between the states for the undisturbed and the disturbed rock mass, with the factor of safety from 1.375 to 0.997.

4. Any further excavation works would lead to the general slope failure.

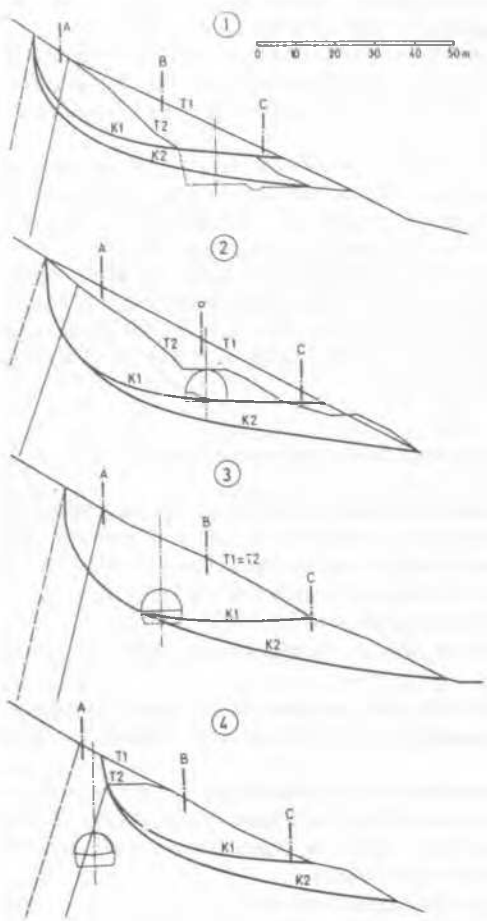

Figure 6. Cross sections 1-4 in the sliding direction

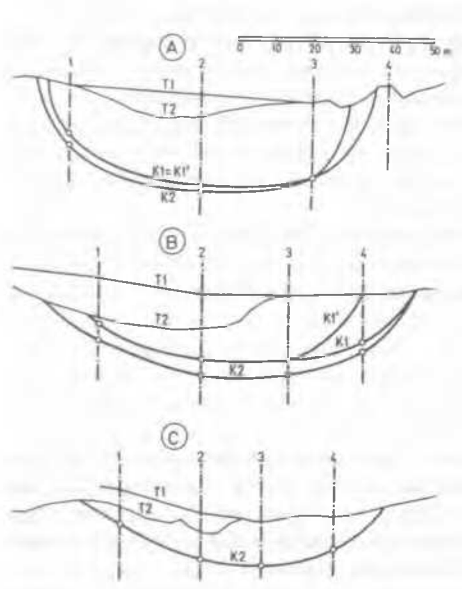

Figure 7. Cross sections A-C perpendicular to the sliding direction

Table 1. 3D stability computations for K1,T2,N

A. Data on the elements

n	F_j	θ_j (deg)	W_j(KN)
1	1.102	30.	109 852.9
2	1.345	11.	326 043.9
3	1.565	-5.	611 754.0
4	1.809	-30.	154 491.1

B. Data on the contacts of elements

k (No)	l_k=l_j+l_{j+1} (m)	$h_v(k)$ (m)	ϕ_k (deg)	H_k (kN)	$Tf(k)_k$ (kN)
1	15.5	5.46	30.0	61 048.4	55 3922.7
2	22.5	7.06	30.0	182 487.9	135 628.7
3	31.0	9.71	30.0	325 237.4	207 835.2
4	17.5	5.01	30.0	98.050.2	54 206.9
				FS_3=1.472	

Table 2. 3D factor of safety for K1, K1' and K2

	K1	K1'	K2
N	1.472	1.384	1.375
P	1.100	1.002	0.997

where:

n - number of element

F_j - safety factor of j-th element

θ_j - average transversal slope at the slide plane of the j-th element

W_j - weight of the j-th element

k - number of contact between elements

lj - length of the j-th element

l_k - length of the k-th contact

$h_v(k)$ - height of water on the k-th contact

ϕ_k - effective friction angle on the vertical plane of the k-th contact between elements

H_k - total horizontal force on k-th contact between elements

$Tf(k)_k$ - shear strength on the k-th contact between elements

FS_3 - 3D safety factor of the considered sliding body

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