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Failure mechanism following peak strength in Carboniferous-Permian soft rock

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This paper was prepared for presentation in San Francisco in 2008 at the 42nd US Rock Mechanics Symposium and 2nd U.S.-Canada Rock Mechanics Symposium, held in San Francisco on June 29-July 2, 2008.

ABSTRACT: Based on detailed geological-geotechnical investigations on the Blagovica-Kompolje motorway section (E-W corridor in Slovenia), the influence of time on the changes of the stress-strain state in the physical behavior of Carboniferous-Permian mixed soft rock mass was defined. Significant volumetric and distortion deformations appeared after a long period of rainfall after unloading of the excavation. Finally, gradual structural degradation caused sliding of the altered or disintegrated soft rock mass as this soft rock mass took on the characteristics of the saturated residual soft soil. Most important in this process were pre-existing weak soft rock zones. As a result of the suction process of pore water pressure dissipation out of the micro-cracks (dilatation suction and dissipation), viscous creeping developed. The presentation focuses on numerical back analyses which were made at three scales: mineralogical petrography specimens at micro-scale, at the scale of borehole samples and at the scale of land-sliding slopes.

1. INTRODUCTION

Based on a detailed geological-geotechnical investigation of the Blagovica-Kompolje motorway section, Carboniferous-Permian soft rocks were divided into three typical layers with different strength and deformability parameters. The purpose of the investigation and analysis was to determine the input parameters for numerical analysis which were needed to specify support and retaining measures for the excavation of cutting slopes.

Excavation for the foundation caused unloading and, along with heavy rainfall during autumn in 2002, activated a deep landslide in the building area of this motorway section. The landslide was successfully stabilized with an anchored pile wall, material exchange below the pile wall and a special drainage system.

With detailed laboratory analysis, “in situ” examination and monitoring results, we have gathered enough data for the creeping analysis of the landslide. The analysis was conducted using a special method of calculation enabled by the software used. During each calculation step, the grid is updated and active pore pressures are recalculated. Geometric nonlinearities, occurring in deformations of this extent, are thus avoided. With the slowing of creeping, conditions were established to begin a planned restoration using an anchored pile wall. Technical observation is ongoing while the structures are operational.

2. GEOTECHNICAL INVESTIGATIONS

Prior to the motorway construction, geological-geomechanical investigations were first conducted in the preliminary project phase and later in the main project phase. For those sections where the route was planned to cut into slopes, additional investigations were conducted. During the motorway construction, the back slopes of the planned supporting and retaining structures became accessible. For the main project, technical observation and monitoring were called for.

For determining rock mass characteristics, we engineering geologically mapped the slopes in detail during the excavations and catalogued bore holes, outcrops and piles. For that we used the new rock mass classification for mixed and soft rock mass by determination of the geological strength index GSI [1]. In selected bore holes, preasuremeter measurements (“in situ” measurements) were conducted and intact samples taken. During laboratory examination, we conducted triaxial consolidated undrained shear tests (“CU”-tests) of fifteen intact samples and nine compressibility and permeability oedometric tests.

3. ENGINEERING GEOLOGY CONDITIONS

The Blagovica-Kompolje motorway section (a small part of the Kiev-Barcelona Highway) is built upon and between side slopes of mixed Carboniferous-Permian soft rock [2]. In the landslide area, soft rocks are covered by a prolluvial layer of silt to clayey gravel of medium permeability. The upper layer of soft rock consists of a highly weathered disintegrated mixed soft rock mass, which in this particular section is composed mostly of slate siltstone with layers of fine-grained sandstone in nearly equal proportions. They are fractured and in some places fragmented (tectonically disturbed). Because of its specific composition, structure and texture, this rock mass layer has medium to low permeability and high compressibility. In this particular motorway section, it lies on a layer of moderately weathered weak (moderately disturbed by ancient landslides) mixed soft rock of the same composition and different thickness. With depth, the lower layer gradually becomes less fragmented and so becomes less permeable and medially compressible. Below that lies a layer of slightly weathered compact undisturbed mixed soft rock mass, mostly composed of slate siltstone in transition to clay shale. Fine-grained sandstone is only present in thinner sheets. This layer has very low permeability, is in places slightly tectonically damaged and has low compressibility. On the land sliding slope sections, we detected, in the transitions between these layers, a clay layer with fine gravel of highly disturbed rock mass, in some places with quartz veins. This layer prevents the transition of water from highly weathered and weak soft rock into the compact undisturbed rock mass. This is a shear zone created by ancient land sliding. Until slopes with similar engineering geology conditions are unloaded by excavation, landslides are rarely activated.

3.1. *Strength and deformability characteristics of the mixed soft rock mass*

The failure mechanism was first estimated by back analysis of the deformations after peak strength at microscopic scale of mineralogical petrography specimens taken from triaxial shear tests [3].

Strength characteristics were determined by the “CU” triaxial test.

The modulus of elasticity, determined by pressiometer testing, was used in the back analysis.

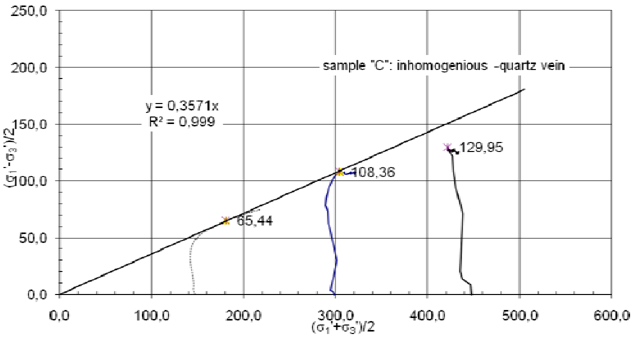

Fig. 1: Stress path of the triaxial shear test from which the mineralogical petrography specimen was ground. Sample “C” was inhomogeneous with quartz veins, although failure occurred faster than in samples “A” and “B”. Sampling was carried out at a 16.8-m depth with a diamond double core drilling sampler. Units: [kPa].

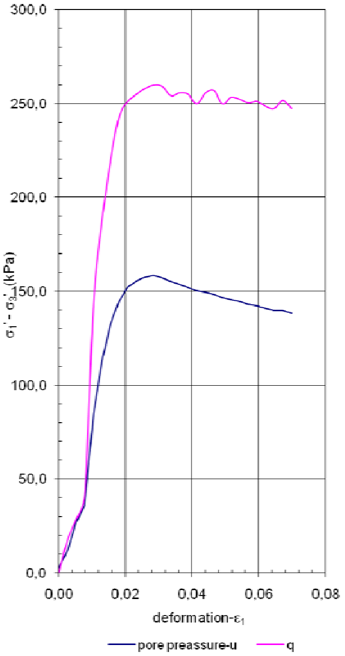

Fig. 2: Deviatory stress and pore pressure paths vs. deformation ε_1.

Table 1: Input parameters for back analysis at microscopic scale estimated by triaxial “CU” test and preasuremeter test at the same depth as the borehole specimen

Material	Silt stone lamina	Clay stone lamina with quartz veins
Loading modulus coefficient K_L	167.2	167.2
Bulk modulus coefficient K_B	55.87	55.87
Friction angel φ (°)	20.3	18
Cohesion C_{ref} (kPa)	2.9	0
Failure ratio R_f	0.97	0.36
Initial modulus of elasticity $E_{init.}$ (kPa)	75,000	75,000

3.2 Numerical back analysis of mineralogical petrography specimens at micro-scale

Problem: discretization in finite elements (meshing)

Quadrilateral finite elements with inter-medial nodes were used in 2D analysis.

Two materials were modelled with a nonlinear hyperbolic model: slate siltstone (brown material) and clay stone (dark brown to black material Fig. 3a).

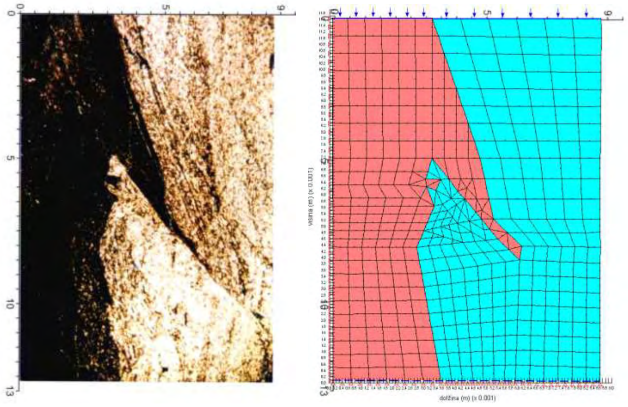

Fig. 3 a) left: mineralogical petrography specimen [4] (scale is in 10^{-3} m), b) right: discretization in finite elements by using the 2D-FEM program SIGMA/W (GEOSLOPE).

Failure mechanisms in silt stones were developed as brittle fractures that were followed by ductile deformations in clay stones. During the simulation of loading, higher pore pressures appeared in the "micro-cracks" of stiff parts of mixed soft rock than in the soft parts of it. Due to the high speed of crack propagation in the brittle fractures, shear deformations were greater than ductile deformations by a significant factor. In the main cracks, a tip-slide hyper elastic region developed and the numerous micro-cracks expanded faster than the main crack slide.

The stress energy included development of pore pressures. These increased at the contact with softer parts of the rock.

The pore pressures decreased with the distance of interfaces.

Microscopic deformations that appeared after the change of stress state were modeled with a nonlinear hyperbolic numerical model.

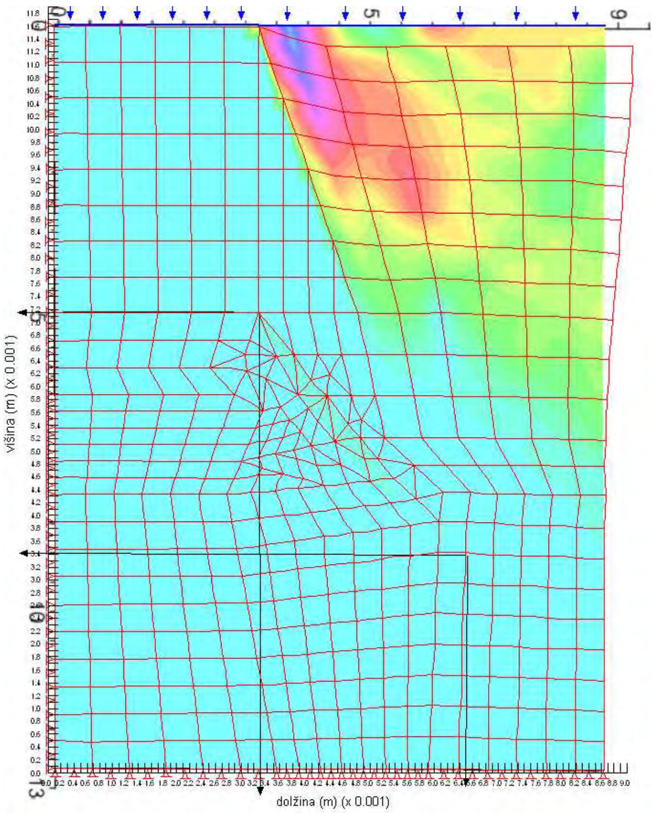

Fig. 4: Deformed mesh with maximum deformation of 1.29%.

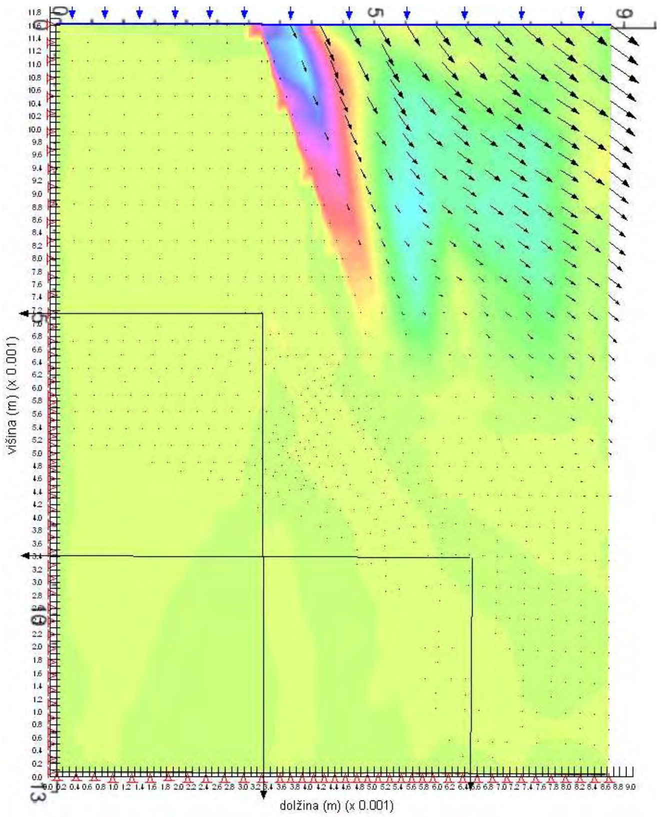

Fig. 5: Shear strains after 100 load steps in nonlinear analysis. The magnitude of the deformation vector is nearly the same as measured in the mineralogical petrography specimen.

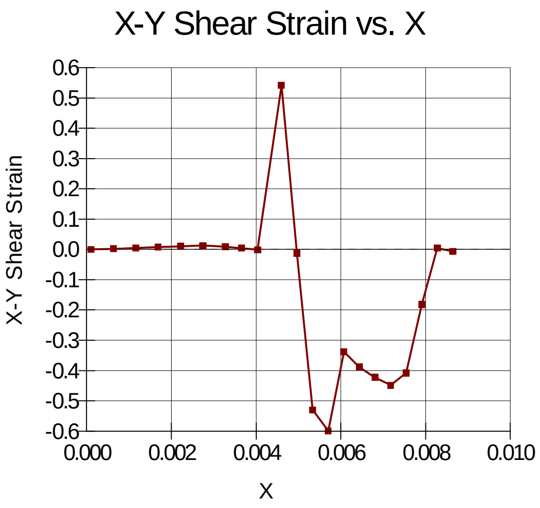

Fig. 6: Diagram of shear strain vs. longitudinal distance (horizontal scale is in m).

The results can be compared by molecular dynamics simulations [5], which show that a localized small hyper elastic region around the crack tip can have significant effects on the dynamics of crack propagation.

In contrast to the linear theory prediction, the authors found that the crack propagation velocity is about 20% greater in the stiffening system and 30% less in the softening system.

The small hyper elastic regions enhance the energy around the crack tip.

The higher crack velocity in the stiffening system and the lower velocity in the softening system are due to enhancement or reduction of the energy flow in the vicinity of the crack tip.

3.3 Numerical 2D / 3D analysis of the genesis of soft clay shale in the micro-folded rock

The structure, texture, mixed composition and level of tectonic disturbance, i.e. anisotropic characteristics of the rock mass, were determined by the Geological Strength Index (GSI). The simulation consisted of a large-scale triaxial test [6] using twelve pairs of effective stresses at failure in the triaxial shear apparatus of intact samples taken from the intact undisturbed mixed soft rock mass at a depth of 23 to 25 m.

The modulus of elasticity determined by the pressiometer test was used in the analysis.

The test was corrected by the GSI and $\mathbf{m_i = 16{,}914}$ values were determined during the excavation for anchored retaining walls. The rock mass disturbance factor $D = 0$ (marginal disturbances) was determined due to the excavation in mixed soft rock [7, 8].

The analysis was conducted by first establishing the initial stress conditions starting from an “in situ” stress state at a depth of 24.7 m. Horizontal stress was determined by empirical formulation (Zoback, 1994) [9]:

$$K = 0.25 + 7E_h\left(0.001 + \frac{1}{z}\right) \qquad (1)$$

The horizontal modulus was determined by pressure meter stress vs. deformation path.

The second step was to apply normal stress to the failure starting with the “in situ” stress state.

The material model for completely dry and compact average mixed Carboniferous-Permian soft rock of slate siltstone with clay stone layers was chosen as nonlinear hyperbolic in order to be comparable with the analysis at micro-scale [10].

Table 2: Parameters used in nonlinear hyperbolic analysis

Material	Silt stone with clay stone layers and quartz vein
Loading modulus coefficient K_L	4540
Bulk modulus coefficient K_B	4451
Friction angel φ (°)	29
Cohesion C_{ref} (kPa)	1
Failure ratio R_f	0.97
Initial modulus of elasticity E_{init} (kPa)	620,170

Fig. 7: Analyzed deformations in 2D compared with those measured in the mineralogical petrography specimen (scale is in 10^{-3} m).

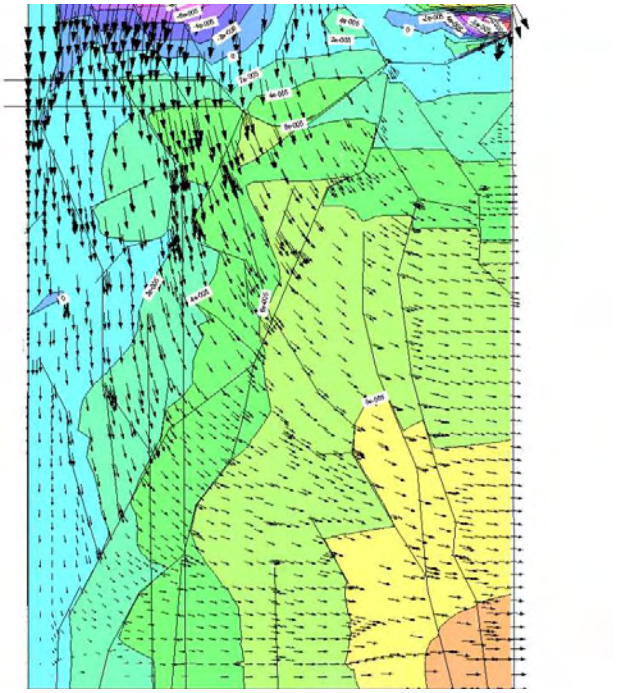

Fig. 8: Deformation after 200 load increments to final calculated historical "in situ stress" (Over consolidated ratio- "OCR" = 3.726). Analysis shows dilatants deformation softening in the Carboniferous-Permian mixed soft rock mass. The magnitude of the deformation vector is 3.5 x 10^{-3} m.

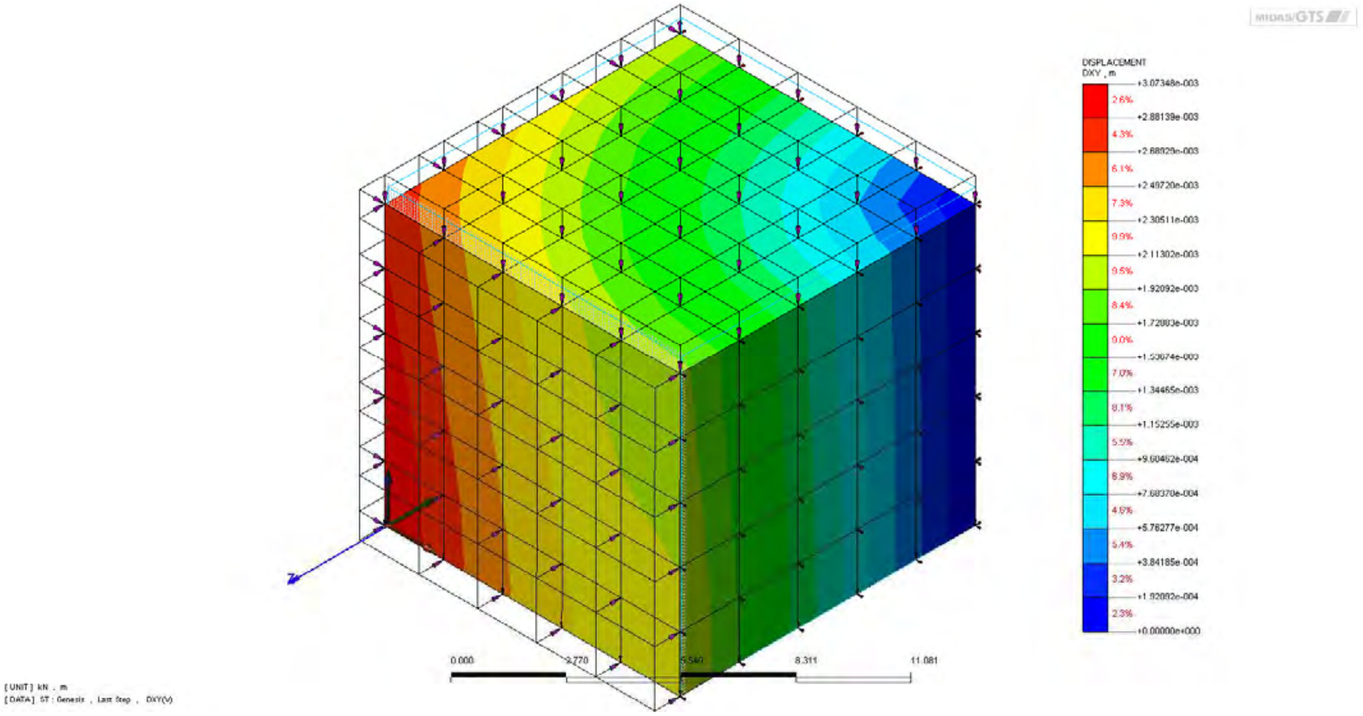

Fig. 9: Simulation of folding by historical "in situ" stress. The material model is modified Duncan and Chang [11] using the 3D-FEM program MIDAS/GTS (ver. 2.02, 2007). The magnitude of displacement is 3.07 x 10-3 m. It is suited very well to measurement of mineralogical petrography specimens.

4. OBSERVATIONAL METHOD DURING THE EXCAVATION PROCESS WITH MONITORING AND PROJECT CHANGES

After cutting wide, gently sloping excavations for permanent slopes at the toe of the side of a hill in highly weathered mixed soft rock, while regulation of the “Radomlja” stream simultaneously took place on the opposite side of the future motorway route, a landslide occurred after a long autumn rainy period. Temporary restoration measures were determined: redirection of the torrential stream outside the landslide area and construction of a protective embankment. The purpose of the embankment was to protect the building site and halt the landslide’s creeping. Geodetic observation was established in order to determine the landslide settling and the possibility of beginning construction of the retaining wall. For that, a new project had to be devised so that it could later protect not only the cutting slopes but also the landslide that had occurred.

For the retaining structures we performed the numerical analyses needed for the dimensioning of retaining and support measures and their correct installation into the given geological structure. These were based on the results of additional geotechnical tests and measurements according to the observation method, complying with geotechnical standards ENV-7.

5. ANALYSIS OF DATA AND MONITORING RESULTS

The purpose of the detailed analysis of examination results and measurements gathered over a long time period was to determine the input parameters for the numerical analysis needed to simulate landslide formation and creeping. Analysis results were also used in a rheological model of temporary physical and mechanical changes of the weathered mixed rock mass of Carboniferous-Permian soft rock.

We analyzed the results of undrained triaxial shear tests in detail, combining analysis results according to the critical state theory with analysis of effective stress pairs of triaxial tests using the generalized Hoek-Brown failure criterion (GHB) and results of “in situ” measurements. Thus we determined the strength and deformability characteristics of the mixed rock mass layers actually found in the researched retaining structures area, not only in the samples. Distinctive anisotropy, in the form of schistosity, though in this area favorably directed steeply into the slope, was, together with mixed composition, structure, level of weathering and tectonic disturbance observing the genesis, unified by determination of the geological strength index GSI. Thus, analysis of the generalized failure criterion (GHB) was used to determine strength parameters and rock mass softening, enabling us to treat those as a continuum. For the determination of deformability parameters, we used data from pressiometer measurements. Based on the results of oedometric testing, we determined the viscous characteristics of highly weathered and fragmented rock mass in the area of the landslide.

The influence of time on the changes of the stress-strain state in the physical behavior of the Carboniferous-Permian mixed soft rock mass was defined. Most important in this process are pre-existing weak soft rock zones. As a result of the suction process of pore water pressure dissipation out of the micro-cracks (dilatation suction and dissipation), viscous creeping develops. The process of self-sealing is possible as long as the deformations are small and in the softer parts of rock mass where are oppress. Significant volumetric and distortion deformations appear after quite a long period of time after unloading the excavation. Finally, gradual structural degradation causes sliding of the altered or disintegrated soft rock mass as this soft rock mass takes on the characteristics of the saturated residual soft soil with few particles of the “in situ” soft rocks.

During the excavation for implementation of retaining measures, we verified the numerical analysis using the monitoring determined in the project. This included measurements of anchor forces, observations of geodetic points and measurements of water level changes in inclinometers and piezometers on the back slopes of the retaining structures [12]. With back analysis we additionally established rock mass strain softening and the eventual need for additional retaining and supporting measures. Technical observation has been ongoing while the structures are operational.

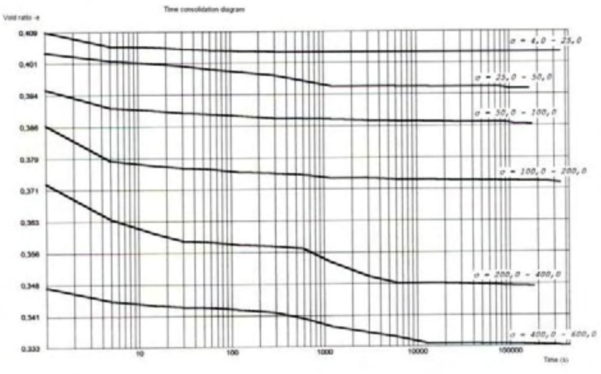

Fig. 10: Presentation of the consolidation curves (void ratio – *e*, dependent on the logarithm of time during consolidation) determined by the standard oedometric test.

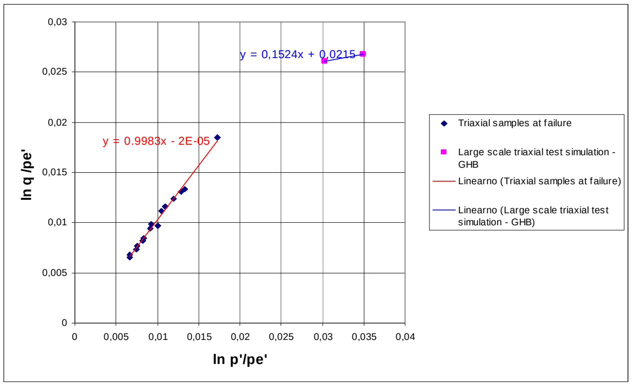

Fig 11: Presentation of effective stress invariants (ln*q'*, ln*p'*) normalized using the equivalent stress invariant p'_e for triaxial test samples and effective stress pairs at the generalized failure criterion (GHB).

6. SELECTING CONSTITUTIVE MODELS OF THE MIXED SOFT ROCK LAYERS

During the project creation phase, we determined interior static values of the required retaining structures using numerical analysis and Mohr-Coulomb model for soils and different types of rock mass. With detailed analysis of the laboratory results, "in situ" field examinations and monitoring results, we gathered enough data for creeping analysis of the landslide.

Based on determined physical and mechanical characteristics of individual layers, we used the following constitutive models of the Plaxis program (version 8.2) for analysis [13]:

- Prolluvial layer of silt to clayey gravel: Hardening soil (HS) model with kinematical hardening [14].
- Highly weathered, disintegrated and, following a longer period of heavy rain, water-saturated mixed soft rock mass subject to dilatation suction during creeping and changing of permeability: Soft Soil Creep (SSC) model with observation of viscous deformations after unloading and reloading.
- Weakened to compact mixed soft rock mass: HS model with kinematical hardening.
- Artificial defensive embankment of silt gravel: HS model with kinematical hardening.

Table 3: Strength and deformability parameters of material layers

Layer (Material model)	φ (°)	Cref (kPa)	$Eref_{50}$ (MPa)	k_{max} k_{min} (m/s)
Diluvial layer (HS)	29	1	30	
Prolluvial (HS)	30.6	1	40	
Highly weathered soft rock (SSC)	23.6	1	27	7.6×10^{-11} 1.3×10^{-9}
Weakened to compact soft rock (HS)	29	40	250	Impermeable
Embankment (HS)	35	1	30	

Table 4: Deformability, viscous parameters and void ratio of the highly weathered disintegrated soft rock zone:

λ*	κ*	μ*	OCR	e_{max}, e_{min},
2.79×10^{-3}	2.02×10^{-3}	9.52×10^{-4}	2	0.37, 0.27

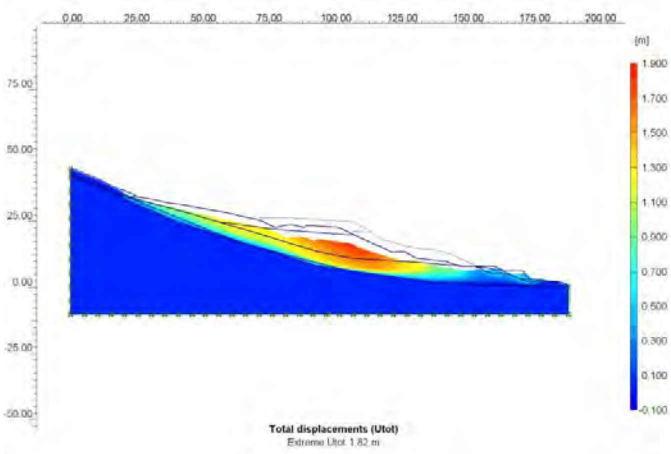

Fig. 12: Presentation of total deformations (1.82 m) after draining of water out of landslide (OZ-04) and embankment construction for defense of landslide creeping during high water levels following a longer heavy rain period.

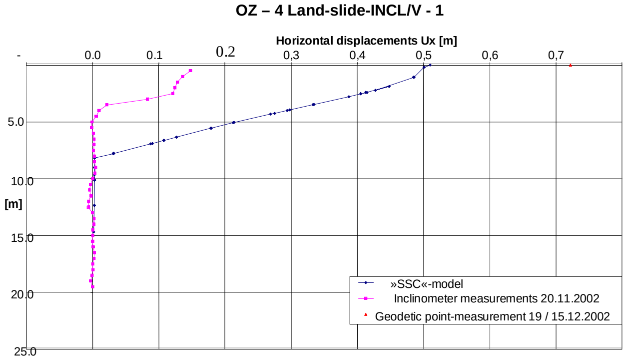

Fig.13: Model versus measured predictions inside the landslide area after construction of a protective embankment.

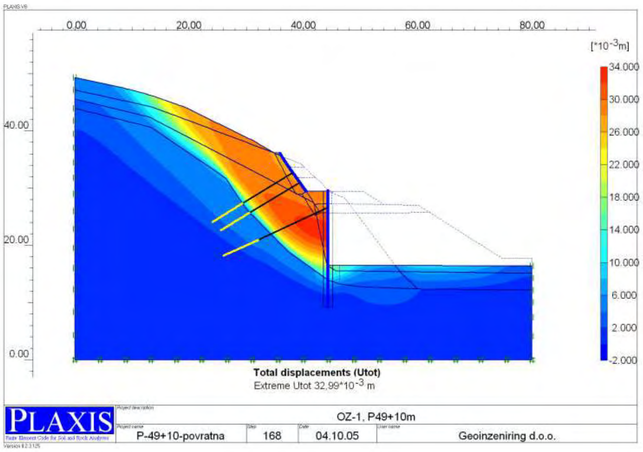

Fig.14: Numerical back analysis of measured deformations. The HS model in Plaxis fitted deformations with measurements quite well.

7. CONCLUSIONS

-Numerical back analysis of mineralogical petrography specimens at micro-scale were used to derive an appropriate failure mechanism in nonlinear materials such as Carboniferous-Permian mixed soft rock.

-Careful study of stress and deformation fields near propagating cracks following peak failure revealed that a nonlinear material model was appropriate for modelling post-failure behaviour.

-Strength and deformation characteristics fitted well with the results of laboratory triaxial tests.

-Analysis showed that pore water pressure rises during the micro-crack propagation and dilatation process (dilatation suction).

-Cracks at interfaces between dissimilar materials indicate a significantly different failure mechanism than that observed along weak fracture layers of the same material.

-In the brittle part of mixed soft rock, represented by siltstone, deformations are seen mainly as dilatancy. These are several times greater than the volumetric deformations found in the soft part of the matrix, represented by clay stone.

-A critical point of stability is conditioned by macro- and micro-structures of weak soft rock, being on the one hand tectonically crushed and on the other displaced and weakened in response to wetting and drying cycles (suction-destructurisation) during past periods of instability.

-The unloading caused by excavation for the foundation and heavy rainfall activated a deep landslide at the building site of the Blagovica-Kompolje motorway section. The landslide was successfully stabilized with an anchored pile wall, material exchange below the pile wall and a special drainage system. Before the start of excavation in the mixed soft rock mass (2002) and after establishing the retaining measures on this highway section, the results of the analysis were verified by monitoring, which is ongoing in the exploitation phase of the construction (2007). New methods of design include the determination of post-peak strength parameters of the rock mass after relaxation, along with routine measurements. Definition of post-peak strength and deformation parameters was focused on numerical back analyses (2D and 3D).

-Optimal placement of a design solution in a given geological environment can be achieved by careful geological and geotechnical supervision of the execution, besides suitably executed geological and geotechnical investigation and geostatic analysis [15].

-The material model with kinematic hardening (Fig.14) for a mixed soft rock mass of disintegrated and tectonically disturbed Carboniferous-Permian siltstone and clay stone with lenses of sandstone agreed quite well with the measured deformations. Highly weathered, disintegrated and, following a longer period of heavy rain, water-saturated mixed soft rock mass subject to dilatation suction during creeping and changing of permeability was contentment-modeled (Fig.12, Fig.13) using the Soft Soil Creep (SSC) model with observation of viscous deformation after unloading and reloading, but not as well as with the HS model, in which deformations were slightly underestimated.

- ✶ The strong influence of time on changes of the stress-strain state in the physical behaviour of Carboniferous-Permian mixed soft rocks mass is evident.
- ✶ Significant volumetric and distortion deformations appear quite a long period of time after the unloading of the excavation.
- ✶ Most important in this process are pre-existing weak soft rock zones.
- ✶ Viscous creeping develops as a result of the suction process of pore water pressure

dissipation out of micro-cracks (dilatation suction and dissipation).

- ✶ Finally, gradual structural degradation causes sliding of the altered or disintegrated soft rock. The whole mass is reduced to the characteristics of the saturated residual soft soil with few particles of the “in situ” soft rock.

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