Reactivation of fossil landslides during motorway construction in overconsolidated Neogene sediments

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Abstract: Worldwide, landslides are one of the most significant natural hazards regarding human and material losses. Landslides can significantly increase road construction or maintenance costs. This paper summarises the slope stability problems associated with the design and execution of road cuts in landslide areas. The case study investigated East Slovakia in central Europe. Slope stability failures are discussed in detail in the article. Fossil shear surfaces were identified during road construction in layered or laminated clays. Moreover, limit equilibrium calculations and finite element analysis results are compared with the measurements during geotechnical monitoring. Calculations are consistent with measurements and support the presented concept of slope movements in overconsolidated clays in the presence of fossil landslides.

Key words: landslides, slope stability, road cut, geotechnical monitoring, overconsolidated clays, inclinometric measurements

1. INTRODUCTION

Worldwide, landslides are one of the costliest natural hazards in terms of human and material losses. Landslides can close roads or cause a need for significant repairs, and they can significantly increase road construction or maintenance costs (Frankovská & Kopecký, 2015; Marschalko & Müllerová, 2002; Hungr, 2004; Şengör et al., 2013; Mahanta et al., 2016; Li et al., 2020). The reasons are the unsuitable alignment of motorways, the location of embankments in a landslide area, the failure of embankments in a short time after their construction (Frankovská & Kopecký, 2015; Brixová et al., 2022) or unpredictable geological conditions which cannot be captured by conventional geological survey (i.e. boreholes or surface geophysical measurements) (Kopecký et al., 2019). The motorway network in Slovakia has been built in an environment with frequent landslides (Fig.1). Over 670 km of road and 67 km of railway networks are endangered by landslides in Slovakia (Kopecký et al., 2012). Obviously, for a risk-based approach to landslide hazards management to be effective, practitioners must continue to strive to improve quantitative methods of hazard and risk assessment through practically oriented research (Hungr, 2004).

Due to the occurrence of landslides, the route of the D1 motorway was changed even twice in the area of central Slovakia. In the case of the Turany-Hubova section, an extensive rock slide was activated in the motorway route before its construction and the motorway route was moved to the tunnel (Fraštia et al., 2014, Kopecký et al., 2021). In the Hubova-Ivachnova section, during the construction of the motorway, deep active landslides were found. They were bypassed by extending the already projected tunnel by 1.5 km (Kopecký et al., 2019).

The article analyses the activation of fossil (Quaternary) landslides during the motorway construction in the overconsolidated Neogene sediments in the eastern part of Slovakia (Fig. 1). In a 15-km section of the D1 motorway, several landslide areas were identified during the field mapping. However, the crucial slope instabilities were recorded during construction in the areas of deep excavations, where the presence of landslides was not expected based on ground investigation and the morphological assessment of the site.

During the excavation works, the existence of so-called fossil (old) landslides, which were probably triggered during the Pleistocene, when Neogene sediments were exposed to considerable water erosion, and thus old extensive paleo-landslides were activated. Later, these landslides were covered (buried) and remodelled during younger sedimentation processes. Similar landslides were discovered during construction activities in several parts of Slovakia. An example of the formation of a landslide during the construction of the Mikšová Waterworks in central Slovakia is well known. During the excavation of the derivation channel, the movement on the old shear surfaces of the Pleistocene landslide was reactivated (Záruba & Mencl, 1987). Holocene landslides are relatively common (Alexandrowicz & Alexandrowicz, 1999; Campbell & Evans, 1990; Pánek et al., 2013; Crozier, 1995); however, landslides in Pleistocene and older sediments have been rarely reported (Mather et al., 2003).

The slip surfaces of fossil landslides were identified based on inclinometric measurements, and visual inspections during the excavation work. The influence of the overconsolidation of soils on landslides was analysed. The interaction of precipitation with cut slopes in overconsolidated soils was addressed by Take and Bolton (2011). A slope model was used in a climatic chamber (stored in a centrifuge) in which pore pressure sensors along the depth were installed. They demonstrated that the accumulation of plastic strains during the wetting and drying of cut in overconsolidated soils would cause failure while the peak shear strength parameters were not mobilised. Thus, critical state friction angle with zero cohesion would be applied for long term stability of such slopes.

Slope stability failures are discussed in this article. Moreover, results from limit equilibrium calculations and finite element analyses are compared with the measurements during geotechnical monitoring. Currently, conventional slope stability analyses

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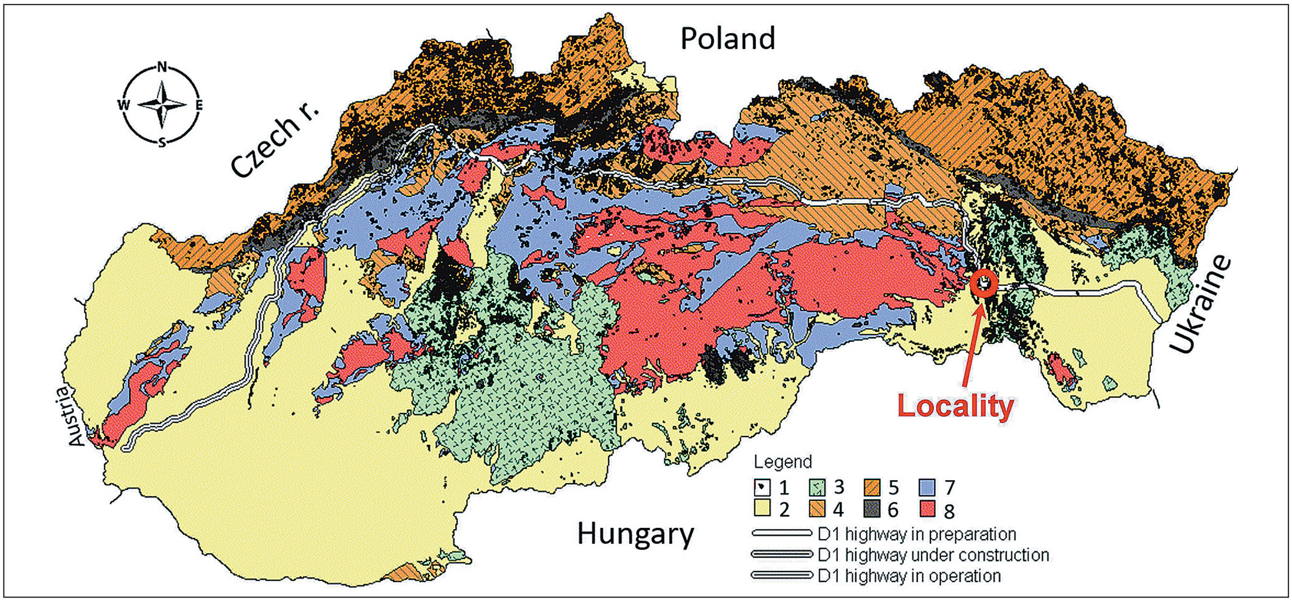

Fig. 1. D1 motorway route in relation to the geology of Slovakia and slope failures in 2018 with the described locality appointed by the mark (according to Martinčeková & Šimeková, 2007)
Legend: 1 Slope failures, 2 Neogene and Quaternary deposits, 3 Neogene volcanic rocks, 4 Innercarpathian Paleogene rocks (flysch formations), 5 Paleogene rocks of outer Carpathian zone (flysch formations), 6 Paleogene and Mesozoic rocks of Klippen Belt (mostly limestones folded into flysch formations), 7 Mesozoic rocks (folded sedimentary rocks), 8 Krystalline rocks (intensively disturbed)

based on the concept of limit equilibrium have been used due to its simplicity, ease of use, and accuracy (Mathews et al., 2014; Yu et al., 2014). A detailed review of equilibrium methods of slope stability analysis was published by Duncan (1996). Numerical analyses are widely used for the recognition of stress and displacement fields in geotechnical engineering, including slope stability (Tschuchnigg et al., 2015; Kaczmarek & Popielski, 2019; Gao et al., 2016). The results of a boundary value problem presented by Tschuchnigg (Tschuchnigg et al., 2015) confirm that the limit equilibrium analysis, the finite element limit analysis, and the strength reduction finite element analysis (Brinkgreve & Bakker, 1991) (assuming associated plasticity) are in good agreement; in addition, the safety factors obtained are remaining slightly conservative. Finite element analysis was adopted in this article to study the effect of over consolidation ratio on resultant safety factor. Also, the same finite element model was used during the construction works to predict the evolution of deformations. In such a way, final deformations (after the overall deconsolidation of the environment due to excavations) were estimated and helped to set the risk associated with progressive long term movements.

2. CHARACTERISTICS OF THE LOCALITY AND DEVELOPMENT OF SLOPE INSTABILITIES

D1 motorway in the analysed section is located in road cut, which has a length of approximately 400 m (10.6-11.0 km; Fig. 2) and a depth of approximately 17 m. Neogene sediments were excavated during the construction of the cut, mainly in clayey zones within sands and gravels. Deposits of weakly consolidated pyroclastic arenites occur in the lower parts of the Neogene formation. The geological structure is very diverse, with vertical and horizontal changes. Fig. 3 shows a simplified geotechnical profile 1 of the cut at km 10.775. The road cut was initially designed with a slope of 1:2.5. Anchored concrete beams at the foot of the cut were designed to increase the stability of the cut. The cut was excavated from km 10.8 to km 11.0. With a gradual excavation of the cut to a depth of about 12 m, landslides developed, especially on the left side in the direction of alignment (northeast). At the 10.75–10.8 km section, an active landslide with a total area of about 25 × 25 m occurred on the left side (Fig. 2). The head scarp reached a height of 2.2–2.5 m, while the tensile cracks reached a depth of 3.0 m from the edge of the modified slope (Fig. 3 and 4).

From November to December 2017 the movement on this local landslide continued, especially in connection with mild winter temperatures, freeze thaw cycles and rainfalls. Concerning the climatic conditions in February 2018 (rainfalls and constant snowmelt), the decrease of consistency (soil behaves as a viscous liquid rather than granular matter) of ground forming the slopes led to additional shallow landslides. The total area of the landslides at that time was about 60 × 30 m (Fig. 5). Such landslides clearly depicture the character of soils presented in cut slopes and under the ground surface. The above-mentioned shallow landslides were triggered due to the presence of overconsolidation of soils and microcracking after excavation. Silty and highly plastic soils sorbs the water and rapidly changed consistency during the winter and freeze-thaw cycles, which led to progressive failure and widening of sliding mass.

The shear surfaces of presented/reported shallow landslides were bound mainly to light grey clays with high plasticity (MH, CH; see Fig. 3), containing clay minerals of volcanic origin (e.g., smectite).

These minerals are expansive and can increase the volume by many times when in contact with water. Such behaviour was also

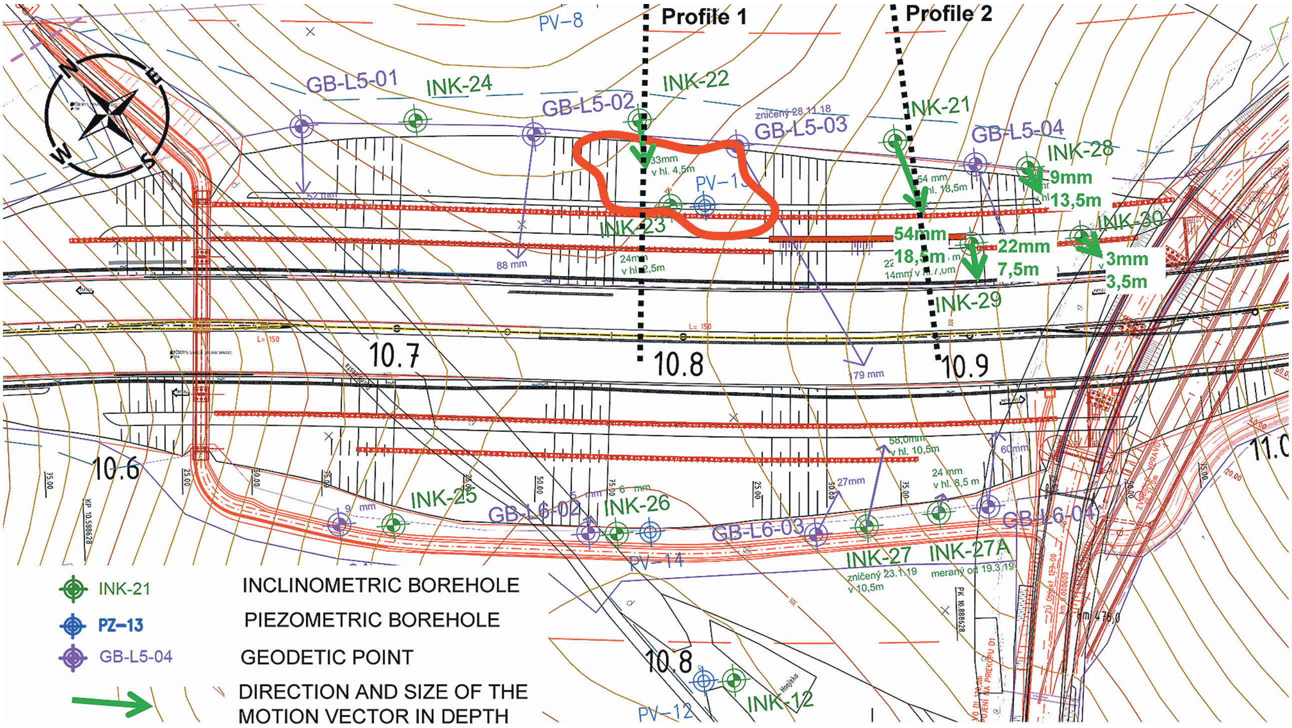

Fig. 2. Road cut at 10.6–11.0 km showing inclinometer (INK) and geodetic points (GB) locations, vectors, and movements at relevant depths (red line = active landslides). **Legend:** CS – sandy clay, CI – clay with intermedium plasticity, SC – clayey sand, CH/MH – clay/silt with high plasticity

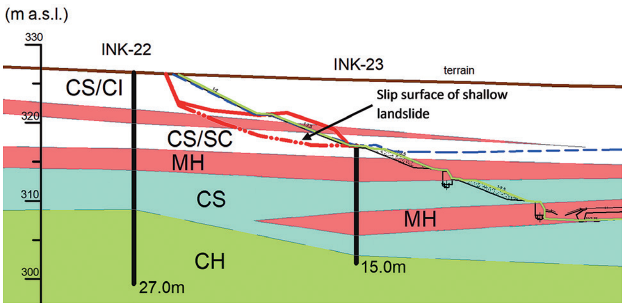

Fig. 3. Simplified geotechnical profile 1 at 10.775 km showing the temporary excavation (blue line) and design excavation (green line).

Fig. 4. Active landslides at 10.775–10.8 km (date: 25.10.2017).

observed in laboratory tests, as it will be discussed later in the article. The layers of these clays is located irregularly in several places on both sides of the cuts (see Fig 2/3, MH soil). Such disintegration is the evidence of micro layering and micro cracks in Neogene soils. Smooth and shiny slip surfaces in the clay layers and shallow landslides were recorded during the construction site inspection of exposed parts of the road cuts (Fig. 6).

The origin of the shear surfaces (weakened zones), which were observed in several exposed parts of the road cuts during its construction, could be the following:

- Old (fossil) landslides that formed in Pleistocen.
- Tectonic processes with predominant vertical movements.

A layer of gravel above the slip surface is typical for this locality. This structure was confirmed in many excavated shafts on the base of the temporary road cut excavation (Fig. 6). Investigation shafts were excavated on both sides of the road cut. In Fig. 7 the vectors of the direction of inclination of the assumed shear surfaces in the shafts are presented. Although these are point values - a certain connection can be found in the spatial distribution of these areas. The predominant direction is to the southwest, in the direction of the slope of the original terrain surface, and the slope inclination is predominantly 10-15 °. It follows from the above that the fossil landslides had a direction of movement to the southwest, their thickness varied in space and landslides of the road cuts were activated along these old shear surfaces after beginning the earthworks.

In addition to shallow landslides on the surface of the cut slopes, there were visually identified other smoothed shear surfaces in the bottom part of excavation (Fig. 6). These shear plane dedicated the presence of old fossil landslides.

3. MATERIALS AND METHODS

3.1. Ground properties

Based on an additional engineering geological investigation, the parameters of shear strength, stiffness parameters, state,

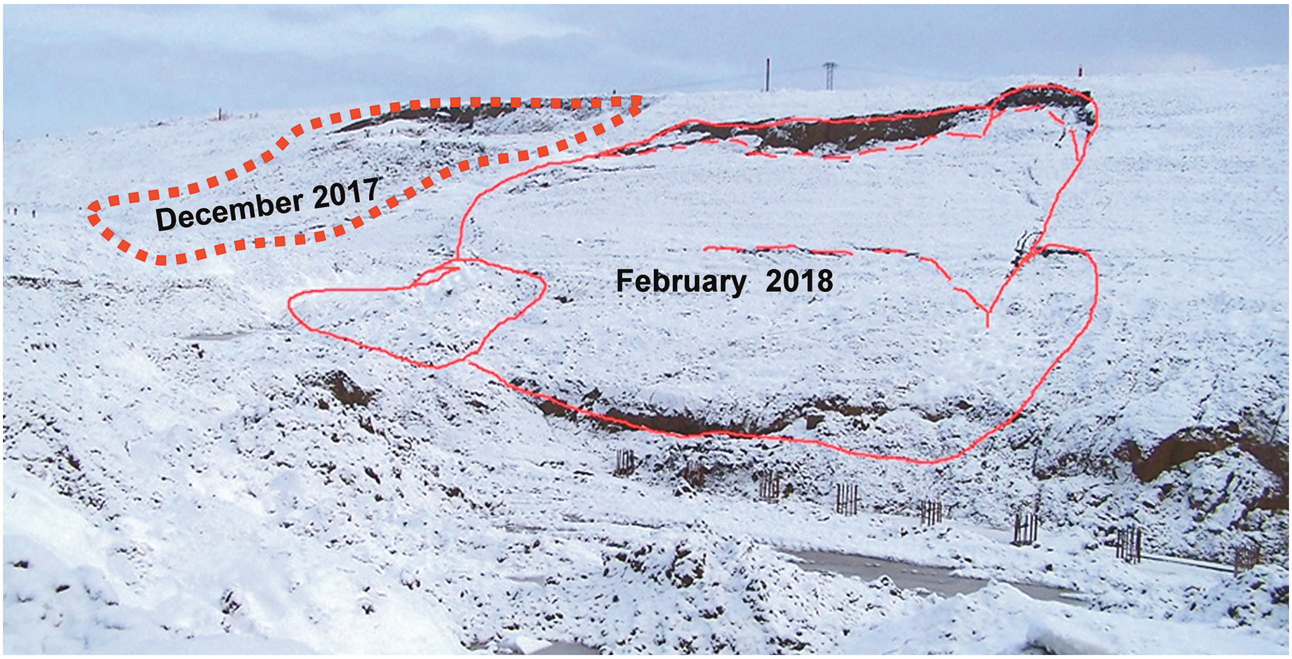

Fig. 5. Landslides at 10.775–10.84 km in February 2018 (narrow red) and December 2017 (dashed).

Fig. 6. Slip surfaces in the excavation of the cuts and the excavated shaft.

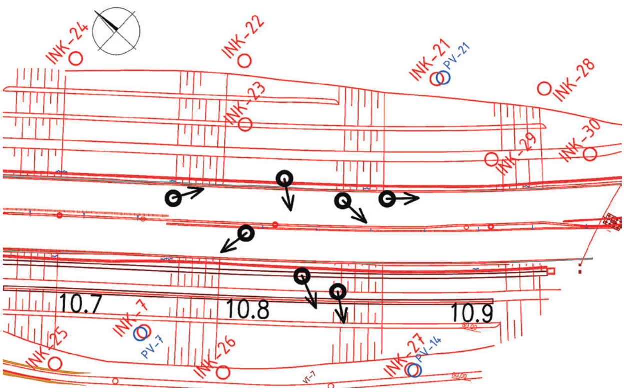

Fig. 7. Vectors of the directions of inclination of the proposed slip surfaces in the excavated shafts.

and intrinsic properties of soils in the cut were analysed. Comparable data and empirical correlations from the literature were also used for the analysis of residual shear strength angles. Fig. 8 shows the grain size curves of 42 soil samples obtained during the installation of boreholes INK-21 to INK-27. All of these soils are highly frost susceptible and contains a significant amount of silty particles.

3.1.1. Residual shear strength parameters

Determination of the residual angle of shear strength is necessary for the slope stability calculation. The results of the laboratory tests (ring shear on reconstituted samples (Bishop et al.,1971) and reverse direct shear on samples from site construction and

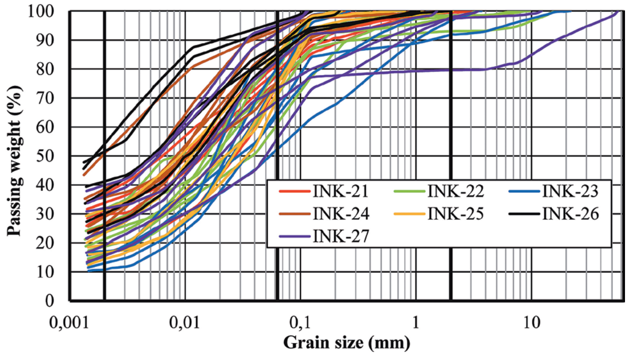

Fig. 8. The grain size distribution of the ground samples.

two others, were used in the analysis. A set of 43 values of the residual angle of internal friction (φr) used for calculation is presented in the summary graph of the relationship between φr and the liquid limit (w_L). The red line in Fig. 9 is the correlation from the literature.

Considering the relationship in Fig. 9 and the hypothesis about critical shear surface development in highly plastic clays, the range ±25 % of residual internal friction angle with w_L = 65 % was determined to be from 6.9^0 to 11.5^0. The average residual angle for samples of high-plasticity clays from the investigated locality was defined as φr = 10.4^0. Based on these values, the design value, verified by back analysis of slope stability, was φr = 8.5^0.

3.1.2. Overconsolidation

One of the aims of the additional engineering geological investigation was to perform field and laboratory tests to determine the overconsolidation ratio (OCR). The overconsolidation pressure was defined as the stress where the compressibility curve bends in the space σ–e on a semilogarithmic scale (log σ) (Casagrande, 1936). The OCR decreases with depth as the effective geostatic stress increases. The OCR values, in the range from 2 to 6, are shown in Fig. 10. In the process of sedimentation and subsequent soil loading, a change in the void ratio occurred at a constant stress. For most fine-grained soils, after primary compression, secondary compression is followed by creep, and thus void ratio change occurs at a constant pressure after consolidation. For this reason, the resulting overconsolidation pressure in the laboratory is overestimated (Boháč et al., 2013).

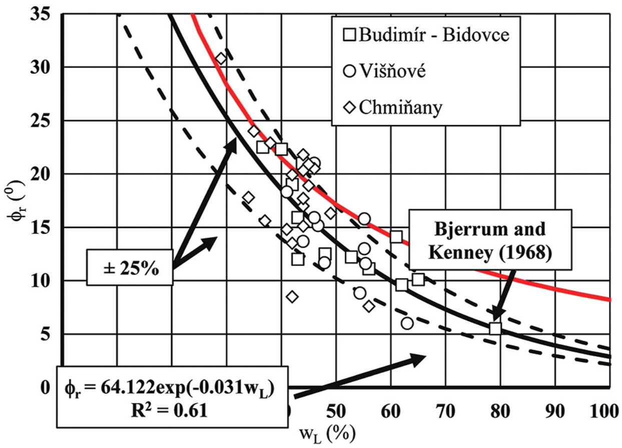

Fig. 9. Relationship between φr and the liquid limit (wL) for the samples from three different localities of the motorway construction (Budimir, Višňove, Chminany) in comparison with relationship from the literature (red line).

The effect of the creep can lead to the indication of false overconsolidation pressure (Powrie, 2004) as indicated in Fig. 11. Due to these findings and the absence of other testing (e.g., field tests), the OCR value was considered to be 3.5 for soils affected by the cut and 1.5 for soils below the bottom of the designed cut.

3.1.3. Swelling pressure

During the oedometric tests, swelling pressure was monitored in four samples (INK-22, 4.7-5.0 m; INK-24, 12.0-12.4 m; INK-25, 11.0-11.3 m; INK-26, 11.0-11.2 m). The zero-deformation method was used. Fig. 12 shows the relationship between the swelling pressure and the liquid limit. It is obvious that soils presented in cut slopes are susceptible to high volume changes when they are in contact with water. As mentioned in part 2.1, such behaviour had been already observed in the field.

3.2. Geotechnical monitoring

Geotechnical monitoring is a set of activities which aim is to determine the state of the ground and monitoring is the progress of this condition in time and space. Inclinometers were used for measuring horizontal movements of subgrade perpendicular to the axis of the borehole. The measurement is based on detection of the inclination of the guided probe in the particularly cased

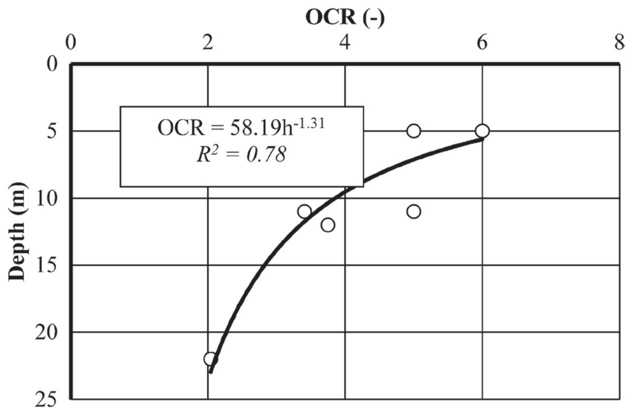

Fig. 10. Variation of overconsolidation ratio (OCR) with depth from laboratory testing of samples in the boreholes INK-21 to INK-27.

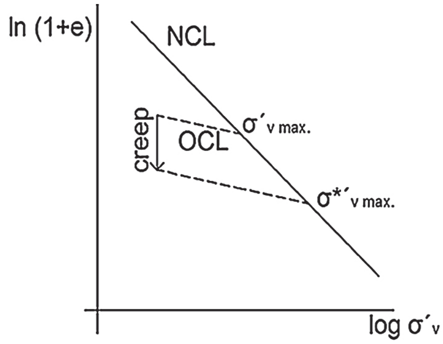

Fig. 11. Failure at OCR determination due to creep (according to Boháč et al., 2013).

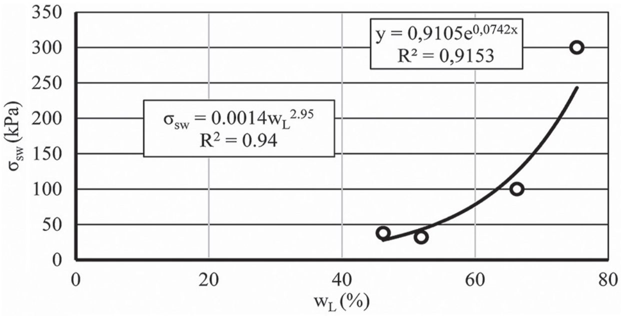

Fig. 12. The dependence of the swelling pressure (σSW) on the liquid limit (wL) of samples in the boreholes INK-21 to INK-27.

borehole. Tests are conducted at regular depth intervals (usually 0.5 m) in two orthogonal axes with automatic determination of azimuth. Repeated measurements allow high accuracy and reliability to determine the speed of movement, depending on the time intervals between the individual measurements. The monitoring system consisting of 2 inclinometers was supplemented with seven inclinometers (INK-21 to 27) in the entire road cut excavation after the shallow landslide occurring (Fig. 4). The first control measurements on inclinometric boreholes in January 2018 verified the movement in the INK-22 inclinometer, located just above the head scarp of the active landslide (Fig. 3). The movement was recorded at a depth of about 3.7 m (Fig. 13) and had an accelerating character. The detected depth of movement corresponds to the shape of the slip surface of the active landslide. Movement in INK-21 also had an accelerating tendency. Inclinometers INK-21 and -22 were monitored weekly. Motion at INK-21 continued at a constant rate of 14 mm/month; at INK-22 the rate increased slightly to 11 mm/month.

3.3. Numerical modelling

For analysis by numerical calculation, the hardening soil model in the PLAXIS FEM software (Schanz et al., 1999)] was used; it can model excavation deformations, stress release and stiffness dependency on various stress paths by optionally entering three different deformation modules E_{50} (modulus at 50 % transformation from peak deformation), oedometric module, Eur (relief/loading module used for excavations or reloading of the soil mass). Safety factors were also calculated by the means of the strength reductions technique (Brinkgreve & Bakker, 1991). As part of the analysis, calculations were performed using the limit equilibrium method implemented in the GEO5 slope stability module, Bishop circular shear surface (Bishop, 1955); Morgenstern-Price polygonal shear surface (Morgenstern & Price, 1965; 1967). Due to the complex hydrogeological conditions at the site and in the soil mass, the Ru coefficient was used to model the water in the soils, determined as the ratio between the water pressure in the soil and the geostatic pressure. The parameter Ru was used in the clay-silt environment of the overconsolidated clays, e.g., by Palladino and Peck (Bjerrum & Kenney, 1968). The values Ru = 0.15 (an increase in pressure in soil pores of 3 kPa/m in the case of volumetric gravity of soil 20 kN/m^3) and Ru = 0.3 in the case of saturation of pores by precipitation

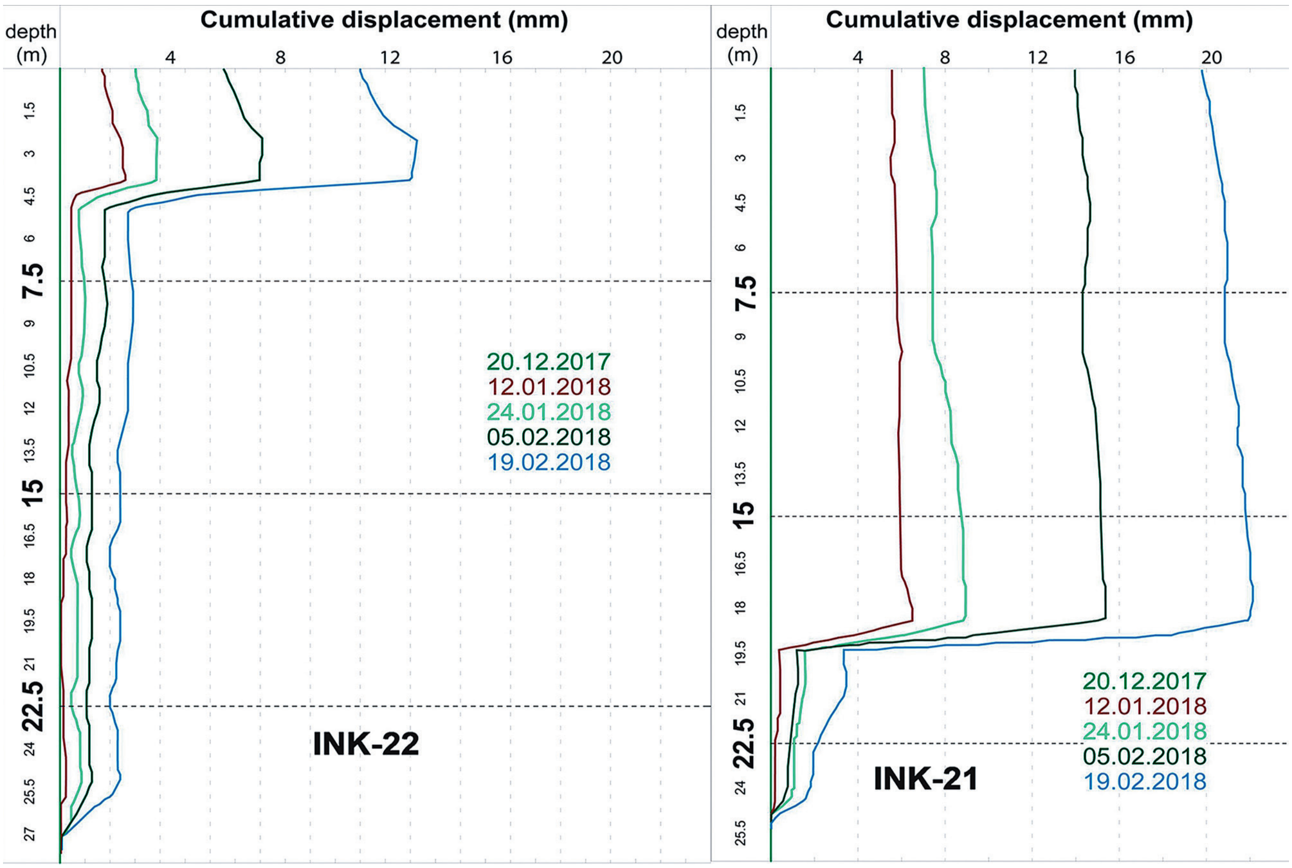

Fig. 13. Cumulative displacement curves obtained by the inclinometers INK-21 and INK-22 in the road cut for the period from December 2017 to February 2018.

were used in the analyses (an increase of pressure in soil pores by 6 kPa/m in the case of soil gravity 20 kN/m^3). These values were later confirmed by the measurement of pore pressure sensors in bentonite grout in borehole filling. The parameters for the calculation model of FEM and calculations using the limit equilibrium method are presented in Tab. 1.

Tab. 1 Soil parameters and input values for numerical modelling
1 Note. ψ is angle of dilation, Φ ef is effective angle of internal friction

Soil	Φ_{ef} (°)	Φ_r (°)	ψ (°)[1]	E_{50} (MPa)	E_{ur} (MPa)	OCR (–)
CS/CI	22–27	15	5.49–6.33	20–25	50–62.5	3.5
MH/CH	16.5–23	8.5	3.62–5.85	15–20	37.5–50	1.5–3.5
SC/CS	29		3.38	30	75	3.5

4. RESULTS OF STABILITY ANALYSIS

The most important information about the global stability of the cut slope was that in the inclinometer INK-21 (km 10.85), where there was a movement on the slip surface at a depth of about 17.7 m (Fig. 13), which would correspond to the contact of clayey gravel with high-plasticity clays. The old, weakened failure surface was activated. The instability on the slope occurred in a partially excavated cut (blue line in Fig. 14). Until the execution of the final shape of the slope of the cut, another 7 m remained to be excavated. A new inclinometric borehole, INK-29 and INK-28, were installed to verify the course of the slip surface on the left side (northeast) of the cut in March 2018 (Fig. 2).

A slip surface at 7.0 m depth was found in INK-29 (Fig. 15) after the first measurements. A slip surface at a depth of approximately 13.5 m was detected by monitoring the INK-28 (Fig. 15).

The results of inclinometric measurements confirmed the existence of deeper failures in the slope of the cut. The largest movements were found in the INK-21 inclinometer, at a depth of about 18.0 m below the surface, while the movement had an accelerating character. For this reason, the further excavation

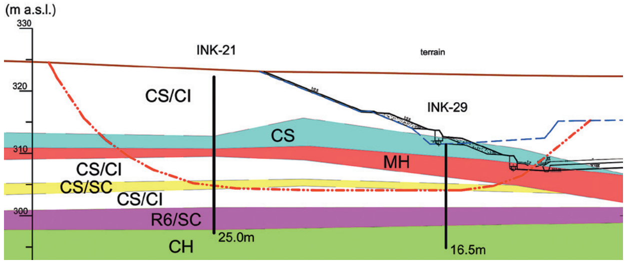

Fig. 14. Profile 2 with slip surface at 10.85 km (dashed red line). Blue line = a level of temporary excavation.

of the cut was stopped for about nine months, and the stability of the slopes was analysed, along with an analysis of the future developments expected after the resumption of excavation work. Using the finite element method (FEM), the following were analysed:

Effect of OCR on slope stability and resultant safety factor,
Deformation of slopes after excavations,
Influence of shear surfaces on horizontal deformations.

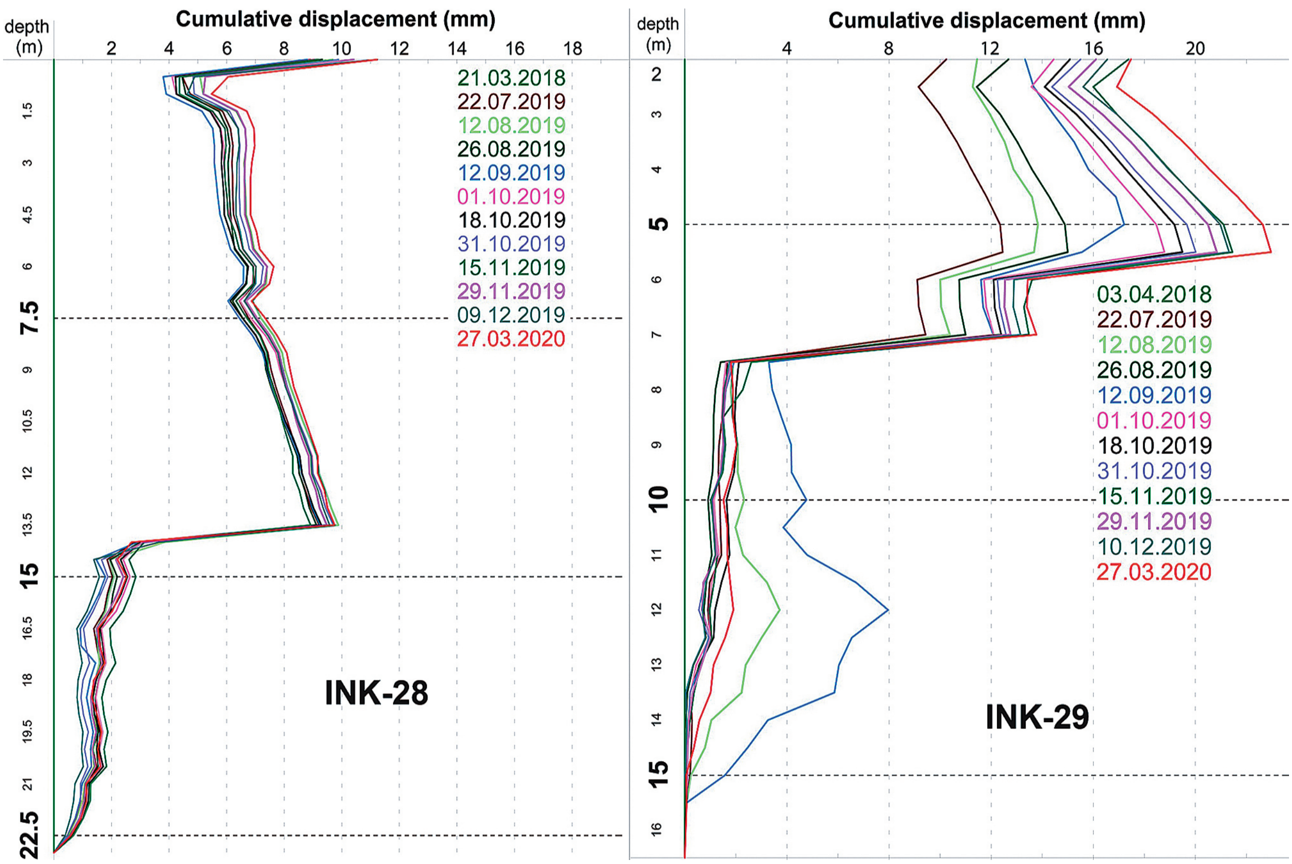

Fig. 15. Cumulative displacement curves obtained by the inclinometers INK-28 and INK-29 in the cut.

4.1. Influence of OCR on slope stability

In the case of overconsolidated high-plasticity clays, large horizontal stresses were released during the excavation of the cuts. Increased horizontal pressure at rest is the result of overconsolidation (Mayne & Kulhawy, 1982). When stress is released, both horizontal and vertical deformations and volumetric deformations occur. These results in microcracks, which create a path for the infiltration of water into the soil mass and thus a change in its water content. Most soils are in a narrow grain size range, and almost all samples contain a sand fraction of 5-40%. Such soil with a significant silty fraction is not impermeable and the value of coefficient of filtration is in the range $k = 10^{-9}–10^{-7}$ m/s. This means that soils will relatively easily absorb water from precipitation or melting snow, which will increase the pore pressures in the ground. The increase in moisture and contact with water also causes further development of microcracks, increase in porosity, and progressive degradation of soil structure. The consistency changes to a soft or liquid one, showing markedly viscous behaviour. This phenomenon is mostly related to presented shallow landslide. The influence of OCR and horizontal stress release on the formation of sliding structures were analysed on a simplified model using a parametric study. The model used was the hardening soil model. In both models, the tensile strength of the ground was 5 kPa. The resulting factor of safety was about 1.42. For the stability analysis, the φ–c reduction method was used, where the shear strength parameters are gradually reduced until failure is achieved. While the significantly different deformations were achieved for different OCR values (Fig. 16) the resultant factor of safety was not changed.

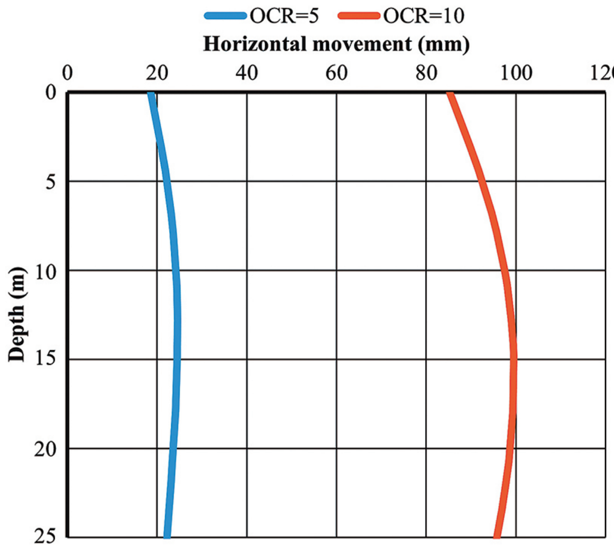

Fig. 16. Horizontal deformations after excavation in the cut slopes with OCR = 5 and OCR = 10.

4.2. Analysis of slope deformations after excavations

Deep movement at a depth of about 18 m below the surface (Fig. 17) on the left side of the cut (northeast) was identified at 10.85 km. The degree of stability (FS) when introducing a factor Ru = 0.15 with a residual friction angle of 8.5^0 is FS = 1.01 for the left side (Fig. 17). It was verified by calculation that the calculated value $\varphi r = 8.5^0$ is close to the real state. Slightly modified terrain was used in the FEM calculation, and part of the heel load (stabilising load) was not considered, and higher degree of stability FS = 1.15 was achieved (Fig. 18). Slightly higher factor of safety can be explained by higher horizontal stresses which acts on the back of the landslide and thus increases the normal force on sliding surface. At the time of assessment, the movement is identified only on the left side, but, based on calculations, it is clear that the slope on the right side (southwest) is not very different in terms of the stability and observed slips surfaces in shafts.

Deformations at the INK-21 site from the FEM analysis were compared for the individual phases of excavation up to the current state with the measured displacement in INK-21 during the geotechnical monitoring. Results are shown in Fig. 19.

The deformations at the INK-21 site from the FEM analysis are presented in Fig. 19. Deformations without landslides were subtracted from deformations with a landslide, so that only the increase in landslides without the effect of unloading of the soil mass was obtained. The total deformation from the beginning of excavation until the cut was made at the second bench (the start of the monitoring) was almost 40 mm on the shear surface. The excavation to the anchorage level was about 10 mm (approximately the current movement in INK-21). The analysis of the section at 10.85 km showed that the left-hand cut (northeast) could not be realized according to the design and appropriate remedial actions were designed These remedial actions were protection of slopes against climate effects. Additionally, anchored pile walls were designed in order to stabilise deep landslides.

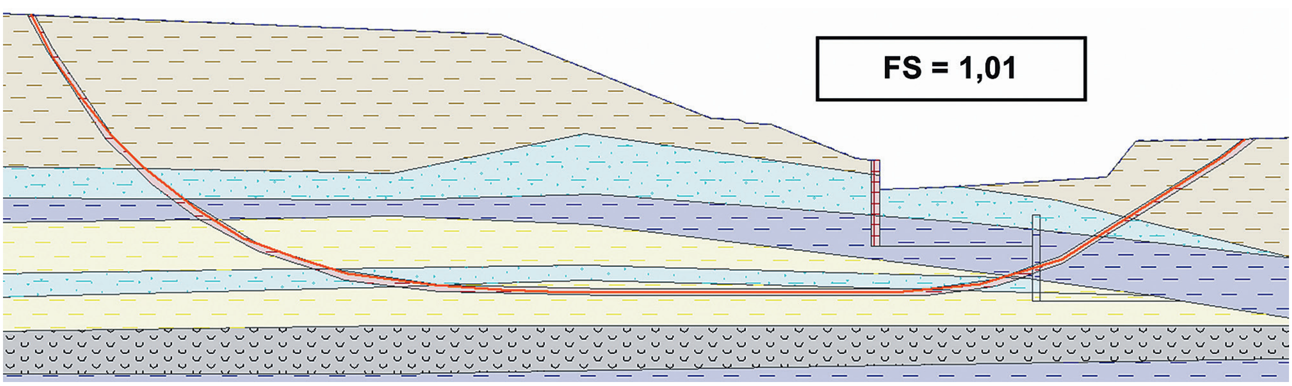

Fig. 17. Slope stability at 10.85 km (GEO 5).

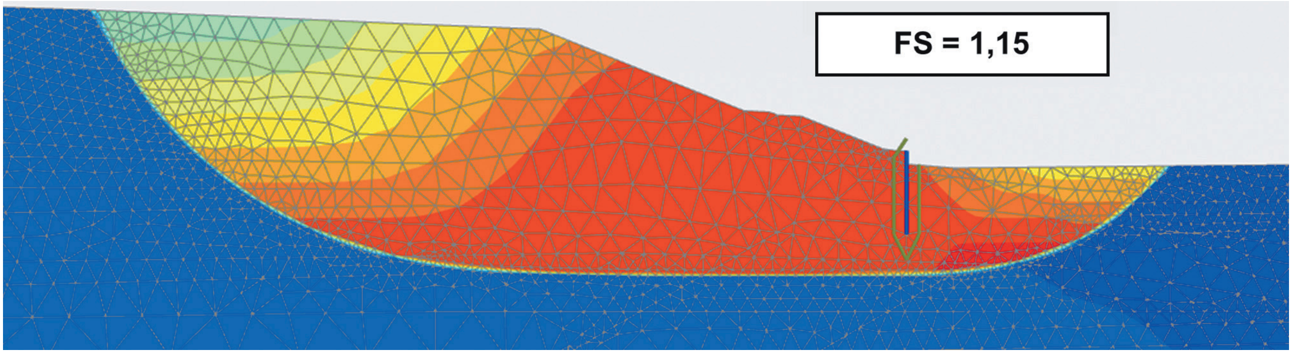

Fig. 18. Slope stability at 10.85 km (FEM) without slope toe loading.

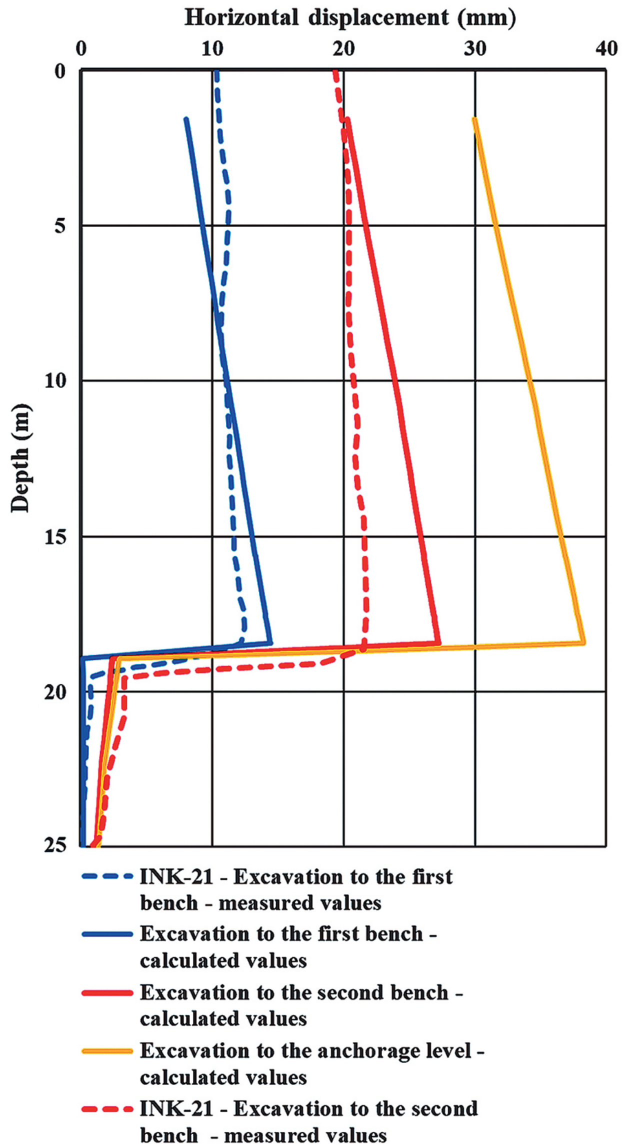

Fig. 19. Deformations around INK-21: measured values in geotechnical monitoring and data from FEM modelling.

5. CONCLUSIONS

It is almost impossible to demonstrate clear evidence of the existence of inactive shear surfaces, especially in layered or laminated clays, by drilling and sampling, or even by using intact sampling shafts (Palladino & Peck, 1972). The existence of the potential shear (weakened) zones in the cut at 10.5-11km was identified only during the construction when uncovered by soil excavation. Identification of these zones from drilling cores during the ground investigation was not possible. The reason for slope stability assessment of the road cut at 10.500-11km on the D1 Budimír-Bidovce motorway route was the appearance of the shallow landslides in October 2017 on the left side of the cut at the excavation site. Thus, the original assumptions considered in the design have been supplemented with results from slope stability analysis. Input data were obtained via soil testing, visual field inspection and additional geotechnical monitoring.

Observational method of construction was applied (Peck, 1969) in the presented case, after first indications of the possible presence of old fossil landslides. Ancient landslides were indicated by the presence of overconsolidated Neogene laminated high plasticity silts/clays underlain by gravelly layers, old slips surface observed in survey shafts and experience of the authors of the presented article. Additional survey and monitoring measurements with the aid of the finite element analysis helped to understand the behaviour of old fossil landslides but also of shallow landslides triggered by climate conditions. The deformations from the FEM analysis are in good agreement with the results from geotechnical monitoring. The slope instability analysis helped to prevent further slope failures and reduce risk in subsequent excavations. If no appropriate actions would be taken, slope failures may extend the period of construction and cause further economic damage. Presented analysis and gained knowledge helped to design remedial actions for analysed slopes. However, further remedial actions were designed for cuts not excavated at that time, and these actions helped to prevent slope failures and reduced the cost of potential repairs. All predicted fossil landslides shown only minor movements due to appropriately designed elements as pile walls and grouted ground anchors, after the full excavation. Slopes were protected by gravelly layer to prevent frosting and impermeable membranes as water infiltration protection.

The presented article demonstrated the power of field observation, laboratory tests and numerical calculations in difficult ground conditions. All the elements, as mentioned above, helped to achieve cost-effectivity and eliminate most of the natural risk.

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