CUT STABILIZATION ON THE HIGHWAY E-763 MADE IN ROCK MASS WITH UNFAVORABLY ORIENTED DISCONTINUITIES

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Abstract: *During the construction of the highway "Miloš Veliki" E-763 through Serbia, in addition to many challenges, it was necessary to stabilize the cut at the section km 81+250 to km 81+650. The cut was supposed to be constructed in the soft layered rocks of the Ljiški flysch complex, made of sandstone, marl stone and shales. The orientation of the rupture assembly of the rock mass as well as the spatial position of the slope led to a series of instabilities on the left side of the cut, while at the same time they had a positive effect on the stability of the right side. All occurrences of instability were accompanied by a dip direction of bedding, which on the left side of the cut had a dominant negative impact on the strength of the rock mass. For the purposes of geotechnical modeling and determination of remedial measures, the parameters of the rock mass were obtained using classifications of the rock mass, while the parameters of rock mass discontinuities and sliding surfaces were obtained from back analyzes of slipped rock blocks or slipped debris material. By using passive anchors, subhorizontal drains, shotcrete, wire netting, reinforced soil and facing RC walls, this cut was successfully stabilised.*
Keywords: *Landslide; Slope stability; Highway design; Orientation of discontinuity.*

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1. INTRODUCTION

For the construction of the highway E-763 and the local road, section 1: Ljig (Donji Banjani) - Boljakovci from km 77+118.23 to km 87+839.44, extensive geotechnical investigations were carried out in several phases. These geotechnical investigations included the section of the cut from km 81+250 to km 81+650, which is the subject of this paper. The terrain in the cut zone was slightly undulating, covered with Quaternary sediments, without open outcrops of rock mass. During the mapping of the terrain and exploratory drilling, in the first phase of design, it was determined that the terrain is built of soft layered rocks of the Ljiški flysch complex, i.e. sandstone, marl stone and shales, which are covered by Quaternary deposits with a thickness of 2.0 to 7.0 m. During the field investigations, it was not possible to determine with certainty the rupture assembly of the rock mass, as well as the potential phenomena that would arise because of the most unfavorable type of instability in the rock mass. Problems appeared immediately at the beginning of the excavation, i.e., there was a planar fallout of larger blocks of rock mass and the formation of a landslide on the left side of the cut (**Figures 1 - 6**).

Figure 1. and 2. Planar sliding of rock blocks above a local road

Figure 3. and 4. Planar sliding of rock blocks in the highway area

Figure 5. and 6. Sliding of Quaternary sediments in the local road

2. TYPES AND SCOPE OF PERFORMED GEOTECHNICAL INVESTIGATION AND TESTING

A total of 15 investigation boreholes were done for the purpose of defining the geological structure of this section. Soil and rock mass samples were taken for laboratory testing. The position of the investigation boreholes is shown on the engineering geological map (**Figure 7**).

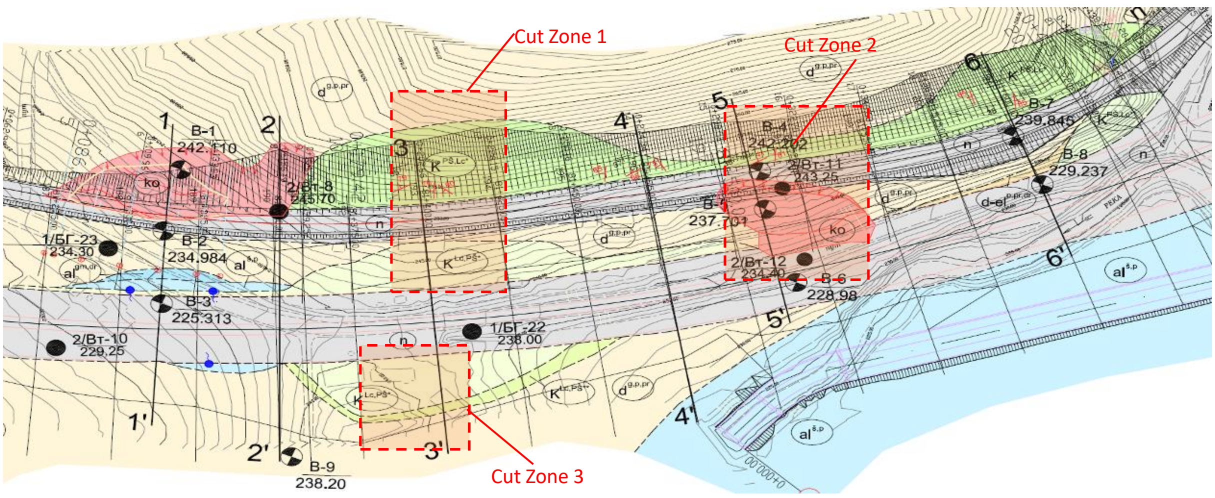


Figure 7. Engineering-geological map with the location of the performed investigation boreholes

Based on the conducted research, the following engineering-geological units have been determined, which participate in the construction of the terrain on which the cut of the local road and highway will be constructed:

- Regional road and highway embankments (n): man-made deposits; the embankment is technically arranged, built of crushed stone to a depth of 0.5 m, well compacted, the deeper parts are built of sandy silt with inclusions of debris (the thickness of the embankment varies from 0.5 to 1.0 m).
- Colluvium (Ko): landslide body; the depth of the landslide body is from about 2.4 to 7.0 m (diluvial clays and diluvial - eluvial sandy clayey silt with inclusions of debris were affected by the sliding process);
- Diluvial - eluvial - sandy clayey silt (d-elp, pr, dr): they build the slope parts of the terrain, with the rock mass of the flysch complex as sub base, they have a variable amount of debris in the mass, the thickness of the layer varies from 1.0 to 7.0 m;
- Cretaceous flysch with sandstones, marls stone and shales (K21): dominant are yellow-gray sandstones and gray-blue marls stone and shales, and locally there are interlayers of marly limestone and shales; the rock mass consists of layers and banks, most often from 5 to 100 cm thick.

The main rupture assemblage of Cretaceous flysch consists of three main joints sets with dip elements P1: 47/65, P2: 118/77 and P3: 339/74, as well as beddings with dip elements S 232 /37 (**Figure 8**). Joints sets are at a medium to long spacing (from 0.2 to 2.0 m). According to the joint roughness classification (Barton & Choubey, 1977), joints have a JRC value of 3 to 6, which places them in the group of flat rough joints. Bedding is of medium to large length (from 3 to over 15 m) so that they form a continuous rupture complex. This continuity of bedding has a dominant negative effect on the shear strength in the direction where the beddings are oriented negatively, towards the excavation, so that they represent the main direction of planar sliding of the blocks, along which larger-scale movements can occur.

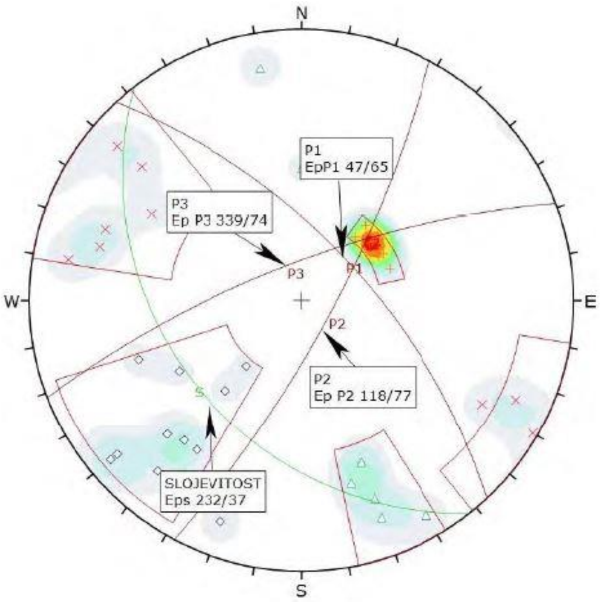


Figure 8. Contour diagram of joint sets and bedding

Uniaxial strength of intact rock samples ranges from 12.30 to 48.57 MPa. The joints have a separation of 0.1 to 50 mm, with present soft to hard joints infilling with a clayey-sandy material. Rock mass pieces are easily separated along the joints. According to the rock quality assessment based on the RQD classification, this rock mass belongs to poor to good rock, considering the large range of RQD from 10 to 80%. Results of laboratory tests of rock samples testing are shown in **Table 1**.

Table 1. Results of laboratory tests of rock samples

BULK DENSITY (γ) kN/m3	COHE-SION (c) MPa	ANGLE OF INTERNAL FRICTION (φ) °	USC (σc) MPa	TENSILE STRENGTH (σi) MPa	POISSON'S RATIO (μ)	ELASTICITY MODULUS (Eei) MPa
22.5 – 25.4	4.4 – 8.8	44.9 – 56.0	12.3 – 48.6	2.2 – 4.9	0.164 – 0.346	2206 - 11410

Within the Ljiški flysch complex, two zones of the rocks are determined according to the representation of lithological members (**Figure 9**), which can then be further divided into two subzones based on the physical and chemical degradation (weathering) of the rock mass, as follows:

1. Cretaceous sandy marl stones and marl stones ($K^{Lc,Pš}$)
   - Highly weathered ($K^{Lc,Pš**}$)
   - Moderately weathered ($K^{Lc,Pš*}$)
2. Cretaceous marly sandstones ($K^{Pš,Lc}$)
   - Highly weathered ($K^{Pš,Lc**}$)
   - Moderately weathered ($K^{Pš,Lc*}$)

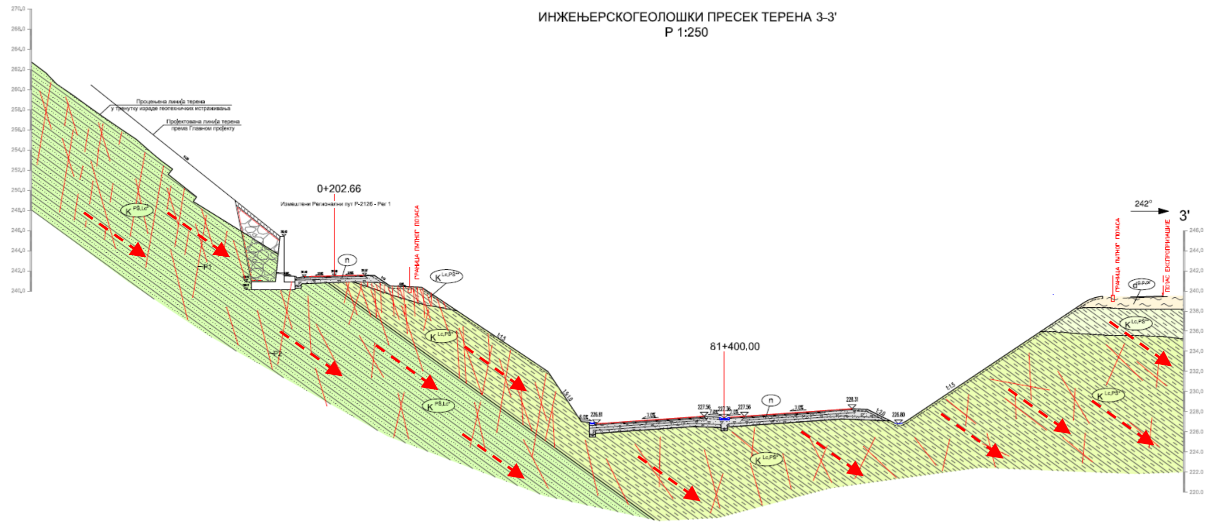
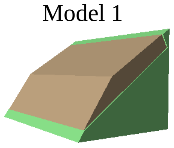
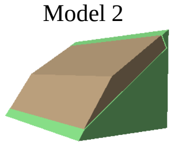


Figure 9. Characteristic engineering geological section of the terrain in the cut zone of the E-763 highway at km 81+400.00

To define the parameters of the rock mass, Hoek & Brown's empirical method was used to assess the shear strength of jointed rock mass. The solid rock mass is represented by the constants mb, s and a, which are calculated based on the values of mi, GSI (geological strength index) and factor D. The values of parameters mi and GSI depend on the type of rock, composition, texture, and jointing of the rock mass. The values obtained in the manner described above were used to estimate the Coulomb-Mohr parameters of shear strength, angle of internal friction and cohesion, which were determined for the average cut height on this road section of 15 m. The Burton-Bandis classification was used to define the shear strength along the discontinuity. This classification uses input parameters such as: base angle of internal friction (φb), joint roughness coefficient (JRC) and joint wall strength (JCS), which are transformed into Coulomb-Mohr shear strength parameters along the discontinuity through an empirical link.

The parameters of the rock mass along the discontinuities and sliding surfaces (obtained by classification) were checked by back analysis of slipped rock blocks or slipped debris material (**Table 2, Figure 10** and **Figure 11**). All the conditions that prevailed in the terrain at the time of the instability, such as the geometry of the terrain before the sliding, the dimensions of the sliding body, the groundwater conditions, defining the position of the sliding surface, and position of lithological members, were simulated in the back analyses.

Table 2. Calculated shear strength parameters along the discontinuity obtained by back analysis

<table>
<tr><td colspan="2" rowspan="2">MODEL</td><td colspan="2">SAFETY FACTOR - Fs</td></tr>
<tr><td>Without the influence of groundwater</td><td>With influence of underground water 50%</td></tr>
<tr><td>Model 1
</td><td>Sliding along beddings with dip 35°; H-5m
φ = 30° L-10m
c= 8 kN/m2
γ = 23.5 kN/m3</td><td>1.18</td><td>0.97</td></tr>
<tr><td>Model 2
</td><td>Sliding along beddings with dip 35°; H-2.5m
φ = 30° L-3.5m
c= 5 kN/m2
γ = 23.5 kN/m3</td><td>1.20</td><td>1.01</td></tr>
</table>

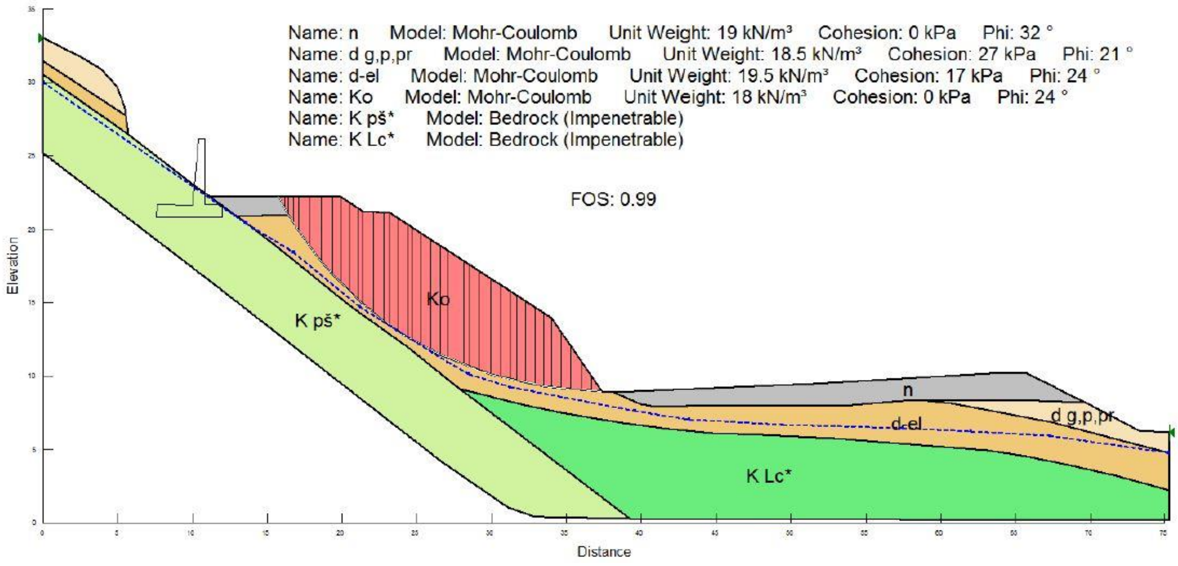


Figure 10. Determination of shear strength parameters along the sliding surface in diluvial-eluvial sediments using back analysis

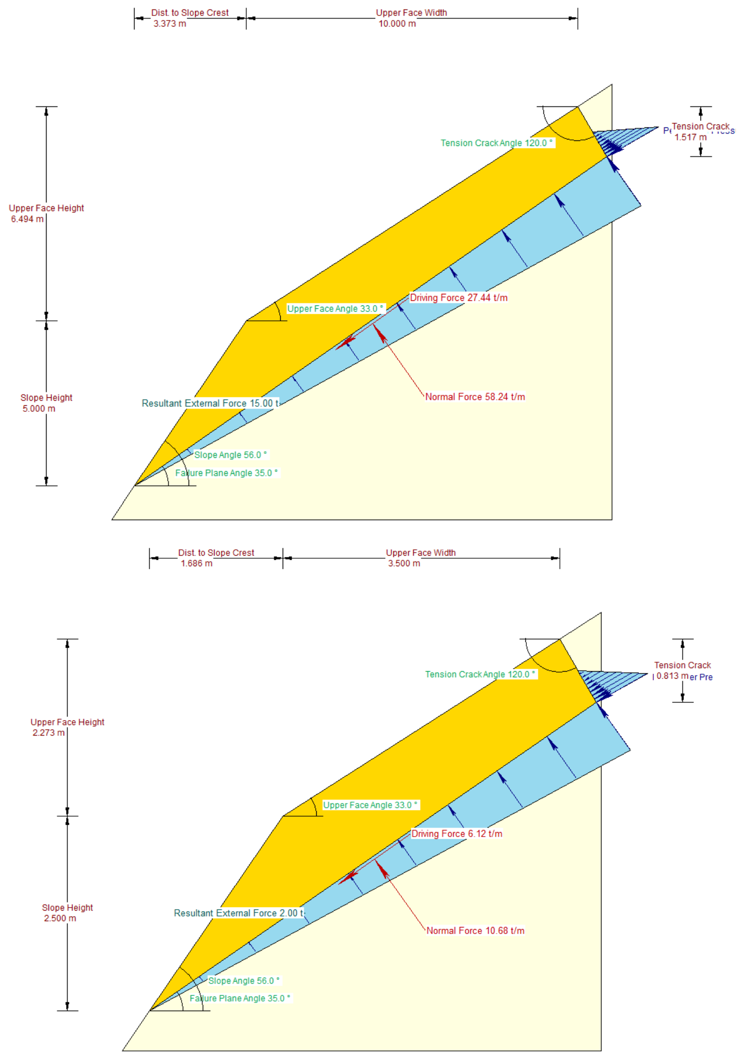


Figure 11. Dimensions of unstable blocks for calculation models 1 (up, block with larger dimensions) and model 2 (down, block with smaller dimensions)

For the remediation of unstable cut on this section of the road, the parameters of the selected geological units were adopted and shown in **Table 3**. The parameters were adopted based on the synthesis of data from the Main Project (The main design of highway E-763, 2011), newly obtained data resulting from field and laboratory research, as well as based on performed back analyses.

Table 3. Physical-mechanical parameters of the represented engineering geological units

ENGINEERING GEOLOGICAL UNIT		PHYSICAL - MECHANICAL PROPERTIES	VALUES
Embankment (n)		Angle of internal friction	32 °
		Cohesion	0 kN/m2
		Bulk density	19.0 kN/m3
Colluvium (Ko)		Angle of internal friction	24 °
		Cohesion	0 kN/m2
		Bulk density	18.0 kN/m3
Deluvium – clay (dg,p,pr)		Angle of internal friction	21 °
		Cohesion	27 kN/m2
		Bulk density	18.5 kN/m3
Diluvial - eluvial sand dust (d-elp,pr,dr)		Angle of internal friction	24 °
		Cohesion	17 kN/m2
		Bulk density	19.5 kN/m3
Moderately weathered- marl stone (KLC,PŠ*)	Rock mass parameters	Uniaxial strength	20.0 MPa
		Geological strength index	GSI = 32
		Material constant	mi = 9
		Disturbance factor	D = 0.5
		Angle of internal friction	40 °
		Cohesion	82 kPa
		Bulk density	23.7 kN/m3
	Shear strength parameters along the discontinuity	Angle of internal friction	30 °
		Cohesion	5 - 10 kPa
Moderately weathered- marly sandstones (KPŠ,LC*)	Rock mass parameters	Uniaxial strength	35.0 MPa
		Geological strength index	GSI = 45
		Material constant	mi = 14
		Disturbance factor	D = 0.5
		Angle of internal friction	53 °
		Cohesion	176 kPa
		Bulk density	23.5 kN/m3
	Shear strength parameters along the discontinuity	Angle of internal friction	30 °
		Cohesion	3-7 kPa

3. STABILITY ANALYZES FOR MODIFICATION, REMEDIATION, AND CUT PROTECTION

This paper describes in detail the applied remedial measures for the stabilization of the cut on three characteristic sections, while on other parts of the cut, similar or slightly modified solutions were applied. An anisotropic soil model was used for slope stability calculations, with the application of remedial measures (**Figure 12**), to include the anisotropy of rock mass strength in the calculations. The strength of the rock mass in the bedding dip is represented by low shear strength (shear strength along the discontinuity), while the strength in other directions is significantly higher (shear strength of the rock mass).

The first model, with applied remedial measures, represents the central part of the left side of the cut, where the planar sliding of the rock mass blocks along the "negatively" oriented beddings occurred.

After a series of performed analyses, a solution was given for the stabilization of this part of the cut, which contained the following remedial measures (**Figure 13**):

- Construction of a cut above the regional road at an inclination of 34-38°, which corresponds to the bedding inclination, with covering with a double-twisted wire mesh.
- Construction of the slope between the regional road and the highway in a dual slope of 1:1.5 and 2:1 with the application of passive self-drilling IBO anchors (Ø32mm) 2 m long (on the flatter part of the slope) and 8.0 m (on the steeper part of the slope), at a pattern of 3x3m (on the flatter part of the slope) and 2x2 m (on the steeper part of the slope), construction of drainage channels, installation of subhorizontal tubular drains (Ø50mm) 6 m long, protection of the face of the slope from weathering, covering the face

of the slope with a road network with the installation of geocomposite, and protection of the face of the slope from surface erosion by hydro-seeding (on the flatter part of the slope), while the steep part is protected by a facing RC wall.

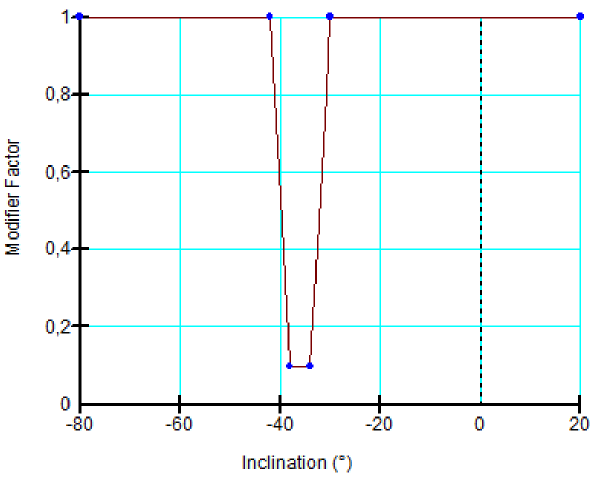


Figure 12. The anisotropic function used for geostatic calculations, which simulated a sudden drop in shear strength in the bedding directions of the 34-38°

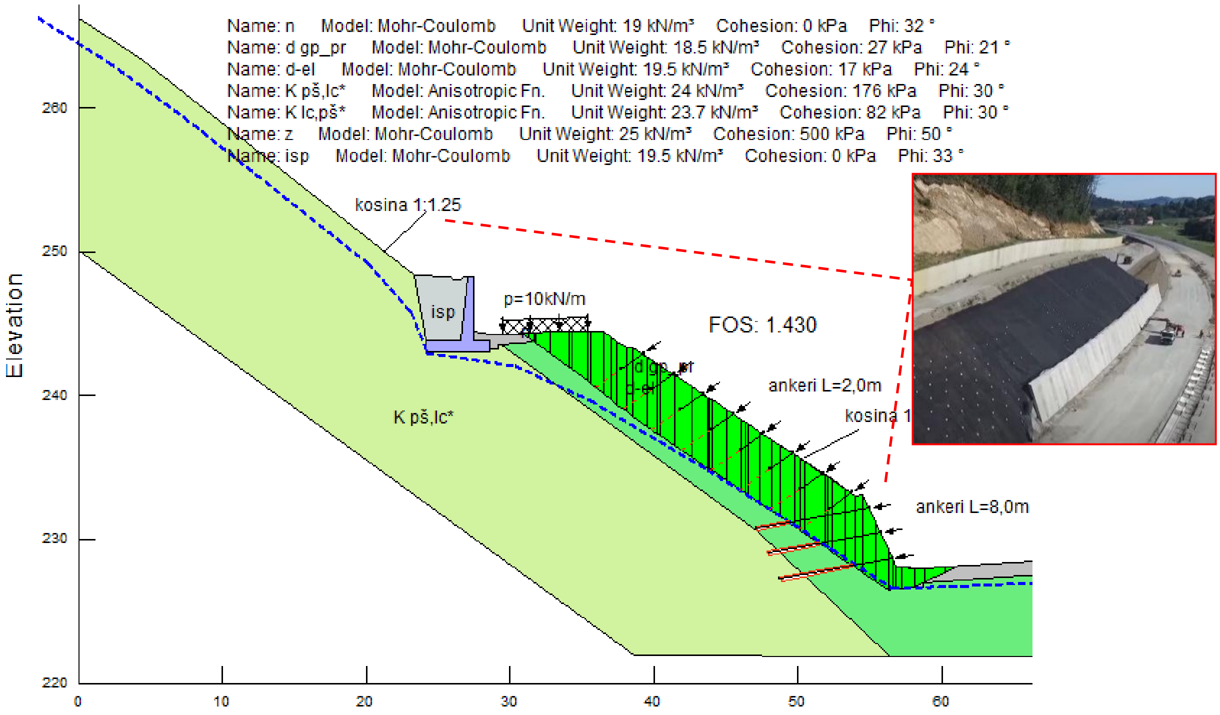


Figure 13. Geostatic calculation of slope stability (in zone 1, see Figure 16) with applied remedial measures

After a series of performed analyses, a solution was given for the stabilization of this part of the cut, which contained the following remedial measures (**Figure 13**):

- Construction of a cut above the regional road at an inclination of 35-38°, which corresponds to the bedding inclination, with facing by double-twisted wire mesh.
- Replacement of the sliding body by embankment, constructed of reinforced soil from a combination of "teramesh" mesh and crushed stone filling (0-63 mm), under the slope of 1:1, making a drainage system to drain the slope, adequate protection of the face of the slope with hydro-seeding.
- For the temporary protection of the slope during the excavation (for the replacement of material in the sliding body and the creation of reinforced soil), passive anchors with shotcrete were used.

The second model (**Figure 14**), with applied remedial measures, represents the central part of the left side of the cut, where diluvial - eluvial sediments slide over the rock mass.

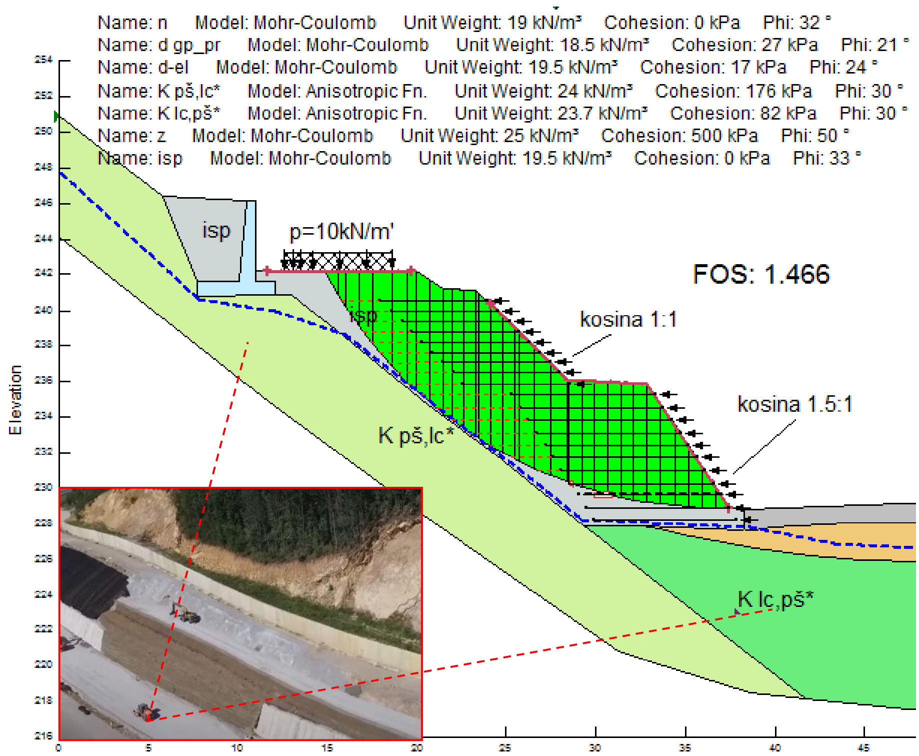


Figure 14. Geostatic calculation of slope stability (in zone 2, see Figure 16) with applied remedial measures

The third model (**Figure 15**) represents the central part of the right side of the cut, where due to the "favourably" oriented bedding "into the rock mass" instability did not occur. After a series of performed analyses, a solution was given for the stabilization of this part of the slope, which contained the following remedial measures:

- Construction of cut in the projected inclination of 1:1.5
- Creation of a drainage channel to drain the slope and adequate protection of the face of the slope with geocomposite and hydro-seeding.

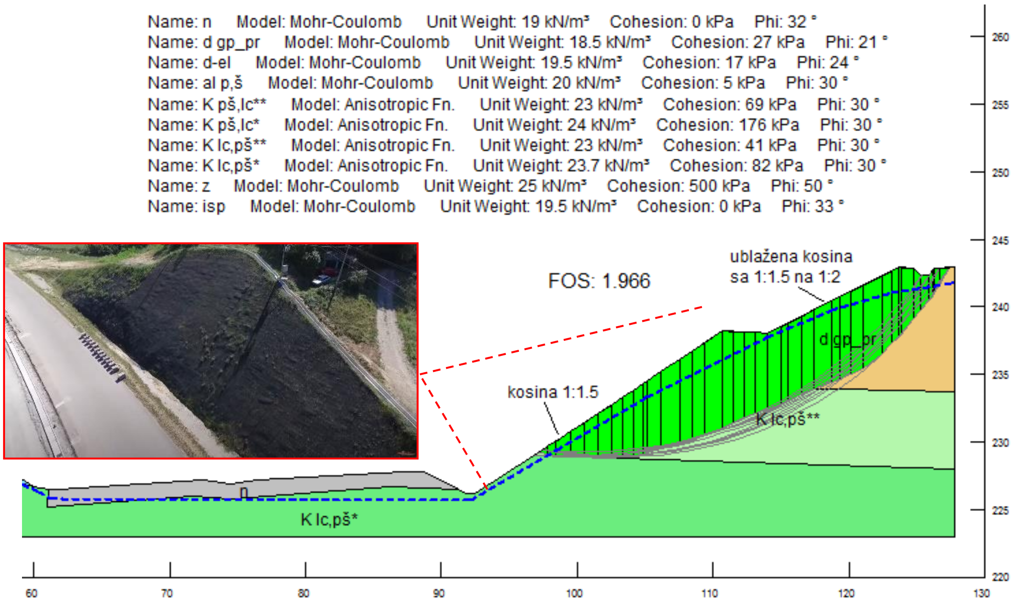


Figure 15. Geostatic calculation of slope stability (in zone 3, see Figure 16) with applied remedial measures

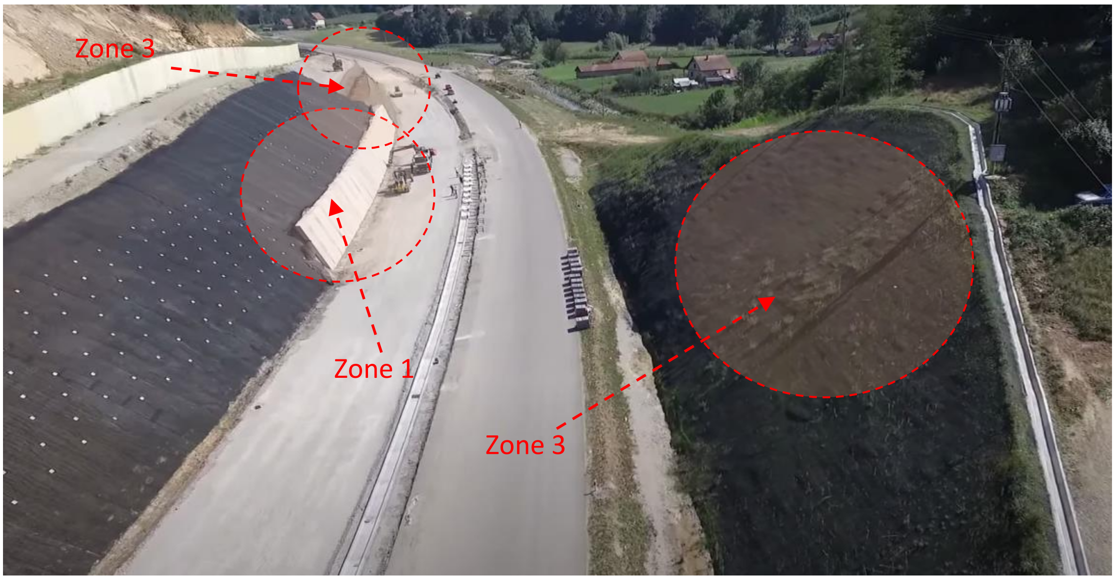


Figure 16. Left and right side of the cut after rehabilitation (with marked position of characteristic zones)

4. CONCLUSION

This paper presents an example from the practice of cut construction in soft rock masses, which are prone to surface disintegration due to atmospheric influences, with a clearly defined rupture structure dominated by bedding. The negative orientation of the bedding, in relation to the excavation, on the left side of the cut caused the appearance of a series of instabilities in the rock mass. On this side of the cut, various geodynamic processes occurred, the most common of which are: the process of surface weathering, planar sliding of rock blocks and sliding of diluvian-eluvial deposits along contact with the rock mass. However, on the right side of the cut, the orientation of bedding had a positive effect on the stability of the cut. To solve this geotechnical problem, field research, laboratory tests and office data analysing were carried out, based on which the representative physical and mechanical parameters of the determined geotechnical units were defined. The parameters were used for geostatic calculations and for the design of remedial measures. In the geostatic analyses, with the application of remedial measures, an anisotropic soil model was used to define the strength of the rock mass with dominant bedding. The strength of the rock mass in the direction of the bedding is represented by the shear strength along the discontinuity, while the strength in other directions is represented by the shear strength of the rock mass. Based on the performed analyses, remedial measures were designed for the construction of this cut. Remedial measures in the form of IBO anchors, subhorizontal drains, drainage channels, facing RC walls, geocomposite, reinforced soil and hydroseeding were necessary to stabilize the left side of the cut. On the right side of the cut, where the bedding is favorably oriented, it was enough to apply remedial measures aimed at preventing the surface disintegration of the soft rock mass under the influence of weathering.

5. REFERENCES

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