

Primljen / Received: 20.6.2014.

Ispravljen / Corrected: 16.1.2015.

Prihvaćen / Accepted: 30.1.2015.

Dostupno online / Available online: 10.2.2015.

Realization and protection of deep flysch excavations in complex geotechnical conditions

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Professional paper

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Realization and protection of deep flysch excavations in complex geotechnical conditions

The excavation technology, and the type and scope of slope protection measures, were selected based on preliminary analyses and investigation results, in specific construction conditions, immediately below some major road facilities, in partly degraded thinly layered volcanogenic sedimentary rocks. The realization and protection of a deep and wide foundation pit was made according to the method presented in this paper, which effectively ensured stability of the existing structures and full safety of work in every phase of excavation, all this under a very tight schedule and at low cost of construction work.

Key words:

slope, stability, excavation, protection, safety, micropiles

Stručni rad

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Izvođenje i osiguranje dubokih iskopa u flišnim stijenama pri složenim geotehničkim uvjetima

U specifičnim uvjetima izvođenja, neposredno ispod objekata magistralne prometnice, u djelomično degradiranim, tanko uslojenim vulkanogeno-sedimentnim stijenama, na osnovu rezultata istraživanja i provedenih analiza, izvršen je izbor tehnologije iskopa, vrste i opsega zaštitnih mjera nužnih za osiguranje stabilnosti kosina. Za izvođenje i osiguranje stabilnosti dubokog i širokog temeljnog iskopa primijenjena je metoda, prikazana u ovom radu, koja je osigurala stabilnost postojećih objekata i sigurnost rada, u svakoj fazi iskopa, i pri tom vrlo kratak rok i nisku cijenu radova.

Ključne riječi:

kosina, stabilnost, iskop, osiguranje, sigurnost, mikropiloti

Fachbericht

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Dürre, standardisierter Niederschlagsindex, Dezil-Methode, Normalprozentsatz, Index der Niederschlagsanomalie, Schrittverfahren

Aufgrund der Ergebnisse durchgeführter Untersuchungen und Analysen sind Aushub-Technologie, sowie Art und Umfang der erforderlichen Maßnahmen zur Baugrubensicherung für spezifische Ausführungsverhältnisse, direkt unter der Konstruktion einer Magistralen-Straße in teilweise abgebautem, dünn-schichtigem Sedimentgestein vulkanischen Ursprungs, bestimmt worden. Die beschriebene Methode ist zur Ausführung und Stabilitätssicherung der tiefen Baugrube angewandt worden, so dass die Stabilität bestehender Objekte sowie eine gefahrlose Arbeitsabwicklung, trotz kurzer Fristen und niedriger Kosten, versichert werden konnten.

Schlüsselwörter:

Hang, Stabilität, Aushub, Sicherung, Sicherheit, Wurzelpfähle

1. Introduction

In practice, the instability of slope that occurs after excavation in rock material is most often due to an inadequate conduct of excavation work. Some slope stabilization measures are applied as preventive measures, before and during excavation work. In some cases, emergency repair measures are applied after manifestation of instability. Measures applied to ensure slope stability, both preventive and intervention-based, are in their constructive meaning essentially identical and are basically concentrated on parts of the structure that should directly withstand the rock mass action (reinforcement mesh, shotcrete, RC ribs, RC panels, or similar elements) and parts of the structure that should transfer these actions or forces into deeper and stable parts of the rock mass in deeper parts of the slope (anchors). The process of remediation is associated with work in areas of increased risk, because if design measures are not carried out, there is a risk of displacement of conditionally stable blocks, or even of unstable zones, when it is impossible or uneconomical to remove them by excavation or mining. On the other hand, the process of slope stabilization is based on the concept of work in stable conditions. Each phase of excavation and application of designed measures is conditioned by previous evidence of stability of the excavation face, and the existing surrounding buildings. In particular, the possibility of performing works in safe conditions at all stages is quite limited when performing stabilization work for excavation in rock. The excavation technology selection, type and scope of measures to ensure the temporary and permanent stability of high slopes is, above all, determined by the slope geometry, rock mass characteristics, terrain morphology, and accessibility of the site where excavation is performed. This paper provides an overview of one such case, with the analysis of all its specific features [1, 2]. The excavation site is shown in the following figures (Figure 1).



Figure 1. Location of the slope and hillside before excavation works [2]

2. Basic geomorphological characteristics of terrain and rock mass properties

The site under study is located just below the route of the Adriatic Motorway (Jadranska magistrala) at the Budva – Petrovac section, in the Rafailovići – Kamenovo zone. In order to improve traffic conditions, this section was reconstructed by motorway widening. During the widening, the semi-construction and retaining wall were located at the edge of the construction lot, where the excavation was planned and then realised.

The wider area of the investigated terrain belongs to the coastal karsts and hilly type of relief. Abrasion forms of relief are manifested as steep rocky sections up to 50 m in height along the southern edge of the lot, and as chamfer-shaped wave notches located at the foot sections [3].

Part of the field at Cape Djevištenje, where the lot is located, is a hillside steeply sloping towards Rafailovići Bay. The lot is at about 35 m above sea level in its south-western part, and reaches up to 55 m above sea level in the south-eastern part. The slope inclination in the area where the excavation is planned varies from 25° to 30°. Locally, along the existing road, in the zone of technogenic activity, the inclination reaches up to 40° due to disposal of surplus excavated rock material [4].

The geological composition of terrain is represented by formations dating back to Mesozoic Triassic age, and by the overlying widespread Quaternary sediments [5].

The volcano-sedimentary series on the location under study is made up of tuffites and tuffs, cherts, marlstones, sandstones, and laminar limestone in the upper parts. All these elements alternate from lower layers upwards, and the most common are tuffites, sandstone, limestone, and marlstone. The entire series is laminar to thin-layered in texture, and in monoclinic dip to the southeast.





Figure 2. Rock mass outcrops [4]

The Triassic carbonate series of sediments lies perpendicularly over volcanogenic-sedimentary series. Jurassic sediments, also perpendicular, lie over this series. At the open cross-section over Cape Djevistenje and the highway, there are clearly separated laminated to thin-layered limestone formations with cherts of reddish colour in the lower part of the series and, in the upper parts, there are banked limestone formations with intermittent layered appearance of cherts, falling toward the southeast at an angle of 25° to 30° (Figure 2.).

A thinner zone of eluvial deposits is situated in the subsurface zone, above the basic fresh rock mass. Below it, the rock mass is intensely fissured and partially altered by physicochemical processes, i.e. by surface alteration processes. The boundary between the basic and the partly altered rock mass is gradual and difficult to notice. Delluvial-eluvial formations and unplanned deposits of rock material from the excavation, construction and recent reconstruction of the motorway, are represented as a continuous sheet on the slope, above the basic rock mass. They consist of clayed silty-sandy deposits with parts of basic stronger rocks, cherts, limestones, and marlstones. Due to the relatively small thickness, they do not have greater practical significance in terms of slope protection, but their presence must be taken into account.

The interlayer surfaces are the best expressed mechanical discontinuities, and the largest and most visible joints are found at the outcrops. Besides interlayer surfaces, two sets of joints are clearly shown at the contour diagram (Figure 3). These sets differ greatly from one another in terms of direction, while difference in terms of gradient is slight [4]. Also, their propagation is different and, as a rule, they are followed with individual thicker or several thin layers. They are most commonly compressed, unfilled, and the walls are even [6]. Tension cracking probably occurred in the process of deposition and diagenesis of sediments. However, bearing in mind the frontal slope azimuth, these cracks are significant for slope stability [7-9].

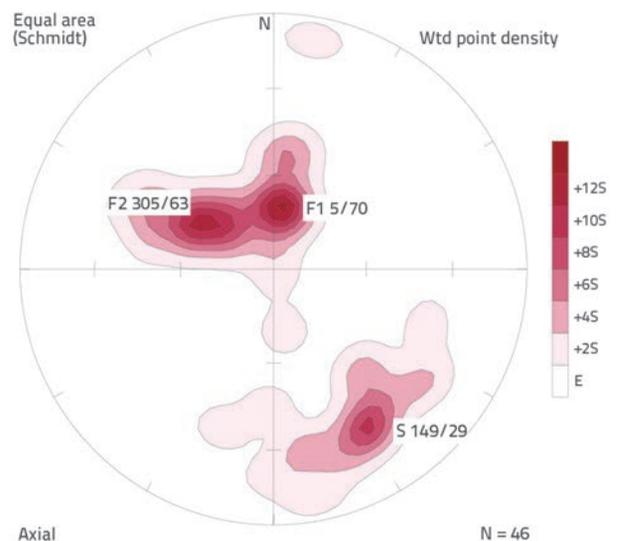


Figure 3. Contour diagram of joints [4]

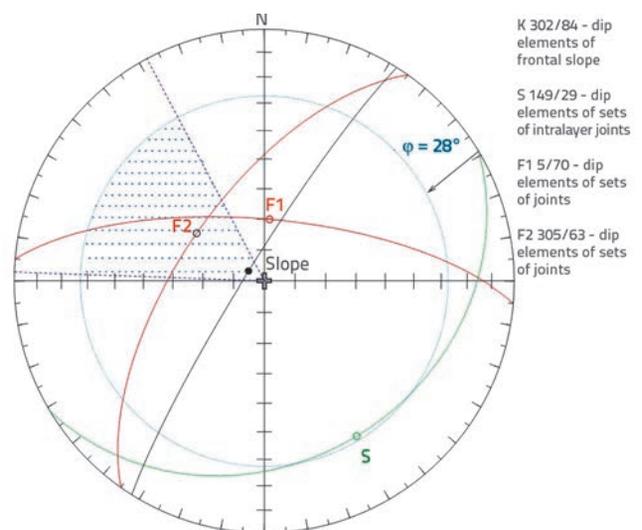


Figure 4. Analysis of kinematic features along shear discontinuities on front slope of the foundation pit [4]

Table 1. Physicomechanical characteristics for geostatic calculation [4]

Lithological part	Parameter	Bulk density γ [kN/m ³]	Friction angle ϕ [°]	Cohesion c [MPa]	Uniaxial strength σ_{ci} [MPa]	GSI	m_i
GT-1. Delluvial-eluvial clay deposits		20.5	25	0.020	-	-	-
GT-2. Limestones and marlstone		25.5	46 30*	0.180 0.045*	32	35	7
GT-3. Tuffites, marlstones and cherts		24	39 28*	0.085 0.035*	18	20	9

Note: * Shear strength values along joints

The tension cracks at one set are approximately parallel to the future slope and, according to the Panetta criteria [7], the kinematic slip may occur along them (figure 4). On parts of the slopes along the highway, the slipping had been occurring locally and periodically, i.e. the breakout and tumble of smaller rock mass blocks was registered, and so this phenomenon can be expected at the slope excavation front, which is parallel to the slope.

In addition to the described sets of joints, some other random joints that divide the rock mass into variable-size monoliths were also registered. Monolith sizes vary in range from several cm³ in extensively cracked areas and in laminated to thin layers, to several dcm³, in the layered pars of limestone.

In the area of construction activity, the rock mass is anhydrite, and the related groundwater aquifers are deep, at sea level (10). When geotechnical model was prepared for the slope stability and stress-strain analysis, the parameters relevant for geostatic calculation were adopted for three lithological parts:

GT-1. *Delluvial-eluvial cover*, clayey sediments and clayey debris from parent rocks.

GT-2. Carbonate flysch-like complex – pavement to layered limestone with cherts, with layers of pavement marlstone. Interlayer cracks are clearly visible, and usually with marl-clay infills up to 5 mm in thickness.

GT -3. Silicate-marly flysch-like complex-tuffites, with layers of sandstone and marlstone. A whole series of thinly layered to pavement, and lithological members are irregularly alternate.

The choice of parameters of physical and mechanical characteristics relevant for geostatic calculations presented in Table 1 was made on the basis of:

- laboratory test results for soil and rock samples, ISRM recommendations [11, 12] from the studied location and from nearby locations [13].
- assessment of intact characteristics of the rock mass, scaled in conjunction zone with the construction, based on its lithological composition, degree of alteration, degree of scaling, orientation and characteristics of joints, [6, 14, 15],
- using the RocLab software [16].

The average value of the related strength parameters determined by laboratory testing of soil samples was adopted for Delluvial

eluvial formations. The friction angle $\phi = 25^\circ$ and cohesion $c = 20$ kN/m² were adopted for the Mohr-Coulomb strength criterion. Parameters for flysch-like rocks of carbonate and silicate-marly complexes were adopted for the Hoek-Brown strength criterion. According to Marinos and Hoek [15], carbonate flysch rocks, limestone, and marlstone, are classified into group C, while silicate-marly rocks, sandstones, and tuffites are classified into group E. The calculation value of GSI = 35 was adopted for carbonate flysch rocks, while the value of GSI = 20 was adopted for flysch-like silicate-marly rocks. It was established by laboratory testing of samples that the average uniaxial strength of limestones and sandstones amounts to 50.9 MPa and 28.2 MPa, respectively. Testing was not conducted on other lithological samples. Although limestones and sandstones are the firmest lithological members within the allocated flysch-like complex, smaller uniaxial strength values were adopted for calculations, in order to represent the complex as a whole, in accordance with the recommendations made by Marinos and Hoek [15]. The values of $\sigma_{ci} = 32$ MPa and $\sigma_{ci} = 18$ MPa were adopted for the calculation of carbonate flysch rocks and silicate-marly rocks, respectively. The value of $m_i = 7$ was adopted for flysch rocks in carbonate complex, which corresponds to micritic limestone and marlstone. The value of $m_i = 9$ was adopted for flysch-like silicate-marly rocks, which corresponds to tuffites. The slope damage factor of $D = 0$ was adopted, because mechanical excavation of the underlying slope is expected, and no damage to the rock mass in the hinterland of the slope is anticipated.

For the slope stability calculation purposes, shear strength parameters were also determined by testing direct shear along the cracks. Average parameters of peak shear strength along joints, resulting from laboratory testing, are shown in Table 1. Similar shear strength values were obtained by preliminary feedback analysis of slope stability for blocks sliding from slope near the motorway (which occurred near the studied location).

The Mohr Coulomb strength parameters for lithological units GT-2 and GT-3, shown in Table 1, were used in the analysis of slope stability, for the case of shear through the rock mass. They were calculated using the RocLab software, based on the input data defining the Hoek-Brown strength parameters.

Table 2. Safety factors for characteristic cross-sections

Safety factors for possible cases	Safety factor for block sliding along the joints		
	Slope in natural conditions	Slope protected with external force of $n \times t/m'$	Protected slope with seismic impact
Cross-section A – in the semi-structure piling zone	0.43	1.50 for 154 t/m'	1.31
Cross-section B – in the substation zone	0.78	1.50 for 51 t/m'	1.33

3. Definition of geotechnical conditions for excavation work

The structure construction was planned under the existing road on a plateau at an approximate altitude of 39 meters above sea level. Cutting up the slope 85 m in length, and from 10 to 30 meters in width, was planned so as to form the plateau. In order to make maximum use of the available land area, the construction was planned as close to the motorway as possible, so as to liberate maximum of space for amenities between the construction site and the sea. Respecting the above requirement, a sub-vertical slope with front inclination 10:1 ($\approx 84^\circ$) and about 6 m in height was planned in the northern part, and 18m in the south part for excavation of the foundation pit.

The excavation had to be performed below existing structures, substations, semi-structures, and the retaining wall of the highway; however, it was important not to compromise their safety during and after completion of the excavation work, and during construction of the planned facilities on the lot. The terrain is steep, narrow and inaccessible and is situated in the zone of heavy traffic flow, so it was almost impossible to use heavy machinery.

Based on the in situ measurement of inclination of layers and joints, statistical analysis of rupture elements was performed in order to assess stability of the foundation pit slope. Based on these results, and the geometric elements of the designed slopes, appropriate analyses were conducted, and possibilities for kinematic shear along the joint were analysed according to the suggestions and recommendations made by Panet [7], Hoek and Bray [8], and Yoon, Jeong and Kim [9]. The stability of the future slope with potentially unstable blocks was discussed taking into account the fact that the shear along the crack does not always occur just because of kinematic possibility, and that boundary conditions of equilibrium must also be considered.

Geostatic and pseudostatic analyses were conducted to check stability of the foundation pit slope. These analyses were made

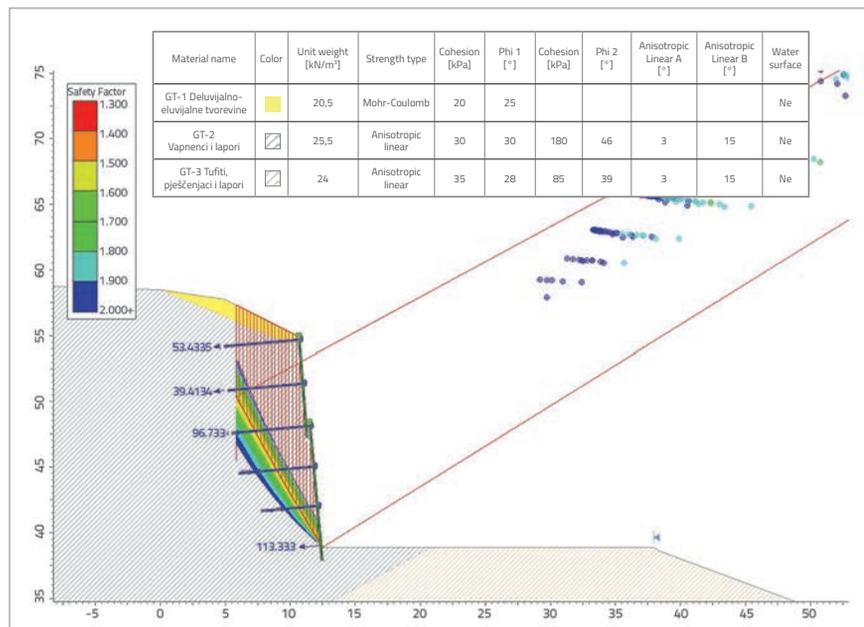


Figure 5. Geotechnical slope model for stability analysis, with security measures at profile B

according to traditional schemes of block sliding along one or more joints, using the RocPlane software [17]. Probability analyses were used to test shear along existing cracks at an unfavorable angle of 63° , being the most unfavorable position for tension cracks. The distance of such positions from the slopes and gradients was determined. Calculations were performed for these cases, and it was established that the results are on the side of technical safety. The stability at cross-section in the zone of the semi-structure (A), including the additional load from the semi-structure and traffic, was checked by calculations. The stability calculations for the worst case scenarios were also performed for the maximum height cross-section of the excavation (B) in the substation zone. The seismic impact was simulated, with an equivalent static load, and the horizontal force of $0.1 W$ (weight of the sliding block), according to the seismic micro-zoning data [18] and as recommended by Seed [19], and Hynes-Griffin and Franklin [20], and in accordance with current regulations [21]. Calculation results for slope stability without security measures, and the calculated force that should provide necessary safety factors for characteristic cross-sections given in Figure 8, are shown in Table 2.

The conduct of stabilization measures should result in the minimum safety factor of 1.30 with the impact of seismic activity, or in the minimum safety factor of 1.50 if the seismic data are not taken into account in the analysis. Considering the size of calculated forces and techno-economic reasons, the construction of micro piles, shotcrete, anchors, and tie beams was envisaged for the primary slope protection, as shown in Figure 8. Control calculations of slope stability with security measures were made using the software package Slide [22] and limit equilibrium methods (Bishop and Janbu). Slope stability calculations were carried out for anisotropic environment in order to simulate rock masses properties as closely as possible. It was presumed that the sliding plane will form partly along the existing fissures, and partly through the rock mass. Shear strength values along the cracks were adopted for the plane, which is inclined at 63°. The calculation scheme for the most unfavorable profile B is shown in Figure 5, where the sliding block with the minimum safety factor is given (Bishop $F_s = 1.52$ and $F_s = 1.49$ per Janbu).

4. Review of various slope stabilization solutions

The following alternative solutions for temporary and permanent protection of slope in excavation, with structures in the background, were analysed:

- first solution: excavation in stages, during which the excavation would be unprotected until the temporary altitude is reached, with stabilisation measures involving a retaining wall construction, or construction of shotcrete RC ribs with connecting beams and anchors,
- second solution: primary protection of excavation by reinforced concrete piles or diaphragm,
- third solution: primary protection of excavation with reinforced concrete piles or diaphragm anchored deeper into the rock mass,

- fourth solution: primary protection of excavation with the structure composed of micro piles, shotcrete, anchors and connecting beams.

Based on geostatic calculations and techno-economic analyses, the fourth solution was adopted, because others proved to be technically and economically unjustified both in terms of completion deadlines, and total price. More specifically:

- the first solution was rejected because in the actual circumstances, considering the low safety coefficient of the unprotected slope, and possible height and width, the working area would be far smaller and the excavation work far more extensive, compared to the adopted version, which would directly affect completion time and costs,
- the second solution was rejected as technically uneconomical and more expensive compared to other alternatives. Considering the narrowness and inaccessibility of the terrain, as well as the impossibility to obtain authorization for occasional interruption of traffic, it would have been difficult to realize,
- the third solution was rejected as being more expensive than the adopted solution, although structurally more economical compared to the second solution, and somewhat easier to perform from the technical point of view.

5. Review of micro pile construction

The idea of the adopted solution, involving construction of micro piles, connecting beams, anchors and shotcrete, is based on producing the necessary space with controlled excavation of rock material, without the use of explosives and with minimum dynamic effects, especially in the foundation zone of the existing semi-structure.

Specifically, it was anticipated to carry out the excavation with successive protection of the newly formed slope, and this by primarily protecting it with pipe anchors - micro-piles,



Figure 6. Installation of micro piles around the edge of the future excavation (left) and realisation of pile cap (right) [28]



Figure 7. Layout and excavation detail at half the projected depth [28]

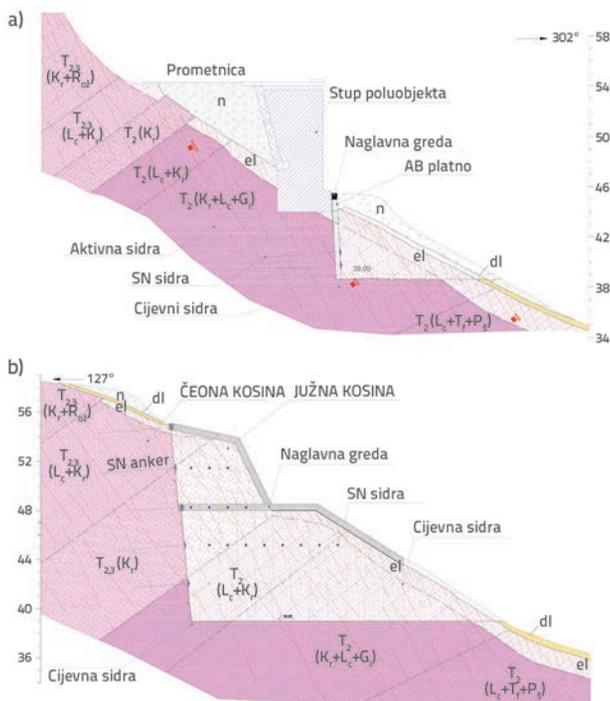


Figure 8. Protection measures at specific cross-sections, excavations A and B [1]

reinforced concrete beams, and passive anchors. Further or secondary protection was anticipated during the excavation work, by applying shotcrete with reinforcing mesh. According to results shown in the table, the semi-structure with load significantly affects the instability of excavation in the cross-section A. It was therefore envisioned to construct the reinforced concrete wall panel and active pre-stressed anchors in foundation zone of the semi-structure. The dimensioning of primary remedial measures (pipe anchors - micro piles, passive and active anchors, and reinforced concrete beams and panels) was made with the objective of accepting all geostatic impacts that occur during excavation of the foundation pit, while secondary measures (reinforcement mesh and shotcrete) were aimed at protecting the newly-formed excavation front against future exogenous influences. In order to prevent aggressive action from atmospheric water and groundwater, pre-stressed anchors were electrolysed, passive anchors were galvanized, and the sulphate resistant cement was used for shotcrete and concrete. The first phase of the works involved construction of the primary protection system, namely by installing pipe anchors - micro piles at the edge of the designed foundation pit. Pipe anchors were placed by installing steel pipes 60.3 / 52.3 mm in diameter, made of steel grade Č0361, into the previously prepared borehole approximately 12 m in depth (Figure 6).



Figure 9. Detail of micro piles excavation and RC panel under semi-structure foundation [28]

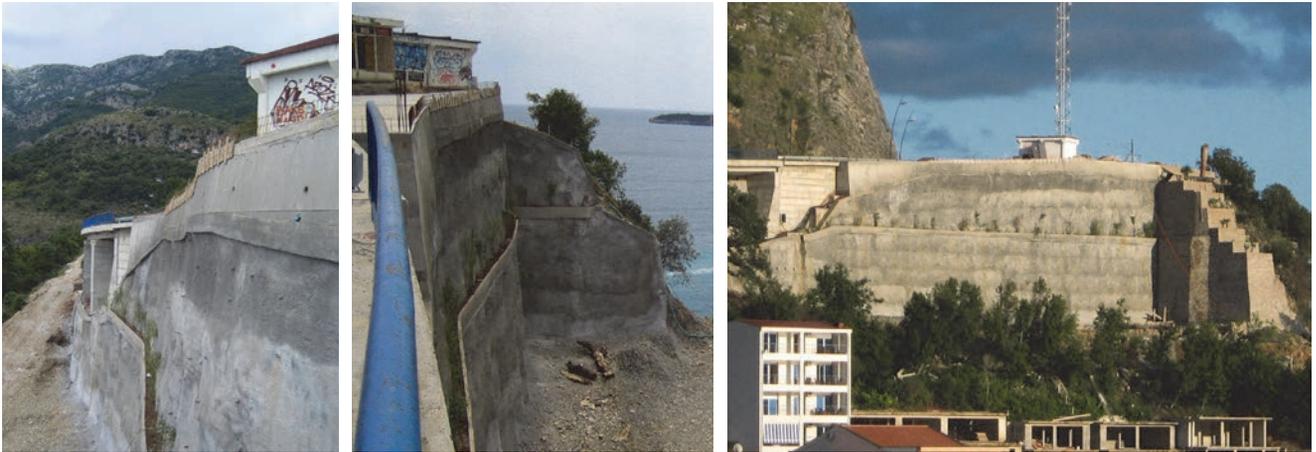


Figure 10. View of protected excavation and detail of southern part of excavation [28] Figure 11. View of stabilized excavation after implementation of design measures [28]

The second phase of works involved excavation of rock material without major dynamic effects – shocks, especially near the piles of the semi-structure and near the new support structure (Figure 7). As the excavation was planned in the material categories III to V (according to GN 200 classification), the design called for the use of hydraulic hammers mounted on the backhoes, while in areas close to sensitive structural elements the design did not allow any dynamic effects, and required excavation by tiller, or manual excavation by pickaxes. It was anticipated that, successively with the conduct of the works of the second phase, and release of the pipe anchors, their protection was approached in the first step by lying the upper-connected pile cap and lateral passive anchors RA Ø

36 mm (Figure 6). At this stage, greater length of the working front provided the opportunity for cyclical execution of concrete works, realisation of AB beams, and placing of shotcrete (Figures 7 and 9). Works on the side of passive anchors were carried out from the altitude of the excavation towards the slopes, so that the slope can quickly be protected. In the zone of the maximum height of excavation, at the foot of the substation, a smaller berm was designed in form of a reinforced concrete beam (Figure 10.). This berm increased stability of the slope and also allowed installation of pipe anchors in two parts, thus avoiding larger borehole deflections and their unwanted entry into the excavation zone, i.e. reduction of the construction lot area.

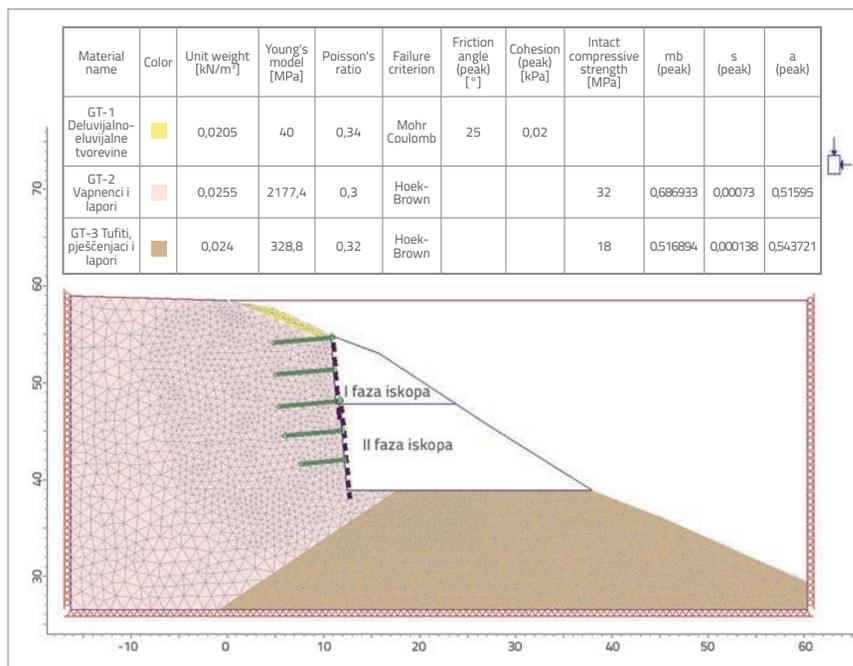


Figure 12. Geotechnical slope model for stress-strain analysis, with security measures at profile B

6. Geotechnical observation, measurements and testing

The design and realisation of construction to ensure stability of slopes in heterogeneous rock masses such as flysch, was approached very cautiously, using methodology similar to that proposed by Bruncic, Juric-Kacunic and Kovacevic [23]. Due to the complex geotechnical conditions at the site surrounded by existing buildings, and lying along the main road, it was indispensable to ensure permanent supervision of the works. The results of geotechnical observations and measurements, which were performed continuously during excavation and slope protection works, similarly as stated by Arbanas [24], were used to check:

- assumptions adopted during the planning and design,
- behaviour of the support construction
- surrounding rock mass system,
- quality control of the support system.

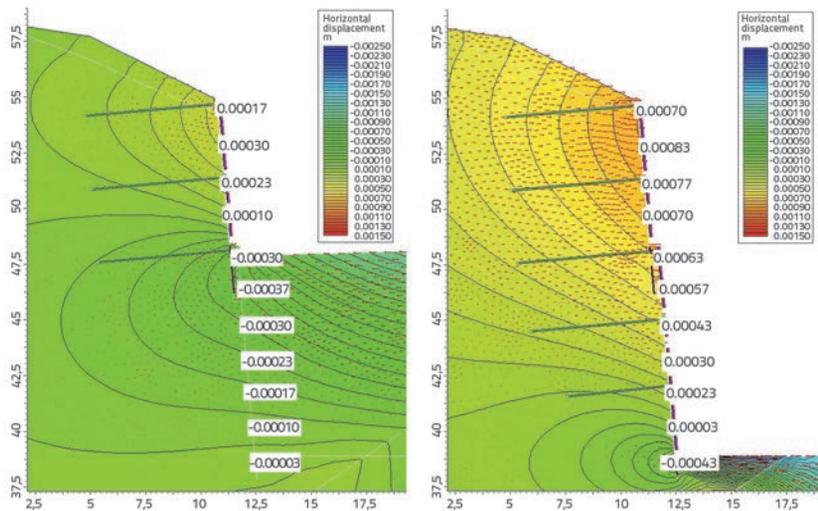


Figure 13. Calculated values of movement in horizontal direction after the first and second phases of slope excavation

The support structure was verified during the design, for all phases of excavation and support work, using the software package Phase2 [25]. It allows numerical analysis on 2D models. The calculation model for profile B, boundary conditions and calculation parameters, stages of excavation, and support structure elements are shown in Figure 12.

Calculation results indicated that stresses due to excavation propagated in the horizontal direction around the entire excavation, but not deep into the rock mass. Very pronounced redistribution of stress would cause lamination (latent fracture) in the surrounding rock mass. The plastic zone will not be significant (about 1 m in thickness in foot zones) during the phase I excavation, and after the final excavation of the slope.

The results obtained by calculating forces in anchors indicate that the forces are substantial within the expected plastic zone, but that the anchor load values will not be exceeded because the expected force varies from 0.05 to 0.1 MN. The results obtained by calculating normal forces and moments in shotcrete and micro piles show that they are under the limit value. They are most pronounced in the horizontal bond beam zone, in the middle of the slope, and at the foot of the slope.

Maximal displacements of about 0.3 mm are expected during the first phase of excavation, and about 0.9 mm after the complete excavation in the slope. The calculated values of movements in horizontal direction, for the first and the second phase of slope excavation, are shown in Figure 13.

The observation program included typical points within the geodetic benchmark network, at nine geodetic control profiles, set successively with the performance of the construction, at pile caps and the existing semi-structure. In order to test the quality of the support system, a special attention was paid to testing quality of the shotcrete and grout for anchors, and to testing anchor resistance to the pulling force. Laboratory

testing of the uniaxial compressive strength σ_p of grout was carried out according to *JUS U.M.8.022. 1984*, while the testing for shotcrete was conducted as per *JUS U.M.8.022. 1984*. The passive anchor capacity testing was conducted as recommended by ISRM [26], while long pre-stressed anchors were tested in accordance with SIA V191 [27].

The geodetic observation of benchmarks revealed displacement of ± 2 mm, which is within the domain of the measurement tolerance. If the measured displacements are accepted as true, they are about 2 to 3 times higher than those calculated in the stress-strain analysis. Values of more than 30 and 35 MPa were obtained during the uniaxial strength testing for grout and shotcrete. These values are slightly greater than the values set in the

construction plan and design [1]. Checking the capacity of five passive anchors revealed that the pulling force is greater than that specified in the design (200 kN). Also, test results for two active pre-stressed anchors show that they meet the criteria in terms of creep, and the coefficient $k < 1$ by Standard SIA V 191. Since the results of geotechnical observations, measurements and testing were compliant with the assumptions adopted at the design stage, it was established that the excavation and protection of pit slope was performed in full accordance with the detailed plan.

7. Conclusion

The works were completed in accordance with the abovementioned design in early 2008 (Figure 11.), and the validity of the adopted solution, time schedule, and cost plans, was confirmed. During the actual work, the equipment and technology applied on the project enabled construction of structures with minimum overhead of construction lots, in keeping with operational safety and traffic safety requirements on this otherwise quite crowded road. The stability of surrounding structures was also preserved.

The presented protection method for the deep and wide foundation pit can be applied on various other locations along the Adriatic coast, in old city cores in restricted working conditions, or along edges of urban communities, in situations similar to the location shown in this paper, especially in terms of geomorphological characteristics of the terrain, and rock mass properties.

Acknowledgments

The research was supported by the Ministry of Science and Technology of the Republic of Serbia, Project No TR 36009.

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