Designing, Constructing and Monitoring of Slopes in Rock Mass in Croatia

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ABSTRACT: This paper gives the review of investigation works, designing and monitoring during the construction, as well as the types of slope protection on the cuts in limestone rock mass in Croatia. For demands of the higher cuts design, detailed geotechnical investigation works are maintained, and they consist of research drilling, geophysical investigations by the method of shallow refraction, engineering geological mapping, laboratory testing etc. At cut design, stability analysis for the failure through the rock mass, plane and wedge failure and stress–strain analysis for the particular cuts as the first design stage are implemented. At the second design stage during construction, by observational method, detailed monitoring of rock mass behavior is maintained, including geotechnical supervision and monitoring of installed inclinometers and deformeters. Based on given data out of measurement devices and engineering geological mapping of the cuts, it is possible to change factor of safety adopt adequate support systems. During the construction of geotechnical structures in rock mass, design process starts with investigation works, and is carried out over the analyses and implementation of the main design, monitoring during the construction and changes of the support systems that are included in the final design.

1 INSTRODUCTION

In the last few years more than 400 km of highways were constructed in Croatia, within cuts in limestone rock masses. Motorways are mostly situated at high-handed region and are fitting to existing terrain by tunnels, viaducts, bridges and cuts. Mainly, high-handed region of Croatia, through motorways are passing, part of carbonate massif with uniform lithographic composition. This rock massif is usually made of limestone rock mass and limestone breccias from Upper Cretaceous and Jurassic.

Limestone rock mass in Croatia belong to group of stronghold sediment rock masses. Uniaxial compressive strength of this intact rock mass, which is target of this paper, are from 75 MPa to 150 MPa. The typical geotechnical rock mass profile of the terrain is divided into two main zones: weathered zone and bedrock. Contact between these two zones is very irregular and hard to perceive, particularly at stage of geotechnical investigation works. Depth of weathered zone is sometimes negligible but sometimes is greater than ten meters. The weathered zone is usually more weathered and fractured than bedrock. The RQD factor, as indicator of fractured rock, is very low and its value is from 0% to 15%. The discontinuities at this zone has grater closure, often they filled with clay or washed out by water. Estimated geological strength index is between 20 and 40 and increase with depth. Crossing from weathered zone to bedrock RQD indexes are increasing (up to 100%), the closure of discontinuities of is reduced or totally closed, the fill of discontinuities is hard (calcareous) or is not present. GSI is also grater with depth, the values vary from 30 to 60) and the discontinuities persistence is greater and increasing.

Cuts in slopes at motorways are excavated up to 50 m of high. Cuts are excavated in stages of 8 to 10 m high, with slopes of 2:1 to 3:1 and the upper floor is excavated in slope of 1:1 because of presence of the weathered zone near the surface. The excavation is executed in stages after which are reinforced and protected if it is necessary. After reinforcing the cuts the excavation is likewise resumed.

2 GEOTECHNICAL INVESTIGATION WORKS AND STABILITY ANALYSES OF CUTS

Detail geotechnical investigation works are part of geotechnical design of cuts. The purpose of these investigation works is to gather as much as possible characteristics of rock mass: input data for design stability of cuts in rock mass, material categorization, recommended slopes of cuts, estimating amount of protection measures of cuts. Investigation works are adapting to the terrain, types of rocks, the high of the cut etc. As most part of the motorway routes are passing through the high-handed terrain and slope of the route is maximum 6%, it is necessary to build objects and to excavate high cuts to adopt the motorway routes to the surrounded terrain. Investigation works of these cuts are customized to limestone rock mass.

Investigation works are consisting of exploration drilling, geophysics testing (refraction and reflection), laboratory testing (uniaxial compression and triaxial compression tests, point load tests) and engineering geological mapping of the terrain.

Engineering-geological mapping of the terrain is focusing on determines input data for stability analysis: elements for Rock Mass Rating (RMR) classification, Geological Strength Index classification, distance of discontinuities, persistence of discontinuities, closure of discontinuities, roughness of discontinuities according to Joint Roughness Index (JRC_{10}), weathering of discontinuities surfaces, strength of discontinuities according to Joint Compression Strength (JCS), direction and dip of discontinuities. During analyzing the input engineering-geological data blocks with same or similar characteristics are sorted and selected (engineering-geological blocks).

Figure 1. Bedding of rock mass dipped to the excavation face

At some cuts bedding of the rock mass is dipped to the excavation face at angle from 35^o to 55^o degrees – Figure 1. At this case it is possible to occur plane failure so the engineering-geological mapping is focused on getting input data for analyzing the strength of discontinuities: distance of discontinuities, persistence of discontinuities, closure of discontinuities, roughness of (JRC_{10}), weathering of discontinuities surfaces, strength of discontinuities (JCS), direction and dip of discontinuities. Based on these information gathered by engineering-geological mapping the strength laws of discontinuities are determined and stability analysis for wedge failure and plane failure are carried out.

Exploration drilling with core sampling was used according to METRIC standard. Ending drilling profile in rock mass was ϕ86 mm. The core was deposited in wooden cases with length of 1.0 m, than it was photographed with corresponding markers of investigation bore and level of core samples – Figure 2.

Figure 2. Rock mass core from exploration drilling

Geophysical investigation methods are chosen based on the geological structure of the site and investigations were determining the structural constitution and valuating the quality of the rock mass. For that purpose seismic refraction method was applied in investigations at small depths. It is possible to determine and evaluate the depth and the configuration of the bedrock, lateral contacts in the bedrock, vertical cuts of the materials and rocks along the given profiles, the positions of the fault tracings and fault zones, the degree of fissures and the rock mass quality, from results of geophysical investigations, and based on spatial arrangement of the velocity of compressive seismic waves, spacing of electrical resistivity and the reflection of electromagnet waves.

The special attention was given to detecting and seeking the poor, more fractional and ruinous zones, cracks systems and faults in bed rock.

The investigation program was performed on the specimens of the intact rock material from chosen core breach. Laboratory tests on the carbonate rock mass were mostly directed on testing of rock mass, and they consisted of uniaxial and triaxial rock mass strength, ultrasound and PLT (Point Load Test) testing.

Seismologic and seism tectonic investigations were also carried out, directed to determine the recent structural relations and seism tectonic activity, and particularly of the seismic parameters for bigger objects and cuts. Because of the expressed seism tectonic activity, seismologic and seism tectonic data from the regional and local area were considered, and the values of the ground oscillations acceleration due to earthquake on the level of the bed rock and the base level of the macro seismic intensity were gained.

Designing of cuts in rock mass concerns finding the optimum geometry of the cut, and if necessary determining the measures of support. For greater cut heights, cuts are performed in levels of 0.8 to 10.0 m height, in inclination from 2:1 to 3:1. On the top of the level berm of 4.0 m wide is performed, and the slope at the top is performed in inclination 1:1. Global stability control is carried out for the failure trough the quasi homogeneous rock mass and planar failure. The failure type presents the macroscopic description of conditions in that the failure appears (Hoek and Pentz, 1968). At the same time the planar failure, wedged failure, rotation failure, and toppling failure are distinguished.

3 ROCK MASS INSTABILITIES TYPES

According to former experience in calcareous rock masses, appearance of instabilities can be divided on the failure through the quasi homogeneous rock mass, planar failure and the wedged failure. In resolving slope stability problems the base problem is assessment of the rock mass strength principle. Depending on the failure type the corresponding rock mass strength principle is used for the stability control: for failure through quasi homogeneous rock mass Hoek - Brown criteria of failure is used, and for failure planar failure and the wedged failure Barton criteria of discontinuity strength is used.

In stability analysis for failure through the quasi homogeneous rock mass Hoek – Brown criterion for rock mass failure is usually applied, close to the strength parameters assessment based on the rock mass classification. The first Hoek – Brown failure criterion was suggested in 1980, and was based on data gained from the triaxial testing of rock mass. Up today original failure criterion experienced more changes and adjustments, and Hoek, Carranza-Torres and Corkum introduced the new failure criterion in 2002 – Equation 1.

$$\sigma'_1 = \sigma'_3 + \sigma_c \left(m_b \frac{\sigma'_3}{\sigma_c} + s \right)^a \qquad (1)$$

where m, s = constants depending on rock mass characteristics; σ_c = uniaxial compressive strength of the intact rock; σ_1 = major main stress at failure; and σ_3 = minor main stress at failure.

For values of parameters m_b, s and a Hoek, Kaiser and Bawden (Hoek et al., 1995) suggest expressions in dependence on the value of Geological Strength Index (GSI). The area of Hoek – Brown criterion application is restricted. The criterion can be used when more than two discontinuity systems on the observed problem exist.

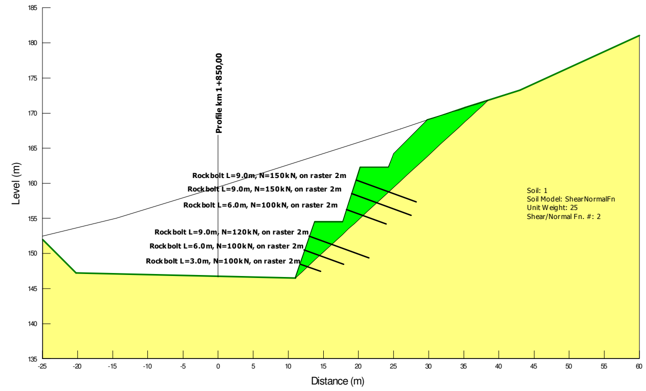

Figure 3. Analysis of planar failure stability

In the slope stability calculation for planar failure and the wedged failure, shear strength of rock mass is dominant, and it presents the function of discontinuity strength and the paths in intact rock mass that separate the discontinuities (Arbanas, 2002.). Barton and Chouby (Barton and Chouby, 1977), Barton and Bandis (Barton and Bandis, 1990) and Bandis (Bandis, 1992) developed the empirical criterion for shear strength of discontinuities which includes the discontinuity area roughness and the compressive strength of the discontinuity wall. Barton's nonlinear criterion for shear strength is:

$$\tau = \sigma_n \tan\left[JRC \log_{10}\left(\frac{JCS}{\sigma_n} \right) + \phi_b \right] \qquad (2)$$

where JRC = Joint Roughness Coefficient; ϕ_b = basic friction angle of the surface, σ_n = normal stress on surface and JCS = Joint Wall Compressive Strength. The example of planar failure analysis and slope reinforcement is shown at Figure 3 and 4.

Figure 4. Slope protection against planar failure

4 SLOPE PROTECTION TYPES

For slope cut protection more types of support systems are using depending on the type of possible instability and failure. Designing the cuts in rock mass is reduced to choosing the stable geometry of slope and slope inclination in combination with application of appropriate support measures. Also it is necessary to consider the possibility of modifications that would result in stability improvement and optimization of the designed solution. (Windsor and Thompson, 1992).

Depending on the field condition and the failure mechanisms in the rock mass, slope protection types could be divided in:
- protection with double twisted galvanized wire meshes,
- protection with galvanized wire meshes reinforced with rockbolts and steel ropes,
- protection with high load-bearing meshes reinforced with rockbolts and
- systematic slope supporting with rockbolts and shotcrete.

First three types of protection are mostly used to ensure local slope stability (erosion stability, detaching and falling of small stone blocks, smaller wedge failure etc.) while systematic slope support is used to secure the global stability of slopes (failure through rock mass, planar failure or bigger wedge failure).

4.1 *Double twisted galvanized wire mesh*

Double twisted galvanized wire meshes are produced in hexagonal shape that provides better and proper tension distribution in the mesh wire. The mesh consists of wire (commonly 3.0 mm thick) that is double twisted and bended to form openings according to production methods (80 x 100 mm or 60 x 80 mm). The mesh is straightened laterally on the connecting point with wire that has higher tensile strength and profile (commonly 3.9 mm thick). Double twisting system localizes any kind of mesh damage and it prevents expansion and distribution of eventual mesh damage resulted from fracture inside the mesh.

Double twisted mesh is used for local slope stabilization (smaller rockfall protection, protection from smaller stone blocks), for building retaining structures (gabions, sack gabions, prefilled gabions and others), for building of reinforced structures (teramesh and green terramesh system) and for reinforcing asphalt (steelgrid) – Figure 5.

This type of mesh is produced under highly controlled conditions; it has to have an exceptionally high strength, high resistance to negative weather influences and chemically aggressive substances, temperature changes, corrosion etc. If necessary the mesh can be protected with PVC covering (polyacrilic and similar). Most commonly used in areas where works are conducted near the water or in case of extremely adverse climatic conditions that could influence negatively on the steel structure.

Figure 5. Slope protection with double twisted galvanized wire meshes

4.2 *Galvanized meshes reinforced with rockbolts and steel wire ropes*

In cases requiring higher tensile strength in meshes double twisted meshes reinforced with anchors and steep ropes are used. Double twisted knitted mesh is installed on earlier drilled rockbolts and on the anchors heads especially steel spike plates are installed to anchoring the steel ropes – Figure 6. Plates are tightened to rockbolts heads with nuts and afterwards the whole area of the slope is interlaced with steel ropes placed in adequate orientation. Steel rope diameter varies from 12.0 to 16.0 mm thickness.

Figure 6. Slope protection with galvanized meshes reinforced with rockbolts and steel wire ropes

4.3 *High load-bearing meshes reinforced with rockbolts*

When the rock mass is extremely fractured and high tensile capacity of the mash is needed because of the stone blocks sizes and masses a load bearing mesh is applied to the rock mass reinforced with rockbolts (geotechnical self-drilling anchor) is used. The wire of the mesh is consisted of high quality steel with great tensile strength (minimal 1770 N/m^2) therefore the technology of double twisting is not possible – Figure 7. Spike plates that are rhombus shaped are installed on rockbolt instead of the normal plates in order to obtain better force transfer from rockbolt over mesh to rock mass.

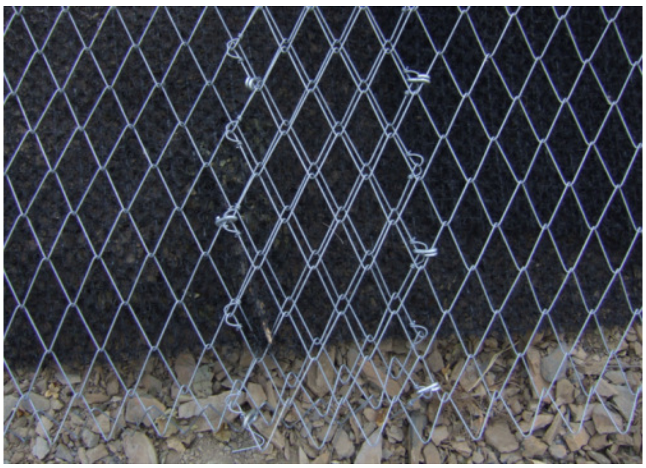

Figure 7. High load-bearing mesh.

4.4 *Systematic slope supporting with anchors and shotcrete*

Rock reinforcement has an important role in maintain and supporting the stability of cuts. Geotechnical self-drilling rockbolt or normal geotechnical steel rod anchors in combination with shotcrete are using for ensuring global slope stability where higher reactions on the support are expected. Anchors are installed on a grid from 2.0 x 2.0 m up to 4.0 x 4.0 m, and types of rockbolts are chosen according to requested bearing capacity (rockbolt diameter, type of self-drilling rockbolts etc.). In all installed rockbolts a pre stressing force in amount of 30% of the calculated force of rockbolt is installed. In that way the rockbolt is activated in the moment when force is installed into the rockbolt and it start to act as an active system of slope supporting. Stone block from the rock mass supported with system reinforcing elements can be exposed to different mechanical behaviors shown on Figure 7.

Load transfer from reinforcing system of the rock mass on to the supporting structure made of shotcrete causes changes in tension status that the shotcrete is not able to take over so it is necessary to reinforce the shotcrete with reinforcement. Shotcrete is installed with wet or dry method in 5.0 cm thick layers. Most commonly two or three layers of shotcrete are installed and they are additionally reinforced with reinforcement meshes.

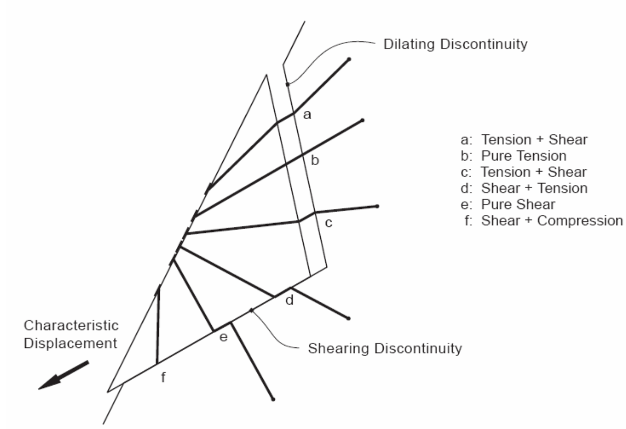

Figure 8. A block with reinforcing elements oriented to reinforce different types of release surfaces (Windsor, 1996)

5 CONCLUSION

Geotechnical design of cuts in rock mass slopes could be divided into two phases. First phase is consisting of geotechnical investigations work (research drilling, geophysical investigations, engineering geological mapping, laboratory testing etc) and stability analysis is part of main design. At this stage behavior of the cut during excavation and slope protection measures are predicted.

At the second stage during construction, by observational method, detailed monitoring of rock mass behavior is maintained, including geotechnical supervision and monitoring by measuring through installed inclinometers and deformeters. Based on given data out of measurement devices and engineering geological mapping of the cuts, it is possible to change factor of safety adopt adequate support systems. During the construction of geotechnical structures in rock mass, design process starts with investigation works and is carried out over the analyses and implementation of the main design, monitoring during the construction and changes of the support systems that are included in the final design.

Because of non homogeneity of rock mass and its engineering-geological characteristics, which change often and locally at the same slope, it is necessary to ensure geotechnical supervision. Local instabilities in karst terrains are frequently presence, unpredictable and it is impossible to discover through geotechnical investigation works. These instabilities appear during the construction stage and are protected by authority of geotechnical supervising engineer.

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