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PRELIMINARY SUPPORT DESIGN FOR UNDERGROUND MINE ADIT, ARTANA MINE, KOSOVO

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Abstract: In this paper, preliminary support design of the main underground opening (i.e., mine adit) located at the Artana lead-zinc mine, Kosovo, was examined by employing both conventional and numerical methods for safe underground excavation and design. In order to conduct field studies including discontinuity surveying and sampling for laboratory testing two empirical methods, namely rock mass rating (RMR) and geological strength index (GSI) were employed. For the purpose of determining necessary support units RMR system was utilized. However, these kind of systems can take into account for neither the depth of underground opening nor in situ field stresses. For this reason, empirical design methods (i.e., RMR system) failed to investigate the performance of rock support units; therefore, a 2D finite element analysis program was used to assess the performance of the proposed support systems. This indicated that RMR system might not be applicable for poor and very poor rock masses located in deep environment (i.e., 300 and 400 m). Moreover, this is linked to the fact that the RMR system does not consider in situ stress conditions. This study showed that when empirical methods are supported by numerical analysis, the preliminary support system design will be much more reliable.

Keywords: *rock mass classification, RMR, GSI, rock support, numerical methods*

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1. INTRODUCTION

In the Artana mine, main galleries, drifts, x/cut galleries, and production galleries are the underground mine openings utilized for the exploitation of ore bodies. Design of these underground mine openings is based mainly on the underground mining method, required production capacity, and mining machinery. Thus, instability of the above mentioned underground mine openings have lead to serious consequences such as injuries/fatalities, production delays, and damage to machinery. Recently, in 2020, a fatal accident in Artana mine occurred due to collapse of a production gallery as a result of which two miners lost their lives due to rock falling. This fatality was investigated and an official report was prepared from the Independent Commission for Mines and Minerals (ICMM) in Kosovo. Other geotechnical problems observed and characteristics of fallouts in underground excavations in another operating mine (e.g., Trepça mine) and in the Artana mine as well are presented in Fig. 1 and was investigated by Zeqiri (2020). Artana Mine is located 51 km south of Prishtina district, Kosovo; the site map is given in Fig. 2. One of the main goals of a geotechnical engineer is to determine the most suitable and economical support system with convenient excavation method and secure excavation against the potential failures and provide safe access for workers and mining machinery (Sari et al. 2008). The key point in this analysis is to understand a fact that, how rock mass surrounding the underground mine adit deforms and how the rock support units acts to control this deformation. The basic input parameters for a safe underground mine adit design are: the rock mass properties surrounding the mine opening, the opening size and geometry, depth and utilized rock support system characteristics (Özsan, Karpuz 2000; Başarir 2006; Geniş, Çolak 2015).

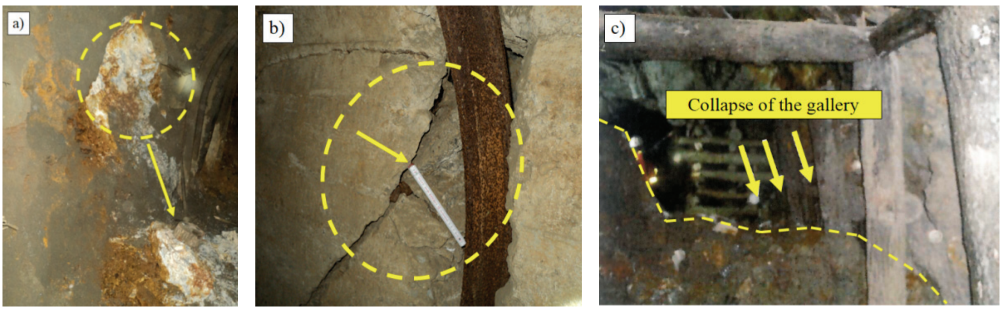


Fig. 1. Different rock failures observed in the Artana mine during mine site investigation: a) thin rock slabs, b) visible cracks in concrete rock support and c) collapse of a production gallery (width/height ≅ 1.5 m)

The main objective of this study is to provide a preliminary support system design required for the underground mine adit at the Artana mine driven in phyllite rock mass based on empirical and numerical methods. Mine site investigations were carried out to identify regions with high potential of stability problems. Moreover, uncertainties in rock strength materials, and a small misjudgments during the preliminary design stage, presents challenges to the geotechnical engineers at the construction phase. Empirical support system suggestions should be augmented by numerical methods (Geniş et al. 2007; Sari et al. 2008). Acting in this way, authors performed a parametric study analyzing the underground opening (i.e., adit) at different depths under different *in situ* stress conditions with unchanged rock properties. With the help of numerical analysis, more realistic underground mine adit and ground conditions can be simulated and it is possible to obtain the extent of failure zones and maximum total displacements around the opening.

Fig. 2. Location map of the Artana lead-zinc mine

2. GEOLOGY

Artana mineral deposit is located in the Kopanik block of the Western Vardar Zone in the further East part of the Dinarides. The Vardar Zone comprises of large cost-effective significant mineral deposits of lead, zinc, silver, bismuth, manganese, copper, iron, and gold. The mineralization belt extends roughly 80 km in the Northern Kosovo and host several mines and mineral occurrences including Stan Trg, Artana, Hajvalia, Kishnica, Cernac and Belo Brdo mines. The Vardar Zone contains fragments of Paleozoic crystalline schist and phyllite with unconformable overlying Triassic clastics, phyllites, volcaniclastic rocks, and Upper Triassic carbonates. The average altitude of the Artana terrain is approximately 1000 m, characterized by hill-mountainous morphology while the lowest point is located in the valley of the Marec River, at a level of 635 m (i.e., mine adit). The geological formations of the Artana mine site mainly consist of metamorphic rocks. Geological map of the Artana mineral deposit is given in Fig. 3.

Based on the lithology and geological structures three main types of rocks are present: footwall rocks: quartz-schists, gneisses, amphibolites schists, amphibolites; ore body (galenite, sphalerite, pyrite, pyrotine, siderite, smithsonite, halloysite, and calcite); hanging wall rocks (phyllite-schist, phyllite, and quartz-schist). Underground mine adit is driven in different rock masses as presented in Fig. 3. Moreover, Fig. 3 presents the longitudinal cross-section view of the overall length of the mine adit driven in level 760 m above sea level (Shabani et al. 2012).

3. ENGINEERING GEOLOGY

Engineering geological studies consist of both field studies and laboratory tests on irregular samples collected from the mine site. A geological cross section along the main adit is given in Fig. 3. The main rock types along the mine adit alignment consist of amphibolites schist, quartz mica schist, amphibolites and phyllite. Amphibolites schist rock type is moderately to highly weathered. The Uniaxial Compressive Strength (UCS) class of amphibolites schist is moderate with an average strength value of 40.2 MPa. The average Rock Quality Designation (RQD) value is 52%. Discontinuity spacing ranges from 5 to 40 cm. Discontinuities show low to medium persistence. Aperture in discontinuities is measured wide. Infilling materials are mainly quartz and chlorite. Quartz mica schist rock type is moderately to highly weathered. The UCS class of quartz mica schist is moderate with an average strength value of 25.9 MPa. The average RQD value is 62%. Discontinuity spacing ranges from 5 to 30 cm. Discontinuities show low to medium persistence. Aperture measured in discontinuities range from open to moderately wide. Infilling materials are mainly quartz and sericite. Amphibolites are slight to moderately weathered. The UCS class of amphibolites is high with an average strength value of 79.6 MPa. The average RQD value is 72%. Discontinuity spacing ranges from 10 to 60 cm. Discontinuities range show very low to low persistence. Apertures in discontinuities are measured in amphibolites ranging from very tight to partly open and open to moderately wide. Infilling materials are mainly chlorite. Phyllite rock type is moderately weathered. The UCS class of phyllite is moderate with an average strength value of 33.4 MPa. The average RQD of phyllite is 33%.

Discontinuity spacing ranges from 6 to 20 cm. Discontinuities are measured in phyllite arranging from open to moderately wide. Infilling materials are mainly quartz and hydrothermal calcite.

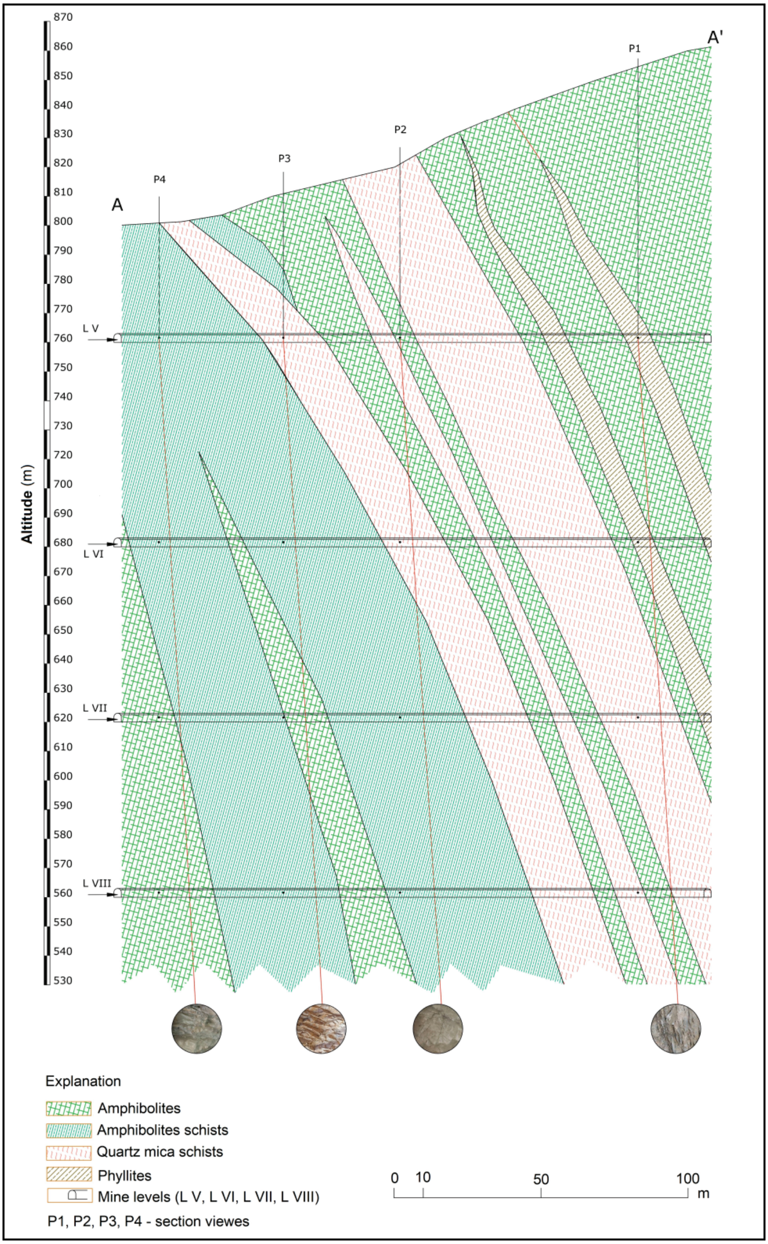


Fig. 3. Geological cross-section along the underground mine adit, Artana mine (Shabani et al. 2012)

4. ROCK MASS CLASSIFICATION

Rock mass classification systems are utilized to evaluate geotechnical properties of rock masses and design preliminary supports for underground construction facilities (Potvin, Hadjigeorgiou 2015). In this study, Rock mass rating (RMR) (Bieniawski 1989) and Geological strength index (GSI) (Hoek et al. 1995) were employed to provide a basis of characterizing rock masses and guide the design of support systems for underground mine openings. The RMR system was initially developed by (Bieniawski 1973) as a design tool for rock mass characterization based on his experiences in shallow tunnel projects driven in sedimentary rocks in South Africa. Later on, the RMR_{73} system was updated to RMR_{89} including changes on the ratings given to joint spacing, joint condition and ground water. For rock mass classification purpose, the UCS of intact rock, rock quality designation, joint condition, joint spacing, joint orientation and ground water are required to be specified. The RMR classification system involves engineering geological feature parameters in determining a quantitative value of their rock mass quality. In this system there is a lack of guideline for support units regarding excavation depth of the underground opening. Although, it is thought that at whatever depth we are dealing with the same rock mass still are not taken into account applied stress. Therefore, numerical modeling techniques play a very important role here, where utilized to verify the empirical results. The RMR results for different rock units are summarized in Table 1.

The GSI index was initially developed by (Hoek et al., 1995) in order to be as practical and simple in application of rock mass characterization as compared to the RMR_{89} system, when used in the Hoek–Brown failure criterion. In field measurements, GSI is estimated from the chart of (Hoek, Marinos 2000), see Fig. 4. GSI index is mainly based on the rock mass structure and condition of discontinuities surfaces and is evaluated form visual observations of the rock mass appearance in surface excavations, out-crops and underground opening faces (Fabich et al. 2015; Ozdogan et al. 2018). GSI results are listed in Table 1.

Table 1. The overall rock rating estimation for different rock mass classification systems

Rock type	RMR		GSI	
	Rating range	Mean value	Rating range	Mean value
Amphibolites	47–71	59	55–65	60
Phyllites	34–42	38	25–35	30
Amphibolites schists	33–48	41	40–50	45
Quartz mica schists	38–55	47	50–60	55

5. GEOTECHNICAL PROPERTIES OF INTACT AND ROCK MASS

5.1. HOEK-BROWN PARAMETERS

Laboratory studies provided in Table 2, were carried out on irregular lamps specimens of various sizes in diameter collected during the site investigation area to determine the physical and mechanical properties of rock materials, including rock, unit weight and point load strength test index. The point load test enables economical testing of lump rock samples. In order to determine Poisson's ratio and the uniaxial compressive strength, the index-to-strength conversion factors are used (Carter, Sneddon 1997; Rusnak, Mark 1999; Vásárhelyi, Kovács 2017). The GSI index was initially developed by (Hoek et al. 1995) in order to be as practical and simple in application of rock mass characterization as compared to the RMR, when used in the Hoek–Brown failure criterion.

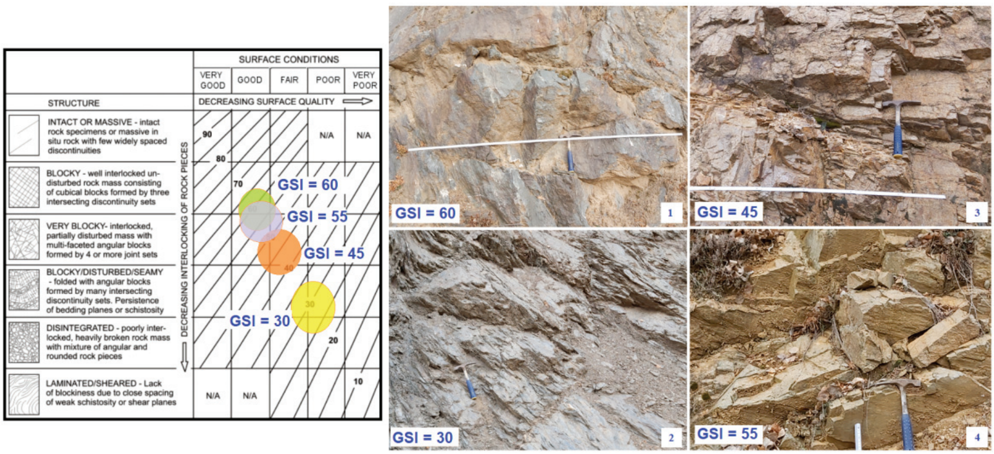


Fig. 4. Rock mass characterization on the basis of GSI:
1 – amphibolites corresponding to the blocky rock type, 2 – phyllites corresponding to the blocky/disturbed/seamy rock type, 3 – amphibolite schists corresponding to very block rock type, 4 – quartz mica schists

Later on, Marinos and Hoek (2000) proposed a quantitative GSI chart. Hoek et al. (2002) suggested the following equations for estimating rock mass properties as follows:

$$m_b = m_i e\left(\frac{GSI-100}{28-14D}\right), \tag{1}$$

$$s = e\left(\frac{GSI-100}{9-3D}\right), \tag{2}$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right), \tag{3}$$

where: D is the disturbance factor, and m_b, s, a are material constants for the rock mass. In this study, the value of D is considered 0.8 since the blasting operations are regarded as very poor blasting in the studied site.

Table 2. Laboratory tests, GSI values and estimated Hoek–Brown parameters

Rock type	Unit weight γ [kN/m^3]	Point load test Is [MPa]	Uniaxial compressive strength σ_{ci} [MPa]	Poisson's ratio v	Geological strength index (GSI) value	m_i
Amphibolites	26.42	3.62	79.6	0.17	60	26 ± 6
Phyllites	25.72	1.52	33.4	0.23	30	7 ± 3
Amphibolites schists	26.31	1.83	40.2	0.22	45	10 ± 3
Quartz mica schists	28.09	1.18	25.9	0.19	55	20 ± 3

5.2. ROCK MASS PARAMETERS

Characterizing rock mass strength along the main underground opening using the generalized Hoek–Brown criterion, three main parameters must be determined; uniaxial compressive strength and m_i values for the intact rock and geological strength index for the rock mass. Hence, these allow the direct estimation of the rock mass properties from the following equations suggested by (Hoek, Diederichs 2006):

$$E_m = E_i\left(0.02 + \frac{1 - D/2}{1 + e^{((60+15D-GSI)/11)}}\right), \tag{4}$$

$$\sigma_m = \sigma_{ci}\left(\frac{(m_b + 4s - a(m_b - 8s))(m_b/4 + s)^{a-1}}{2(1+a)(2+a)}\right), \tag{5}$$

$$\phi_m = \sin^{-1}\left[\frac{2am_b(s + m_b\sigma'_{3n})^{a-1}}{2(1+a)(2+a) + 6am_b(s + m_b\sigma'_{3n})^{a-1}}\right], \tag{6}$$

$$c_m = \frac{\sigma_{ci}[(1+2a)s + (1-a)m_b\sigma'_{3n}](s + m_b\sigma'_{3n})^{a-1}}{(1+a)(2+a)\sqrt{1 + (6am_b(s + m_b\sigma'_{3n})^{a-1})/((1+a)(2+a))}}, \tag{7}$$

where: $\sigma_{3n} = \sigma'_{3\max}/\sigma_{ci}$, σ_{ci} is the uniaxial compressive strength of intact rock, m_b, s, a are rock mass constants. Estimated rock mass properties for modeling purposes are listed in Table 3.

Table 3. Geotechnical rock mass properties

Parameters		Amphibolites	Phyllites	Amphibolites schists	Quartz mica schists
Modulus ratio (MR)		450 ± 50	550 ± 250	560	375 ± 75
Young's modulus, E_i [GPa]		35.82	18.37	22.51	9.71
Deformation modulus, E_m [GPa]		6.12	0.60	1.52	1.22
Uniaxial compressive strength, σ_m [MPa]		3.77	0.13	0.58	0.83
Cohesion, c_m [MPa]		4.43	0.52	1.13	1.19
Friction angle, ϕ_m [°]		33.6	11.5	19.1	28.9
Hoek–Brown constant	m_b	2.40	0.11	0.37	1.37
	s	0.0023	0.000024	0.0002	0.0011
	a	0.503	0.522	0.508	0.504

6. STABILITY ASSESSMENT

6.1. EMPIRICAL METHOD

The relationship between the rock mass rating to stand-up time of an unsupported underground span is shown in Fig. 5. The interpretation of RMR classification ratings in terms of stand-up times for the underground mine adit driven in phyllites is given in Table 5. For the favorable situation the underground mine adit need to be oriented such direction that dominant joint set strikes perpendicular to the drive direction (Hoek, Brown 1980). The interpretation of RMR classification results in terms of stand-up times and rock mass strength parameters are given in Table 5. Suggestions for rock support units to be used for individual rock type are listed in Table 6.

Table 4. Meaning of rock mass classification RMR_{89} (Hoek, Brown 1980)

Rock mass properties	RMR (rock class)				
	100–81	80–61	60–41	40–21	<20
	I	II	III	IV	V
	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock
Average stand-up time	20 years for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hours for 2.5 m span	30 minutes for 1 m span

Despite the fact that rock mass classification systems are broadly utilized to carry out the support system design of underground excavations, such empirical approaches fail to predict interaction between host rock mass and supporting units, therefore, fail to provide descriptions on behavior of supported structures.

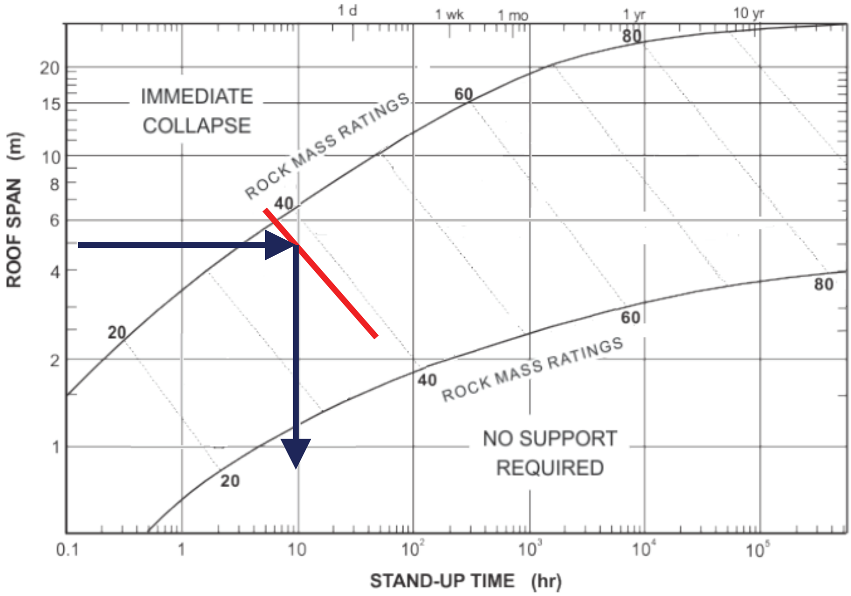


Fig. 5. Relationship between the stand-up time of an unsupported underground span and the RMR_{89} classification (Hoek, Brown 1980)

Table 5. Maximum unsupported span using RMR_{89} (Hoek, Brown 1980)

Rock type	RMR	Class No.	Description	Average stand-up time
Amphibolites	47–71 (59)	III	Fair rock	10 days for 12 m span
Phyllites	34-42 (38)	IV	Poor rock	10 hours for 4.2 m span
Amphibolites schists	33-48 (41)	III	Fair rock	1 day for 6 m span
Quartz mica schists	38-55 (47)	III	Fair rock	2 days for 8 m span

Accordingly, in this study for safe underground openings support design empirical and numerical methods were utilized. Taking into account the support units and excavation method recommended by empirical methods, four combined support systems with same excavation method were proposed for the Artana underground mine adit on the basis of rock mass quality (i.e., phyllite rock mass section). The proposed rock support systems were designed according to the concept that as the excavation depth increase (e.g., 100, 200, 300 and 400 m), the support requirements will increase, as illustrated in Fig. 6. This is indicated by the use of a wire mesh and shotcrete for excavation depth of 100 m (Fig. 6a), while for excavation depth of 300 and 400 m requires a systematic rock bolts, thicker shotcrete lining as well as the use of steel sets (HE 100A and HE 140B), see Figs. 6c and 6d.

Table 6. Proposed rock support systems for the underground mine adit based on RMR_{89} (Hoek, Brown 1980)

Rock type	Suggested support units			Excavation method
	Rock bolts	Shotcrete	Steel sets	
Amphibolites (59)	Systematic bolts 4 m long and spaced 1.5–2 m in crown and walls with wire mesh in crown	50–100 mm in crown, and 30 mm in sides	None	Top heading and bench: 1.5–3.0 m advance in top heading; Commence support after each blast; complete support 10 m from face
Phyllites (38)	Systematic bolts 4–5 m long and spaced 1–1.5 m in crown and walls with wire mesh	100–150 mm in crown and 100 mm in sides	Light ribs spaced 1.5 m where required	Top heading and bench: 1.0–1.5 m advance in top heading, install support concurrently with excavation 10 m from face
Amphibolites schists (41)	Systematic bolts 4 m long and spaced 1.5–2 m in crown and walls with wire mesh in crown	50–100 mm in crown, and 30 mm in sides	None	Top heading and bench: 1.5–3.0 m advance in top heading; Commence support after each blast; complete support 10 m from face
Quartz mica schists (47)	Systematic bolts 4 m long and spaced 1.5–2 m in crown and walls with wire mesh in crown	50–100 mm in crown, and 30 mm in sides	None	Top heading and bench: 1.5-3.0 m advance in top heading; Commence support after each blast; complete support 10 m from face

6.2. NUMERICAL ANALYSES

In this research, the Phase2 v8.0 (Rocscience 2012) plane strain analysis program, based on the finite element method was used to model an underground excavation (i.e., mine adit) surrounding phyllite rock mass, considering suggested support units based on RMR classification systems.

Accordingly, the Phase2 is used to design underground openings and surface excavations and their support systems, respectively. The objective of using numerical modeling tools is to check the permissibility of the proposed support units by empirical method and propose a possible preliminary support system design. The modeled rock mass is considered as a homogenous and isotropic medium. Assumptions were considered adequate, especially for poor rock mass rating (e.g., phyllites) as in this study. In this study, an underground mine adit with the opening size and geometry

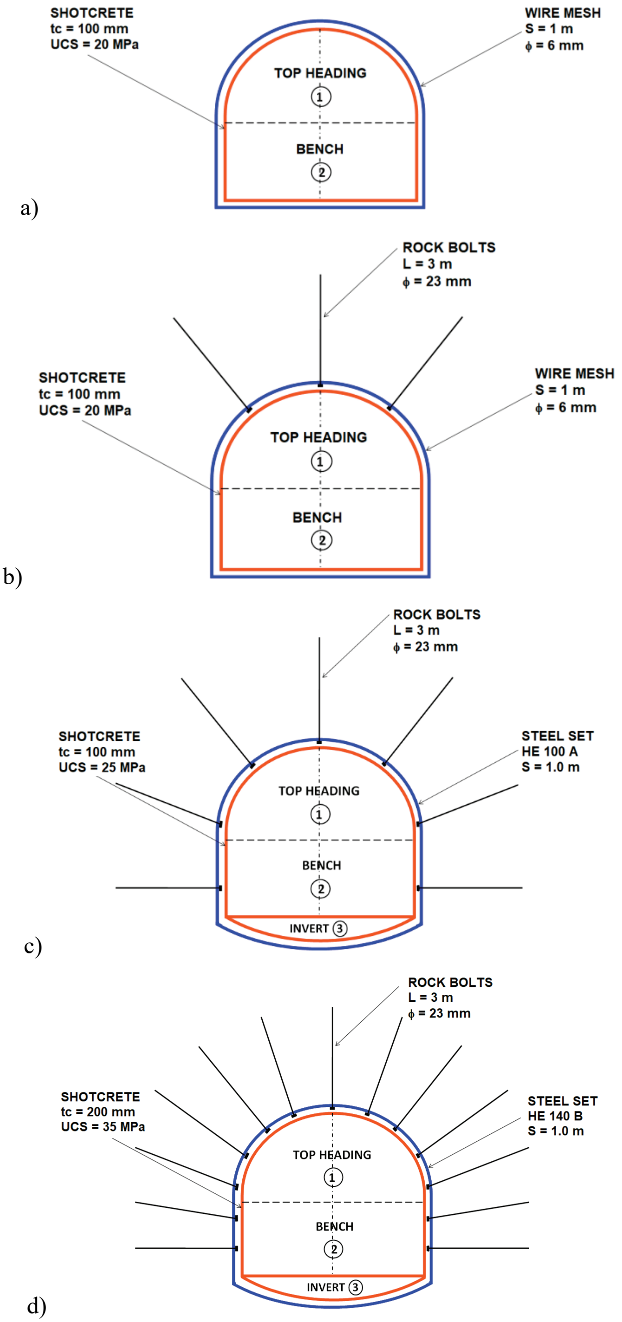


Fig. 6. Proposed support units and excavation method for phyllite rock type with dimensions 4.2 m in width and 3.5 m in height a) proposed design at a depth of 100 m, b) proposed design at a depth of 200 m, c) proposed design at a depth of 300 m, d) proposed design at a depth of 400 m

with respect to excavation depth was modeled. The external model boundary was considered 10 times the diameter opening. At the vicinity of the underground opening finer mesh was set up to increase the accuracy of results within the interested study area. In situ stresses were applied to the outer horizontal and vertical boundaries. This research is a parametric study, carrying out analysis at different depth with same rock material properties. The input rock material properties listed in Tables 2 and 3. Displacements and failure zones around the underground mine adit are estimated based on the Hoek–Brown criterion. Herein, it was assumed that host rock mass behave as an elastic-perfectly-plastic material.

Since in situ stress data were not available, two assumptions were considered. The first assumption was that the rock mass stress conditions are primarily a function of gravitational effect meaning that vertical stress (P_v) is one of the principal in situ stresses. The magnitude of the vertical in situ stress is a function of overburden depth. The second was assumed that the ratio of horizontal to vertical in situ stress could be considered as ($k = 1$), this means hydrostatic in situ stress state (Hoek 2003; Geniş et al. 2007; Langford et al. 2016). Both assumptions seem to be reasonable as poor rock mass such as phyllites are unable to sustain high shear stresses. The in situ stress state according to excavation depth was considered as 2.7, 5.4, 8.1 and 10.8 MPa for studied underground opening. In order to describe the underground structure, ground section was divided into more than 10 000 triangular finite elements. This analysis consists of two models. The first model was set up to investigate excavation without any support, and the second model involves application of different support units to excavation boundary. In reality, installation time of rock support systems takes some time and meanwhile some deformations are allowed to occur. Hence, to simulate delayed support installation load splitting and material softening can be applied in the second model, set up models were analyzed in different stages.

6.3. NUMERICAL RESULTS AND DISCUSSION

Numerical analyses were interpreted in terms of thickness of failure zones and displacements for unsupported and supported models. Hence, the ratio of thickness of failure zones (i.e., plastic zone) (Rp/R) and displacements for unsupported and supported underground mine adit at a variable excavation depth are illustrated in Figs. 7 and 8. From Fig. 7, it can be seen that extent of plastic zone increases with excavation depth. Accordingly, the extent of plastic zone and yielded elements alarm us that there would be observed some stability problems for the underground mine adit driven in the phyllites. Therefore, observing carefully the unsupported and supported openings, it seems to be much more important to consider of extent of plastic zone rather than the magnitude of displacements. For the underground mine adit excavated in phyllites, the radius of plastic zone for unsupported opening at a depth of 100 m is 3.49, whereas, for supported the radius plastic ratio value decreased to 2.66. Managing the extent of failure zone to a ratio value of 2.66 wire mesh together with shotcrete proposed by empirical methods are considered as sufficient support elements. In addition, the radius of failure zone generated around the underground opening at depth of 100 and 200 m, before and after support installation are presented in Fig. 7. The maximum displacements around supported and unsupported opening at a depth of 300 m and 400 m are shown in Fig. 8 and the overall displacements results are presented in Table 7. Support units such as rock bolts proposed by RMR system for poor rock masses did not help to decrease significantly the thickness of failure zone. Herein, installation of rock bolts for such cases is used only to hold failed rock blocks. The rock bolt capacity diagram is given in Fig. 9, this illustrates an example of an installed rock bolt at a depth of 400 m to help stabilizing failure zone which serve as an artificial arch in a failure

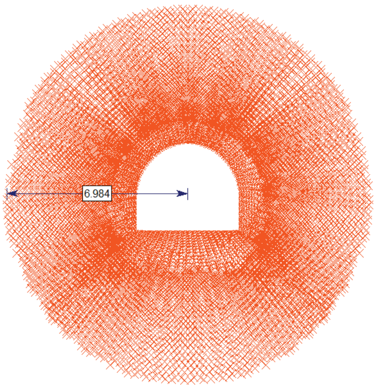
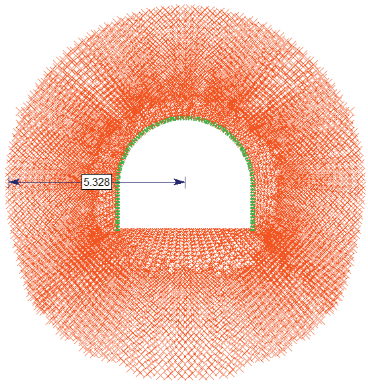
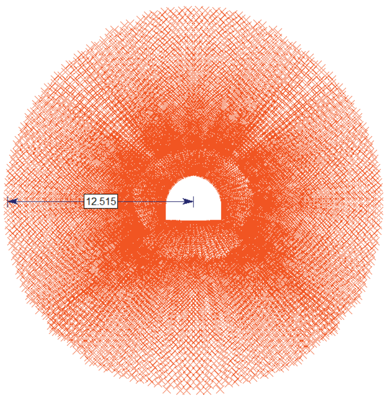
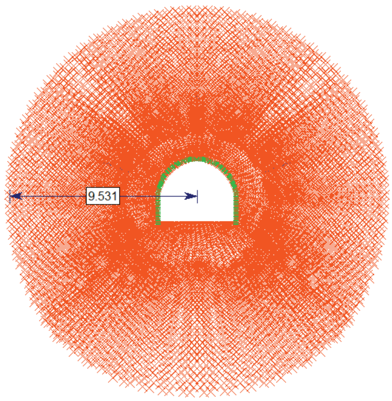
Depth [m]	Unsupported	Supported
100		
	$Rp/R = 3.49$	$Rp/R = 2.66$
200		
	$Rp/R = 5.88$	$Rp/R = 4.76$

Fig. 7. Plastic zone for unsupported and supported underground opening in phyllite rock mass

Table 7. Thickness of plastic zone and maximum total displacements for unsupported and supported underground mine opening

Depth [m]	Unsupported		Supported	
	Extent of failure zone	Maximum total displacements [mm]	Extent of failure zone	Maximum total displacements [mm]
100	3.49	3.8	2.66	2.4
200	5.88	15.2	4.76	9.5
300	9.48	47	6.51	18
400	13.05	100	8.35	36

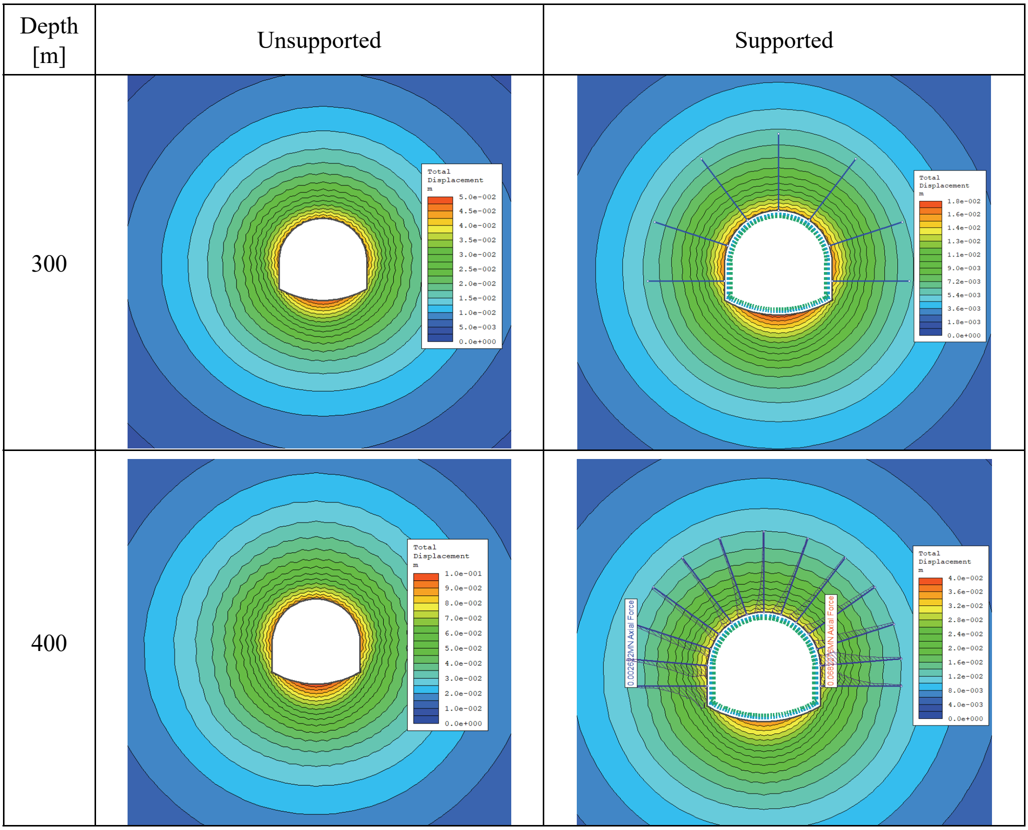


Fig. 8. Maximum total displacements before and after support for phyllite rock mass

zone. This means that RMR system might not be applicable for poor and very poor rock masses located in deep environment (i.e., 300 and 400 m). Moreover, this is linked to the fact that the RMR system does not consider in situ stress environment (Başarir 2008). Investigation of the response of proposed support systems to the excavation sequence and resulting underground opening deformations, a set of support capacity diagrams have been plotted in Fig. 10. The support capability plots given in Fig. 10 for underground mine adit are self-generated by the program.

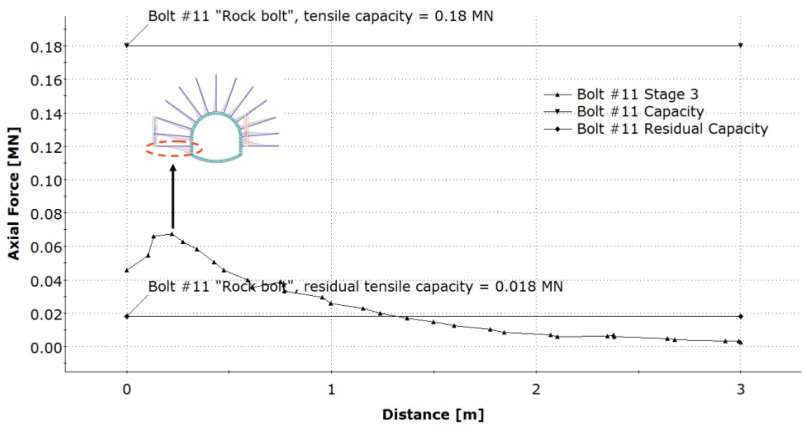


Fig. 9. A typical example of rock bolt capacity diagram for underground mine adit located at a depth of 400 m

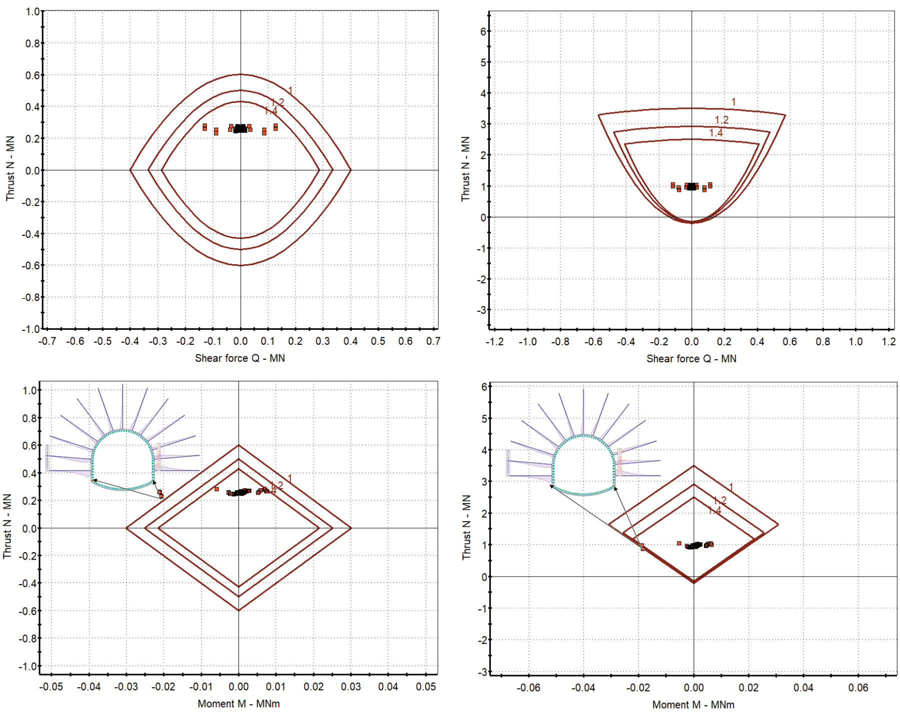


Fig. 10. Support capability diagrams for a 200 mm shotcrete lining reinforced with HE140A steel set (see Fig. 8b) in a phyllite rock mass at a depth of 400 m below the ground surface

According to numerical analyses and results the combination of support elements (i.e., rock bolts, shotcrete lining with steel sets) is acceptable for depths greater than 300 m at Artana mine. In a typical underground mine adit design in which support contain of steel set fixed in shotcrete lining, a geotechnical engineer sees it necessary to know the contribution and performance of these support elements and to be able to modify the number and/or dimensions of each to accommodate the loads imposed on the lining. Therefore, these loads are obtained from numerical analyses in which "beam elements" are attached to the underground opening boundary and the axial thrust, bending moments and shear forces induced in these elements are calculated directly (Hoek et al. 2008; Langford et al. 2016; Khadka, Maskey 2017). Axial thrust force, shear force and bending moment capability envelopes gives the factor of safety of shotcrete lining and steel sets, respectively. Thus, the calculated lining and steel set values fall within an envelope (e.g., between 1.2 and 1.4 safety factors) which means that they have a factor of safety greater than the envelope value. It is found that the lining safety factor exceeds the designed safety factor. The shear forces and bending moments would be higher at the corners of underground mine adit, see Fig. 10. It might be a good idea to increase the shotcrete lining thickness at the corners of right and lefts walls and invert. However, apart from the empirical methods, numerical modeling would give a clear understanding to design rock support systems for such poor rock masses.

7. CONCLUSION

According to the field investigations and rock mass classifications systems it is indicated that some stability problems exists in the phyllites rock mass along the main underground opening and different rock support units are required. Using of numerical method it was possible to verify the existence of some stability problems for the unsupported underground mine adit. With the help of numerical methods it was possible to evaluate the performance of recommended support units by empirical method. Numerical analyses and results indicated that empirical methods are not valid and fail to investigate the performance of rock support units for poor and very poor rock masses located in deep working environments (e.g., 300 m, 400 m). However, back analyses of the underground mine opening behavior will allow both the refinement of the numerical model and design of different rock support units.

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