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MINING OPERATIONS AND GEOTECHNICAL ISSUES IN DEEP HARD ROCK MINING – CASE OF BOUKHADRA IRON MINE

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Summary

Underground mining operations are a very problematic task, especially in poor geotechnical conditions. The right choice of excavation and support techniques leads to adequate and secure mining operations. This should ensure the overall stability of the underground mine with the best productivity and stability performance. In this paper, an empirical model for obtaining support systems for underground galleries was applied. Then, a numerical model for the evaluation of the performance of support measures for rock masses in the Boukhadra iron mine was introduced. Extensive field and laboratory tests were performed to obtain geological, geotechnical, and mechanical data on the entire geologic formations of the (1105 m) level. The performance of the design is supported by the selection of a common support plan between RMR, Q, and UBC systems for each geotechnical unit. Therefore, the rock masses classification based on the geo-mechanical model has determined the suitable support systems. The finite element model (FEM) was used for the analysis of rock mass behaviour, displacements, stress, and plastic point distribution. The results permit the optimization of the plastic zone thickness around the gallery. The outcomes of this study could improve the stability of the mine by choosing the right direction of excavation in consideration to the direction of the discontinuity planes. In order to choose between the current and the recommended mining operations, an equivalent calculation sequence was verified. Our study demonstrated that the consideration of discontinuity sets in the orientation of excavation highly improves the mining conditions with or without support.

Keywords

geo-mechanic • mining gallery • discontinuity • iron ore • underground mines

1. Introduction

Underground extraction of raw materials is constantly facing security and productivity challenges [Zahri et al. 2017, Zerzour et al. 2020]. This problem concerned mining operators and geotechnical engineers [Mouici et al. 2017, Manchar et al. 2018, Zeqiri et al. 2019, Fredj et al. 2020]. It is common that the geological and hydrogeological conditions

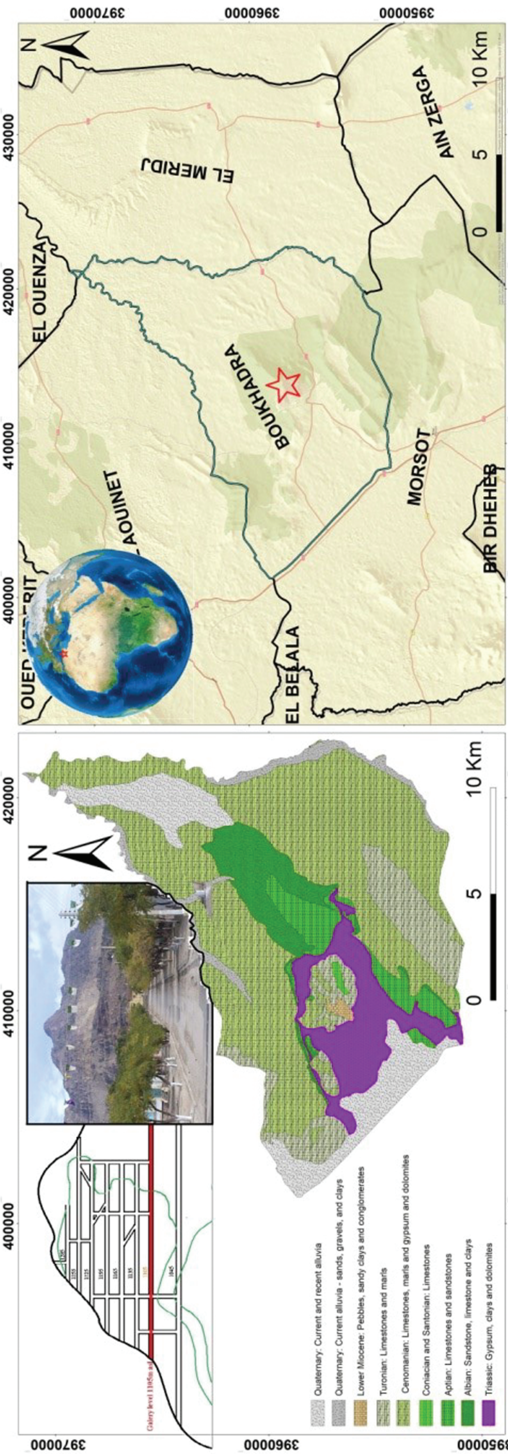
[Demdoum et al. 2015, Anis et al. 2019, Hamed et al. 2017, 2022] determine the feasibility and cost of underground work projects. These factors affect the excavation methods and supporting systems [Gadri et al. 2012]. The stress relaxation, deconfinement process, lining interaction, pore pressure variation, time effect, construction sequence, and soft delayed response accompanying the excavation operations lead inevitably to a progressive loss in the rock matrix rigidity [Hadji et al. 2014a, Saadoun et al. 2020]. Under the effect of strain, tiny fissures expand in the rock structure and gradually join together until plastic deformation. The theory of fracture mechanics and macroscopic homogenization responds perfectly to the advanced modelling of structures in damageable materials. Compared to shallow extraction, working conditions in underground mines raise more technical issues. The rock mass behaviour depends on the geomechanical characteristics and the morphometric schemes of discontinuity, and the boundary conditions of structure [Gadri et al. 2015]. Deformations and stress analysis of soil have always been a matter of interest to researchers [Hadji et al. 2013, Zerzour et al. 2021]. Uncertainties in rock behaviour are related to discontinuities, anisotropy, heterogeneity, the no-elastic mode of the rock mass, etc. Complications in designing underground excavation are mainly caused by the content constitutive laws in numerical models that do not reflect accurately the real rock behaviour. With the help of empirical methods, numerical modelling has become more precise [Raïs et al. 2017]. A small misinterpretation in the preliminary stages of the design [Hadji et al. 2014b, El Mekki et al. 2017] can lead to disastrous results in both the construction and the operation of a tunnel. The basic parameters that influence the stability of tunnel construction must be carefully assessed and applied as early as the initial steps of the design phase. Rock mechanics engineering is interested in rock slope stability, rock bolting, foundations on rocks, tunnelling, blasting, underground and open pit mining, mine subsidence, dams, bridges, and highways [Dahoua et al. 2017a, b, 2018, Saadaoui et al. 2022]. The term rock mass is applied to a large extent of rock, from several meters to a few kilometres, which can include diverse discontinuity sets of different forms and natures. In underground excavation, once deformations of mined areas have expanded, adequate support becomes imperative. When designing a shallow tunnel in a poor-quality rock mass, the designer has to face a number of problems that appear less often in deeper tunnels [Hoek 2004]. Therefore, it is necessary to examine basic concepts of how a rock mass surrounding a tunnel deforms and how the support system acts to generate the deformation [Hoek 1998]. The dimensioning of the required support to stabilize a tunnel excavation is a four-dimensional problem. The reduction of the internal 'support pressure' results in a redistribution of stress within the model, and can lead to the increasing plasticity of the rock mass around the tunnel. When underground engineering structures are designed, empirical and numerical methods are the two main methods for analysing the stability of rock mass surrounding a tunnel. Rock mass classification systems are very useful tools for the preliminary design stage of a project, when very little information on rock mass is available. Numerical modelling is an integral part of modern tunnel engineering design, because it enables the assessment of rock-structure interaction and stability as well as assists choosing the optimum excavation method and support measures. An accurate selection of empirical methods for estimating the stability of mining structures excavated in fissured rock masses are the main subject of this study. This approach is complemented by numerical assessment of the results found following the geo-mechanical study.

2. Research area characteristics and conditions

The ferruginous deposit of Jebel Boukhadra is lying in a carbonate rock mass mount located in north-eastern Algeria, 45 km from the chief town of Tebessa province, near the Algerian-Tunisian border. This mountain is characterized by a simple anticline structure in the NE-SW direction with a periclinal termination NE. This anticline structure extends over 7 to 8 km in length and 3 to 5 km in width across the NE-SW direction (Fig. 1). It is composed mainly of Mesozoic-Cenozoic formations covered by a Quaternary deposit [Tamani et al. 2019, Kerbati et al. 2020]. The Triassic diapir contacts the Cretaceous limestone in the west, south, and southeast parts of the anticline [Hamad et al. 2021]. There is not any aquifer in the mining site at the 1463 m a.s.l. The only aquifer in this area is located at 818 m a.s.l, well below the mine [Ncibi et al. 2020]. Underground mining covers three mining districts (along three axes): north axis, south-east axis, main axis. The excavation of the Boukhadra deposit is performed according to the downcast method, guaranteeing the opening using exposed galleries. On each floor, an intermediate level or sub-level is dug, dividing the floor into three sub-floors. These levels are created at least every 20 m, and they are intended for drilling. The transport of the blasted ore from an upper level (head level) to a lower one (base level) is carried out using an inclined extraction stack (average inclination is 70°).

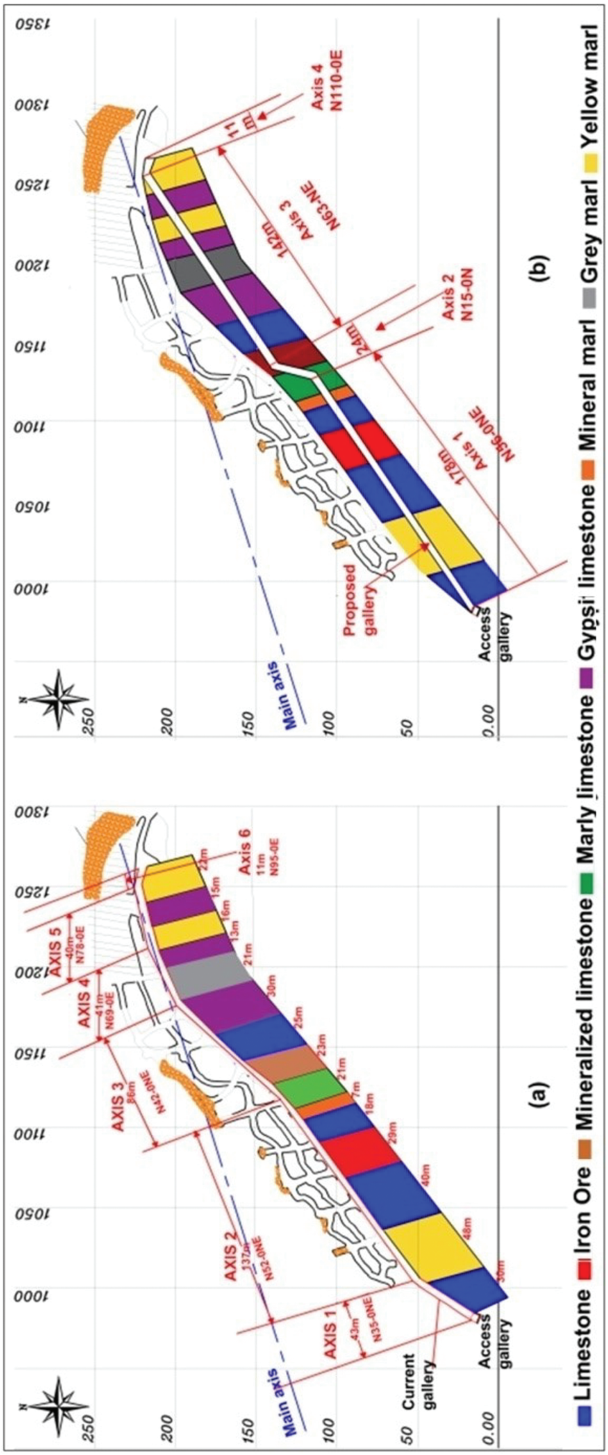
Stability problems (rockslides) at 1105m a.s.l. (north axis), which is the level of the entrance to the gallery, meant that it was necessary to dig another tunnel in order to extract the ore from the bottom. The latter was excavated in parallel with the main axis orientation. The mining in the main gallery at the 1105m a.s.l. (Fig. 2) has reached an advanced stage with 358 m of length, and 17 m^2 of an average section through different facies (iron ore, limestone, mineralized limestone, marly limestone, gypsiferous limestone, mineralized marl, yellow marl, gray marl). Currently, this gallery is the only access to the deposit. To prevent any possible reappearance of the instability problem, we decided to check the state of stability of the main gallery by studying the impact of the stresses induced around the walls using geotechnical tools. This way we could look for unstable areas and their causes, and then recommend suitable technical solutions.

The stereographic projection shows that the massif is extremely fractured, which complicates this work (Fig. 3). The axes of the current gallery are oriented respectively N 35 – 0 NE /43 m, N 52 – 0 NE /137 m, N 42 – 0 NE /86 m, N 69 – 0 E /41 m, N 78 – 0 E/40 m, and N 95 – 0 E/11 m orientation/length of axes 1 to 6. Dominant discontinuity sets for the different geological formations have been grouped into four major sets, namely F_{M1} (N90–37S); F_{M2} (N35–55NW); F_{M3} (N90–56N) and F_{M4} (N148–60NE). In order to improve the stability of the gallery (1105 m a.s.l.) we have simulated a change of orientation of the gallery axes. These changes in orientation are based on a treatment of the structural characteristics of the rock mass. We have developed the analysis of



Source: Authors' own study

Fig. 1. Geographical location and geological map of the study area



Source: Authors' own study

Fig. 2. a) current gallery of 1105 m a.s.l. b) proposed gallery of 1105 m a.s.l.

discontinuities as a stereographic projection using Dips 7.0. We have proposed a new gallery on the basis of a geomechanical description proposed by Bieniawski [1989]. This description takes into account the orientation of the discontinuities with respect to the direction of digging of the gallery. The proposed gallery consists of four axes, with respectively N56-0NE/178 m, N15-0N/24 m, N63-0NE/142 m, N110-0E/11 m orientation/length of axes 1 to 4.

The proposed structure involves a reduction in the number of axes (from 6 to 4) and in the total length (from 358 m to 355 m) with an improvement of stability of its axes. This progress is represented by an adjustment (parameter B) that varies between unfavourable and average (−10 ÷ −5). However, the adjustment of the axes of the existing gallery varies between unfavourable and very unfavourable (−12 ÷ −10). The distance that needs to be supported is reduced from 114 m (excavated) to 61.5 m (recommended).

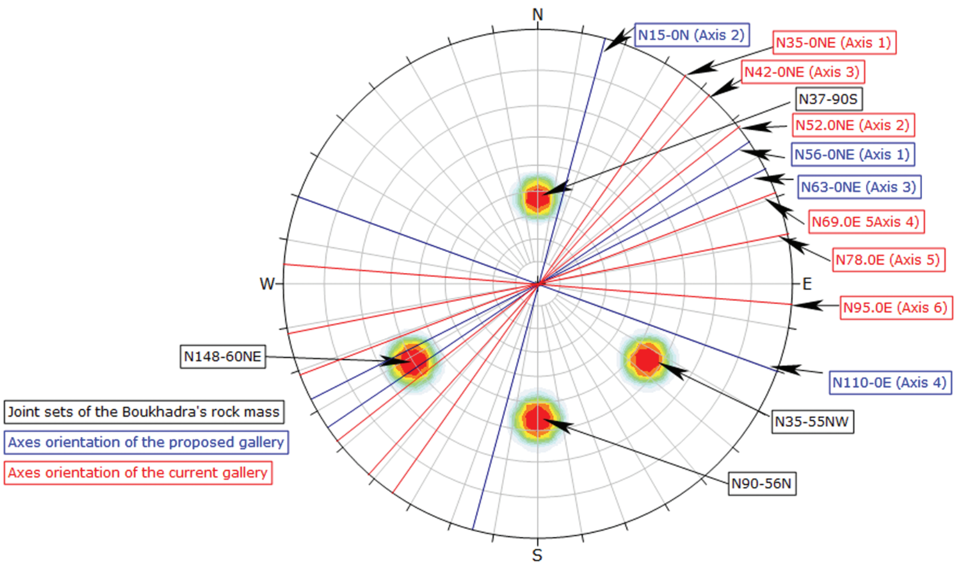


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Fig. 3. The relation between the direction of digging of galleries and the discontinuity planes

3. Material and methods

Ground behaviour around an excavation is generally influenced by several factors such as lithology, the geological structure, groundwater and the in situ stress field. Some of these factors have several sub-factors with widely varying properties. For instance, the geological structure includes joints, bedding shears and faults etc., as well as their orientation, spacing, continuity, surface characteristics and filling materials etc. Lithology, on the other hand, may represent intact rock characteristics such as strength, elastic or plastic deformability, and swelling and slaking etc. In strong discontinuous rock masses, the physical characteristics of geological structural features, i.e. orientation, persistence, spacing, aperture etc., can widely vary, and their impact on the rock mass can be significant. However, in weak or weathered rock, the intact rock strength may become more important compared to geological discontinuities. The effect of geological structure may also depend on the size and orientation of the excavation. Similarly, in deep excavations in situ or induced stress field may be the governing factor of stability, while geological structure plays only a secondary role. Ideally, the classification method should assess the relative importance of these factors and represent as exactly as possible their relative influence on different excavations. All relevant parameters should be accounted for by giving adequate ratings in the system, but no parameter should be counted more than once as this may reduce the weight of the other relevant parameters. In other words, the ground should be characterised by parameters that are exactly congruent with the true factors and their relative influence; and one should be taken careful not to count the same parameter twice. Naturally occurring rocks are anisotropic and non-homogeneous with widely varying properties. For instance, as already indicated, the characteristics of geological structural features, i.e. joint spacing, orientation, persistence etc. that govern the behaviour of the rock mass can widely vary. Similarly, other factors such as groundwater and lithology etc. can also vary significantly. The determination of representative conditions or values of these factors should not involve uncertainty nor subjective bias.

RMR rock mass classification system was initially developed at the South African Council of Scientific and Industrial Research (CSIR) by Bieniawski [1974, 1984].

$$\begin{gathered}\text{RMR basic} = \Sigma \text{ parameters } (\text{A1} + \text{A2} + \text{A3} + \text{A4} + \text{A5}) \\ \text{RMR} = \text{RMR basic} + \text{adjustment for joint orientation}\end{gathered} \tag{1}$$

Bieniawski categorized the RMR systems as following: good rock RMR = 61–80, fair rock RMR = 41–60, poor rock RMR = 21–40, and very poor rock with RMR < 20 [Zahri et al. 2016].

The Q-system of rock mass classification was developed by Barton et al. [1974] at the Norwegian Geotechnical Institute (NGI), and updated to include 1000 cases [Grimstad and Barton 1993]. Barton [2002] compiled the *Q*-system again and made some changes to the support recommendations.

$$Q = \frac{RQD}{Jn}\frac{Jr}{Ja}\frac{Jw}{SRF} \tag{2}$$

The first quotient (RQD/*Jn*), representing the structure of the rock mass, is a crude measure of the block or particle size [Hoek 2007].

The second quotient (*Jr/Ja*) represents the roughness and frictional characteristics of the joint walls or filling materials (representing shear strength).

The third quotient (*Jw/SRF*) is a complicated empirical factor describing active stress.

The Mathews-Potvin method is based mainly on field observations [Potvin 1988]. The stability graph is a plot of a stability number (N') against a shape factor (S), referred to a hydraulic radius (HR) [Suorineni 2012]. Hydraulic radius is the ratio of the stop surface area on the stop surface perimeter [Potvin and Hadjigeorgiou 2001].

N' is calculated according to the equation:

$$N' = Q' \cdot A \cdot B \cdot C \tag{3}$$

where: Q' is a modified rock tunnelling quality index of Barton et al. [1974].

The rock stress factor A is defined as a function of the ratio between the unconfined compressive strength of the intact rock (σ_c) and the induced principal stress (σ_1) on the studied exposed walls of a stop [Pagé et al. 2018, Grenon and Hadjigeorgiou 2003].

Factor B depends on the difference between the orientation of the critical joint and the face of the stops [Hoek et al. 1993]. C: is a gravity factor; it considers the influence of the inclination of the exposed surface, and the inclination of the critical joints.

To avoid subjective preferences in the classification the rock mass parameters should be considered quantitatively, and their variations should be accounted for in the assessment of rock mass. The assessment or the calculation of numerical values for the parameters should not be user-dependent. Possible use of lump-sum ratings should be avoided as this leads to subjective adjustments when proposing rating values.

4. Geomechanical classification of Boukhadra's underground mine

The analysis of the gallery (current & proposed) was carried out at all cross-axes. Drilling core specimens taken from these axes were used in order to perform rock mass characterization. The basic RMR, Q and UBC were estimated for all sections of the gallery's axes and the values are given in Tables 1 and 2.

The RMR method classifies the rocky massif of Boukhadra as being a massif formed of medium quality rocks (marl), and the rest of the massif is considered as a good quality rock (limestone and iron ore), with the score of RMR89 varying between 44 and 74.

For the proposed new gallery, the rating of RMR 89 has improved thanks to the restoration of the direction of excavation of the gallery in relation to the plans of discontinuities (46 ≤ RMR89 ≤ 76). This improvement is the result of an adaptation study of the adjustment parameter, which takes into consideration the direction of mining with respect to the planes of discontinuities.

The *Q*-System classification method classifies the rocky massif of Boukhadra as a massif formed of rocks of very bad (marl), good (limestone: marly, mineralized and gypsiferous) and very good quality (limestone and iron ore), with a *Q* score varying between 0.3 and 44.8.

The absence of direct inclusion of the orientation of the gallery in relation to the planes of discontinuities, and the conservation of the same height of cover along the work (current and proposed) led to the conservation of the same results for the *Q*-System method.

Table 1. Empiric classification of Boukhadra's underground mine

Axis length	Current gallery						Proposed gallery						
	Facies	*N'*	*HR*	RMR_{89}	*B*	*Q*	Axis	Facies	*N'*	*HR*	RMR_{89}	*B*	*Q*
1–43 m	Limestone	105.8	3	72	–12	42.66	1 178 m	Limestone	105.8	2.99	74	–10	42.66
	Yellow marl	5.5	2.38	44		0.3		Yellow marl	3.7	3.2	46		0.3
2–37 m	Yellow marl	3.7	3.08	44	–12	0.3		Limestone	105.8	3.15	74		42.66
	Limestone	105.8	3.16	72		42.66		Iron ore	410.6	2.95	76		44.8
	Iron ore	410.6	2.99	74		44.8		Limestone	105.8	2.63	74		42.66
	Limestone	105.8	2.72	72		42.66		Mineralized marl	1.2	1.99	47		0.3
	Mineralized marl	1.2	1.88	45		0.3		Marly limestone	24.3	2.5	63		40
	Marly limestone	24.3	1.81	61		40	2 24 m	Marly limestone	24.3	2.23	63	–10	40
3–86 m	Marly limestone	24.3	2.44	61	–12	40		Mineral-limestone	38.3	2.38	64		40
	Mineral-limestone	38.3	2.83	62		40	3 142 m	Mineral-limestone	38.3	2.31	64	–10	40
	limestone	105.8	2.88	72		42.66		Limestone	105.8	2.83	74		42.66
	Gypsi limestone	36.7	2.88	62		40		Gypsilimestone	36.7	2.94	64		40
4–41 m	Gypsi limestone	36.7	1.5	64	–10	40		Gray marl	0.5	2.8	46		0.3
	Grey marl	0.5	2.78	46		0.3		Gypsilimestone	36.7	2.34	64		40
	Gypslimestone	36.7	2.41	64		40		Yellow marl	0.7	2.58	46		0.3
5–40 m	Yellow marl	0.7	2.55	44	–12	0.3		Gypsi limestone	25.8	2.6	64		40
	Gypsilimestone	25.8	2.5	62		40		Yellow marl	0.7	2.31	46		0.3
	Yellow marl	0.7	2.23	44		0.3	4 11 m	Yellow marl	0.7	2.23	51	–5	0.3
6–11 m	Yellow marl	0.7	2.23	44	–12	0.3							

Table 2. Support systems proposed by RMR-system

Rock mass of Boukhadra	Recommended support		
	RMR method	Q-System method	Mathews-Potvin method
Iron ore, limestone gypsiferous limestone marly limestone mineralized limestone	Locally, bolts in the crown 3 m long, spaced 2.5 m with occasional wire mesh in the crown. Shotcrete in the crown 50 mm thick.	Unsupported	Unsupported
Yellow marl grey marl mineralized marl	Systematic bolts 4 m long, spaced 1.5–2 m in the crown and walls with wire mesh in the crown. Shotcrete in the crown 100 mm thick, and 50 mm in the walls.	Fiber reinforced shotcrete 5–9 cm, and bolting with diameter of 20 mm and 4 m length.	Cable bolt with density of 0.4cable bolt/m^2, and length of 5 m (equal to the length of SPAN)

The problem of tetrahedral wedge instability in jointed rock masses at relatively shallow depths can be addressed in three distinct analytical steps. The first is to identify the kinematically possible modes of potential collapse in the block assemblage. The second is to determine the state of equilibrium of the kinematically unstable rock wedges identified in the rock mass. The third is to determine the support required to stabilise the potentially unstable rock wedges. However, during the last two decades, this difficulty has been overcome by using the Shi's Block Theory [Goodman and Shi 1985]. It is a method to identify the types of blocks that can be formed in a jointed rock mass and to separate those that are kinematically moveable into the excavations. It can handle an unlimited number of discontinuities and identify the shape and location of the movable blocks anywhere in the periphery of an excavation. By using the block theory, computer software packages were developed for the identification and analysis of the potentially unstable rock wedges in underground excavations. One such package developed specifically for tetrahedral rock wedge analysis for underground excavation design is UNWEDG. The UBC method classifies the Boukhadra massif as a massif formed of stable rocks (limestone and iron ore), and other unstable ones (marls), with a note of N' varying between 0.5 and 410.6, and the hydraulic radius (HR) – between 1.5–3.16 m. For the proposed gallery, the UBC method takes into consideration the dip of the discontinuities as a factor influencing the stability of the underground works by neglecting the orientation of the surface of the discontinuities. This neglect and the conservation of the same height of cover along the structure (current and proposed) have led to a conservation of the stability number N' and a change in the HR parameter (1.99÷3.2), resulting in a shift in the state of the marls (yellow and mineralized) located in the 1st axis towards the stability. Support and construction procedures should neither be overly conservative nor optimistic and compromise safety. Ideally, the degree of safety should be known and be congruent with the requirements of different projects and different stages and/or sections of the same project. This means a pre-determinable factor of safety, which may change from case to case, should be included in the method. Built-in safety factors of unknown magnitude are not desirable. Intact rock strength (IRS) is a factor in the SRF term of the *Q* system, only if the excavation stability is affected by the in situ stress field. In contrast, IRS is always included in the RMR value. If the IRS changes, while all the other parameters remain virtually the same, several RMR values are possible for a single *Q* value.

The in situ stress field is not accounted for in the RMR system in classifying a rock mass. In the *Q* system it is a factor in the SRF term if excavation instability is stress driven. Thus for a rock mass with a given RMR value, several different *Q* values are possible depending on the SRF value used. Joint spacing (JS) is a key parameter in the RMR system; the closer the JS the lower the RMR value and the wider the JS the higher the RMR value. This is not so in the *Q* system. If three or more joint sets are present and the joints are widely spaced, it is difficult to get the *Q* system to reflect the competent nature of a rock mass. For widely spaced jointing, the joint set parameter *Jn* in the *Q* system appears to unduly reduce the resulting *Q* value. Thus for a single *Q* value, several RMR values are possible depending on *JS*.

RQD is used in both methods, and is a function of joint spacing, albeit it does not fully represent the true nature of joint spacing. In addition to RQD, as already mentioned, *JS* is also a key parameter in the RMR method. In the *Q* system, although the number of joint sets is taken into account, spacing is not considered directly. This means joint spacing is counted twice in the RMR method, while the *Q* system uses it indirectly only once.

Joint orientation (*JA*) is accounted for directly in the RMR method by giving a rating between 0 and –12. In the *Q* system, this is considered implicitly, but no guidelines are provided to identify the adversely oriented discontinuities. Thus, the selection of the most critical discontinuity set is user-dependent. In any case no rating is given to the *JA* in the *Q* system. Thus, for a given *Q* value, different RMR values are possible depending on the orientation of the excavation relative to the discontinuity set orientation.

From the foregoing, it is clear that the predictions made by the two systems are unlikely to match perfectly for all rock mass conditions in underground excavations. A universally applicable single formula for linking RMR and *Q* value is also unlikely to be achieved.

5. Numerical analysis of the case study

The primary function of numerical modelling in the underground excavation design process is to simulate the stress distribution and the rock mass behaviour around the excavation and the influence of different support systems on the excavation stability. The main benefits of numerical modelling are that both stress and displacements around the excavation can be computed, and different constitutive relations for the rock mass can be employed. The numerical models used in rock engineering assume that rock masses can be mathematically represented either as a continuous media with elastic properties or as an assemblage of discrete blocks formed by pre-existing weakness planes; the blocks may be rigid or elastically deformable. The geological, geometric, physical and mechanical parameters concerning the different geological formations intersected by the main gallery at the 1105 m a.s.l. are collected in order to model the rock massif of Boukhadra using the Plaxis 3D digital code.

The FEM uses variational methods from the calculus of variations to approximate a solution by minimizing an associated error function. The numerical model is characterized by:

- The material model used is jointed rock,
- The type of material behaviour is drained.

6. Laboratory tests

A very large number of samples were taken from the two galleries, to determine the physical and mechanical properties of the rocks necessary for the numerical study. The results of the tests are presented in the tables below (3 and 4). Some simple physical testing may also be used.

Table 3. Physics and mechanics characteristics of rock mass of Boukhadra

Parameters		Limestone	Mineralized limestone	Iron ore	Gypsiferous limestone	Mineralized marl	Marly limestone	Grey marl	Yellow marl
Current gallery	C(MPa)	1.173/2.108	0.359/0.525	0.837/1.316	0.373/0.516	0.151/0.296	0.235/0.335	0.078/0.167	0.08/0.166
	φ (°)	53/55.08	36.5/43.11	48/50.8	38/43.37	23/37.1	30.5/37.53	15.5/28.24	15/27.34
	E_m (GPa)	20.782/25.4	2.560/6.69	8.604/19.8	2.895/6.7	0.134/2.2	1.082/4.24	0.034/1.2	0.034/1.1
	RMR_{89}	72	62	74	62	45	61	46	44
	GSI	67	57	69	57	40	56	41	39
Proposed gallery	C' (MPa)	1.366/2.356	0.373/0.553	0.926/1.446	0.384/0.544	0.157/0.308	0.245/0.349	0.083/0.167	0.085/0.172
	φ' (°)	53/55.19	38/43.63	48.5/51.07	38.5/43.89	24/37.72	32/38.1	16/28.24	15.5/27.91
	E'_m (GPa)	22.949/28.5	2.895/7.51	9.4365/22.2	3.076/7.51	0.150/2.44	1.225/4.76	0.038/1.2	0.038/1.2
	RMR_{89}	74	64	76	64	47	63	46	46
	GSI	69	59	71	59	42	58	41	41
UCS (MPa)		91.28	20	44	20	15	9	4	4
ν		0.22	0.25	0.24	0.25	0.3	0.25	0.3	0.3
γ_{dry} (KN/m^3)		26	28	29	27	24	25.5	19.5	21
γ_{sat} (KN/m^3)		26.5	29	30	28	25	26	22	23

Table 4. Properties of the applied supports

Rock mass of Boukhadra	Rock bolt				Shotcrete		
	Diameter	E	Length	Spacing	Thickness	UCS	E
Limestone; mineralized limestone: marly limestone; gypsiferous limestone; iron ore	Good quality of rock mass (Stable, No support recommended)				Good quality of rock mass (Stable, No support recommended)		
Mineralized marl	20 mm	200 GPa	5 m	1 m	50 mm	20 MPa	30 GPa
Grey marl	20 mm	200 GPa	5 m	1 m	50 mm	20 MPa	30 GPa
Yellow marl	20 mm	200 GPa	5 m	1 m	50 mm	20 MPa	30 GPa

Only limited number of parameters can be determined from the data sources available at the pre-construction stage. During construction in the mine many details can be detected, but time is often limited. Parameters that can be easily obtained from outcrops and boreholes, or quickly observed or measured in the excavation site, are desirable. Ideally, the parameters obtained at any stage of a project should lead to the same conclusions regarding the rock mass conditions and support requirements for excavations.

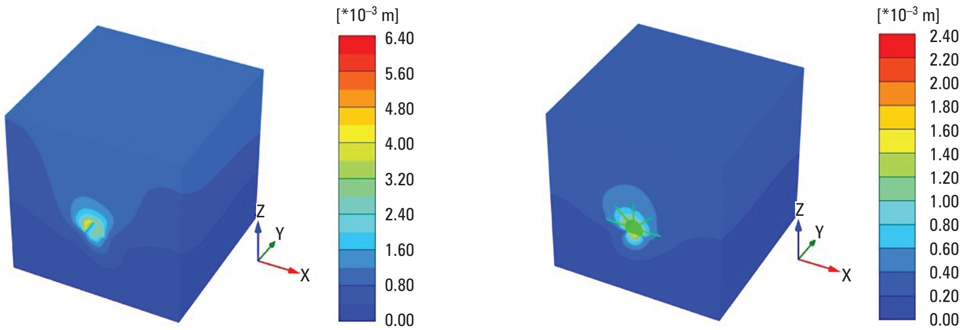
$$GSI = RMR_{89} - 5 \text{ [Hoek 1994]}$$
$$E_m = 1000 \cdot [\sigma_c / 100]^{0.5} \cdot 10^{(GSI - 10)/40} \; (\sigma_c < 100 \text{ MPa}) \text{ [Hoek and Brown 1997]} \tag{4}$$

7. Numerical simulation

After carrying out the calculations of the displacements around the walls of the gallery using the Plaxis 3D digital tool, it can be observed that there are facies presenting a certain instability in the form of large deformations, and other facies showing stability with only small deformations. The stability analysis by the numerical method showed that the displacements around the current gallery vary between 259E-6 and 192.1E-4 m. However, the latter varies between 230.4E-6 and 102.1E-4 m along the proposed gallery marking some improvement in stability. The reinforcement of the walls of the gallery by bolts and a concrete ring presented an improvement in the state of the stability of the gallery. The installation of the support resulted in a change of stresses, and critical displacements from the roof to the slab. The comparison made between the different developed approaches showed absolute agreement between the results obtained by the empirical methods and those given by the numerical method analyzing the state of stability of the structure. Thus, efficiency of the support design recommended for stabilizing unstable places (Table 5, Fig. 4).

Table 5. Total displacements

Rock mass of Boukhadra	Current gallery		Proposed gallery	
	Utot (m) unsupported	Utot (m) supported	Utot (m) unsupported	Utot (m) supported
Limestone	259E-6	×	230.4E-6	×
Mineralized limestone	973.8E-6	×	932.6E-6	×
Marly limestone	158.4E-5	×	141.8E-5	×
Gypsiferous limestone	923.5E-6	×	805E-6	×
Iron ore	307.5E-6	×	283.4E-6	×
Mineralized marl	62.52E-4	220.3E-5	38.02E-4	186.9E-5
Grey marl	101.1E-4	341.6E-5	101.1E-4	341.6E-5
Yellow marl	192.1 E-4	664.7E-5	102.1E-4	494.3E-5



Source: Authors' own study

Fig. 4. The FEM 3D-Plaxis model of the proposed gallery

8. Conclusions

Amongst the several rock mass classification methods developed for application in underground excavation engineering, two have stood out. These are known as rock mass rating (RMR) and tunnelling quality index (*Q*), introduced by Bieniawski [1973] and Barton et al. [1974], respectively. Over the years, the two methods have been revised and updated so as to improve their reliability as support design tools, yet the two methods are known to have limitations, and their reliability has long been a subject of considerable debate. Nevertheless, attempts to assess their reliability in a systematic manner have been limited. Further, some practitioners in the field of rock engineering continue to use these methods as the sole methods of support design for underground rock excavations.

This study assumed that the reliability of the RMR and Q methods can be assessed by comparing their support predictions with those derived by other applicable methods and also with the actual support installed. Such an assessment can best be carried out during the excavation of an underground opening, because representative data can be collected by direct observation of the excavated ground conditions and by monitoring the performance of the support installed. In this context, the geotechnical data obtained during the construction of several case tunnels were reviewed and the two classification methods were applied.

In contrast, the rational or theoretical approach to underground excavation design uses explicit models representing the behaviour of rock masses developed based on the principles of the mechanics of materials. The application of this approach requires access to accurate information on the rock mass properties, groundwater conditions and in situ stress condition, and is often time-consuming and costly.

While both approaches serve the same purpose, the classification methods are used if there is insufficient information to establish an explicit model or if time and cost limitations prevent the use of other models. A classification method should be applicable to a wide range of ground conditions, opening sizes and shapes, different construction procedures and support types. Although some experience in underground excavation design and construction may be a prerequisite, the application process of classification methods should not require a high level of skills. After a few applications, a user should be able to easily and confidently judge the situation and make required decisions. Simplicity of form along with clarity and un-ambiguity of the terminology used is also important. The conventional drill-and-blast method of excavation proved problematic for part of the gallery because the blasting caused damage to the rock mass. Despite the fact that the RMR system suggested drilling and blasting excavation methods for the entire gallery, jackhammer techniques will be proposed for more than 350 m thus greatly reducing unnecessary damage to the rocky massif.

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