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RMR based tunnel support

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Abstract

For professional tunnelling engineers and geologists, particularly in the early stages of a project, this study could be useful as it provides a critical assessment of different solutions works best in the RMR rock mass classification system and how the values derived from it can be translated into the establishment of a practical rock safety system 53 years after its development and extensive accumulated experience.

It addresses three aspects of practical tunnel design: 1) new design tables for determining the required rock support as a function of tunnel span and rock quality; 2) the determination of tunnel cross-section shapes and secondary linings; and 3) an updated RMR grading method for practical applications. The practical solutions presented in this paper are based on the experience accumulated over several decades by Lowson and Bieniawski (2013). After years of validation and application, the authors are glad to observe that the recommendations are genuinely aligned with reality and are effective. Not only the Q method guidelines can be used to establish rock support, but also the RMR. In this paper are presented some updated relations from the original publication (Lowson and Bieniawski 2013).

Keywords

RMR89, RMR14, RQD, Rock support, Rock mass.

1 Introduction

In the form of a table (Bieniawski 1989), the original RMR-based tunnel support standards gave support advice for a tunnel span/diameter of 10 meters. As the technology for steel ribs, shotcrete, and rock bolting has advanced, tunnel designers have been given the responsibility of adapting these standards for different tunnel sizes, which has proven successful. It is now clear after 37 years of usage that practical tunnel designers would benefit from charts and equations that guide the choice of rock support based on the tunnel size and the rock mass quality. Consequently, this is the primary goal of this paper, along with modernizing the methodology for better calculating the RMR values and forecasting the adequate and safe tunnel support (Lowson 2012, Lowson and Bieniawski 2012).

The rock support methods for each rock mass quality are a combination of different support types, as shown in Table 1. The next section discusses how to determine the support needs for particular elements, like rock bolts, shotcrete, and steel ribs, since, for instance, two support approaches are somewhat additive.

Table 1. Original guidelines for support of rock tunnels based on the RMR system (Bieniawski 1989).

Rock Mass Class	**Excavation**	**Support**		
		Rock Bolts (20 mm Dia., Fully Grouted)	**Shotcrete**	**Steel Sets**
Very good rock I RMR: 81-100	Full face 3 m advance	Generally, no support required except spot bolting		
Good rock II RMR: 61-80	Full face 1.0-1.5 m advance	Locally, bolts in crown 3 m long, spaced 2.5 m, with occasional wire mesh	50 mm in crown where required	None
Fair rock III RMR: 41-60	Top heading and bench 1.5-3 m advance in top heading	Systematic bolts 4 m long, 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
Poor rock IV RMR: 21-40	Top heading and bench 1.0-1.5 m advance in top heading. Install support concurrently with excavation 10 m from face	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and wall with wire mesh	100-150 mm in crown and 100 mm in sides	Light to medium ribs spaced 1.5 m where required
Very poor rock V RMR: <20	Multiple drifts 0.5-1.5 advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt in invert	150-200 mm in crown, 150 mm, in sides, and 50 mm on face	Medium to heavy ribs spaced 0.75 m with steel lagging and fore-poling if required. Close invert

Before making any recommendations, it is crucial to consider what practicing tunnel designers currently need. Based on the opinion of the authors of this article and earlier publications (Lowson and Bieniawski 2013), when comparing numerical modelling with empirical estimates based on extensive accumulated experience, the use of continuum models often provides unreliable results regarding rock support, especially at shallow depths, although they can be useful in extreme cases where plastic deformation effects are present and the rock mass exhibits high plasticity. The practicing tunnel engineer needs design tables to work efficiently. Accordingly, the practicing tunnel engineer is concerned with design rules for both shotcrete and rock bolts, depending on the tunnel dimensions and rock quality, to assess the final rock support requirements. We believe that the tunnel construction industry takes a pragmatic approach, as it is satisfied with an easily determinable RMR rating, including the orientation of joints as a component.

Following the original 2013 article (Lowson and Bieniawski 2013), an update and revision of the RMR rock mass classification method was published in 2014, launching the modified RMR14 method (Celada et al. 2014).

The RMR14 system has been used in numerous projects in various locations around the world; in contrast, while, the rock mass rating guidelines recommended in 2013 have been used less frequently (Rehman et al. 2018, Kanik et al. 2015, Liu et al. 2021, Zhang et al. 2021).

After a brief introduction to the RMR89 and RMR14 methods, we present in more detail a minor modification of the RMR89 recommended by Bieniawski in 2013, and we present rock mass classification methods based on this system.

The rock mass classification system based on the RMR89 method, developed in 2013, was first applied in Hungary in the geotechnical design and report underpinning the detailed design of the M85 tunnel (Deák 2019).

2 Rock support systems for underground excavations

2.1 Design load

Rock load on a support system is a function of rock mass condition and initial stress state. Where the rock mass is not overstressed and squeezing is not occurring, the design load is taken as (See Eq. 1):

$$\mathrm{P_r}(\mathrm{RMR}, Span) = \frac{100 - \mathrm{RMR}}{100} \cdot 10\mathrm{m} \left(\frac{Span}{10\mathrm{m}}\right)^{\frac{1}{2}} \cdot \mathrm{p_r} \cdot \gamma_r \tag{1}$$

Where γ_r is a partial factor; $\mathrm{p_r}$ is rock density. Assuming a partial factor of 1.5 and a density of 27 kN/m³, this yields the following results, which are shown in Fig. 1.

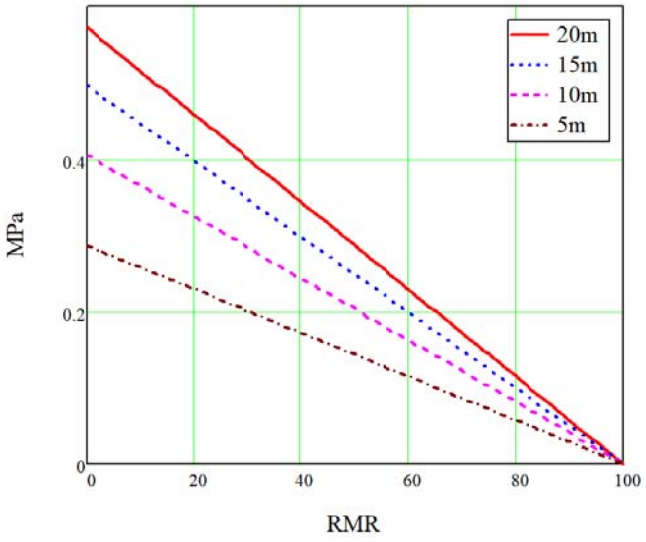


Fig. 1 Rock load versus span.

2.2 Stand up time

Our research in this area is still ongoing; we still need to analyze several historical projects in which we examined underground mined systems. We still aim to classify the rock mass and then examine its stand-up time. For instance, the mined cavity system of the old Roman gold mine Alburnus Major at Rosia Montana - Romania (with varying rock mass quality) has stood unsupported for more than 2,000 years. Based on the original Bieniawski table, several sections of it should have collapsed long ago. We plan to revise this table in the near future.

2.3 Bolt spacing

In our recommendation below, we consider the spacing of the rock bolts solely in terms of the RMR. The spacing must reflect the fracture frequency and ensure that the shotcrete provides adequate support between the bolts at the right end of the scale, where the shotcrete cannot realistically be expected to function as an arch on its own. Experience shows that for RMR ≥ 85, only individually spaced rock anchors are required (Eq. 2-4 and Fig. 2).

if $20 < \mathrm{RMR} \leq 85$

$$S_b = 0.5m + 2.5m \cdot \frac{RMR - 20}{65} \tag{2}$$

if 10 < RMR ≤ 20

$$S_b = 0.25m + \frac{(RMR - 20)^{1.5}}{140} m \tag{3}$$

if RMR ≤10

$$S_b = 0.25m \tag{4}$$

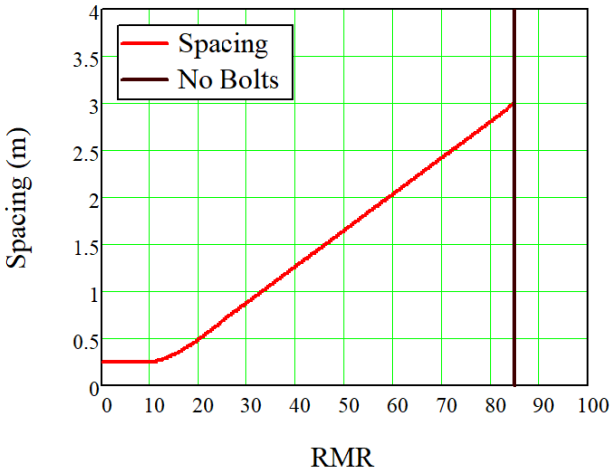


Fig. 2 Rockbolt spacing versus RMR.

2.4 Rock bolt length

Bolt length must vary with span and RMR. Based on empirical guidelines used in mining and the results of numerical modelling studies, the following relationship was obtained (Eq. 5 and Fig. 3):

$$Span = \frac{(L_b + 2.5)^{\frac{RMR+25}{52}}}{3.6} \tag{5}$$

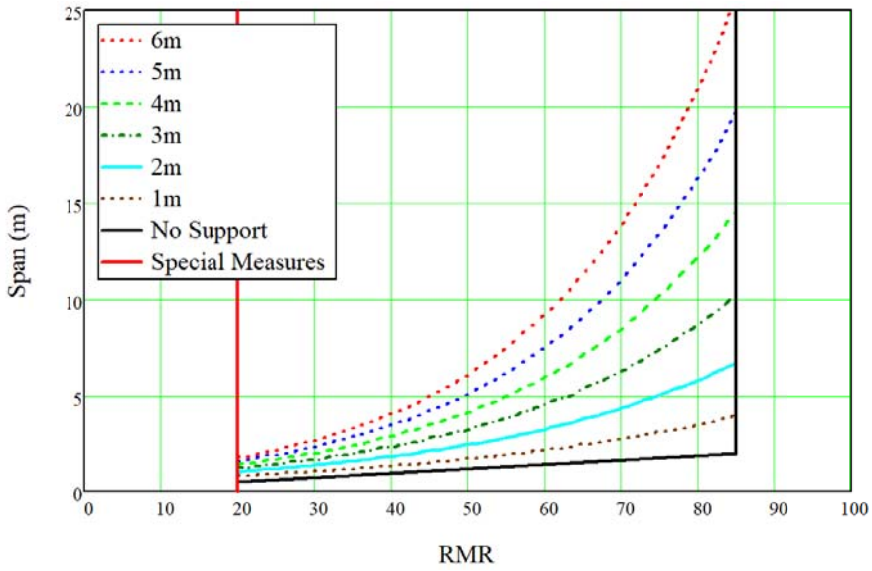


Fig. 3 Rockbolt length chart.

2.5 Rock bolt capacity

The support capacity of pattern rock bolting is considered to be determined by dividing the capacity of each bolt by the area it must support. The capacity of bolts decreases with a lower RMR due to the increasing challenge of achieving an effective bond and the decreasing ability of the bolts to initiate arching action within the rock mass. The resulting relationship is presented below (Eq. 6 and Fig. 4). The graph represents the capacity of a standard 25 mm bolt with an ultimate tensile strength of 25 tons.

$$F_{ub}(RMR) = \frac{F_b}{\gamma_b} \cdot \left(\frac{RMR}{85}\right)^{\frac{40}{RMR}} \tag{6}$$

Where F_b (bolt capacity) = 250 kN, γ_r (bolt partial factor) = 1.5.

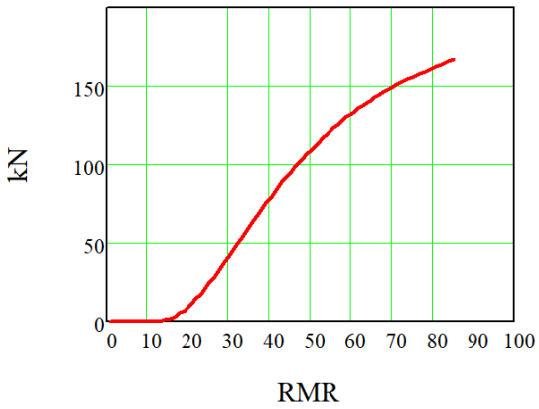


Fig. 4 Design rockbolt capacity versus RMR.

2.6 Shotcrete capacity

If the rock mass has good quality (RMR > 60), only a thin layer of shotcrete is required. Applying a thin layer of shotcrete to an irregular surface in blocky rock results in a layer of shotcrete that is itself irregular, and assuming it behaves as a stand-alone arch is not credible. Where a thin layer is applied to an uneven surface with a texture depth much bigger than shotcrete thickness, the key supporting mechanism is the shotcrete prevents inter-block movement and general loosening, and so mobilises arching in the rock.

For medium quality rock, say RMR = 35 to 60, the shotcrete can more reasonably be thought of as acting as an arch. If large parts of the tunnel are in rock of this quality the tunnel shape will probably be a horseshoe with curved sidewalls, and certainly at the lower end of this scale the shotcrete will have filled a lot of the irregularities in the surface and the inside shape will be curved with few re-entrant areas.

Below RMR values around 35 the excavation would require multiple headings unless the tunnel is very small. This is done to keep the size of the face and therefore the size of any heading collapse to manageable proportions. Analysis of the support system by numerical methods or otherwise finds that the shotcrete is increasingly subjected to flexural and shear forces as well as simple axial compression.

If the axial load values on the Y-Axis are divided by member thickness the result is a Moment / Axial Stress curve. The curve shown below is for 32MPa cylinder strength shotcrete with a thickness of 300mm.

The figure below shows the design strength of 30 MPa (f_{ck}) shotcrete (Fig. 5).

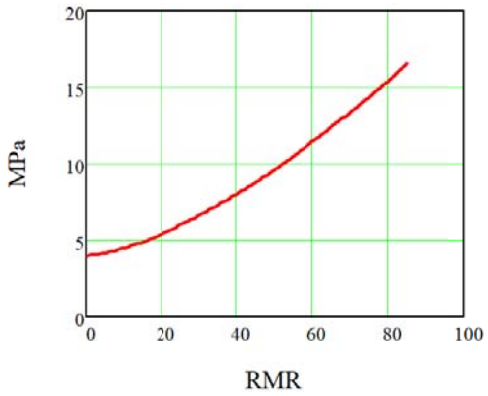


Fig. 5 Design shotcrete strength.

$$f_{cd}(RMR) = \frac{f_{ck}}{\gamma_s} \cdot \left[0.2 + 0.8 \cdot \left(\frac{RMR}{100}\right)^2\right] \tag{7}$$

2.7 Shotcrete support solutions

Based on the above relationships for the capacities of shotcrete and rock bolts a chart relating Span and RMR for a given thickness of shotcrete can be derived. The capacities of the rock bolts and the shotcrete are considered to be additive.

The irregularities in the lines around RMR = 50 to 20 indicate that the rock bolts' ability to provide meaningful support has ended. The middle option in the equation for rockbolt spacing, for RMR between 10 and 20, is used as a transition to create a somewhat seamless curve in the graph below (Fig. 6).

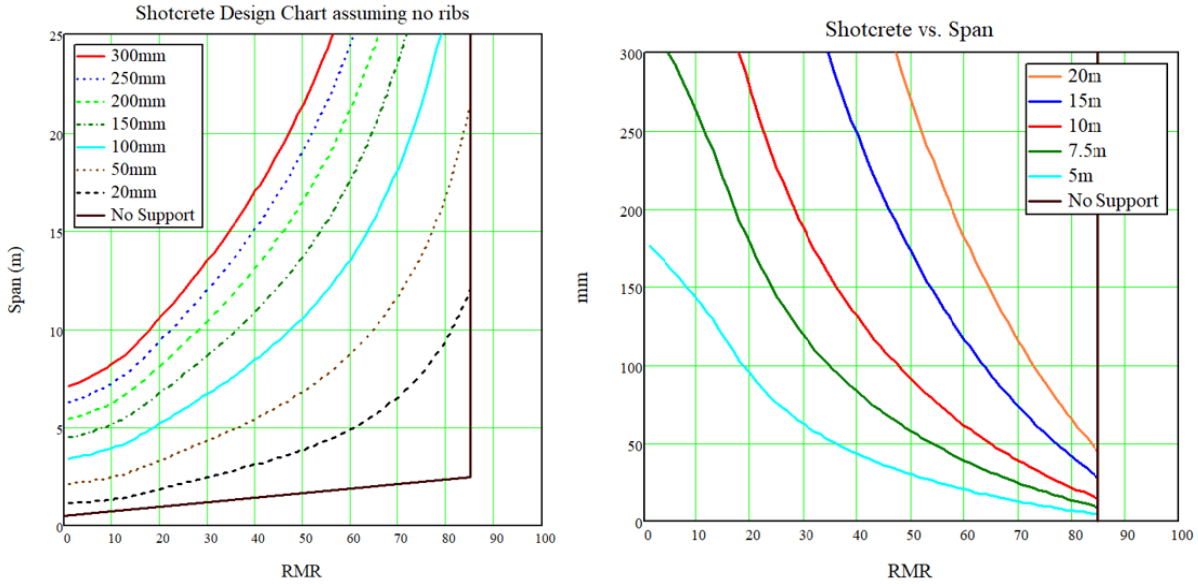


Fig. 6 Shotcrete design charts.

3 Replacement of the RQD during the RMR calculations

This parameter was originally included among the six RMR parameters because all of the case studies collected in 1972 included RQD. Over the years it became apparent that RQD was difficult to determine at the tunnel face, being directed to borehole characterization, and it was subsequently combined with parameter "discontinuity spacing" ("joint" spacing) - and named "spacing density" since the two are interrelated. For the best practical use, this led to the preferred use of "fracture frequency" as an invert of "fracture density" – as depicted in Fig. 7. None of these approaches altered the fundamental assignment of rating values to these parameters.

In this work, we re-examined the schematic summary diagram prepared by Bieniawski in 2013, refined the descriptive equation, and accordingly updated the diagram (Eq. 7 and Fig.7):

$$\text{RQD} + \text{Joint spacing} = -6.6 \cdot \left(\frac{Joints}{m}\right)^{0.46} + 40 \tag{7}$$

Therefore, when conducting engineering-geological descriptions and mapping of tunnels and rock slopes, we recommend continuing to use the JOINT FREQUENCY (number of joints per meter) instead of the distances between RQD (No. 2) and Joint Spacing (No. 3) among the six fundamental factors of the RMR89.

For RMR drill core documentation, we continue to recommend the use of the original RMR89, but for newer core documentation work, it is recommended to include a data column for recording joint frequency as well, so that this information to be available for later correlation studies. At the same time, we would like to point out that for rock material whose intact rock fragments do not exceed a UCS value of 6 MPa, we do not recommend using either the RQD or the RMR to characterize the rock mass and determine rock support systems.

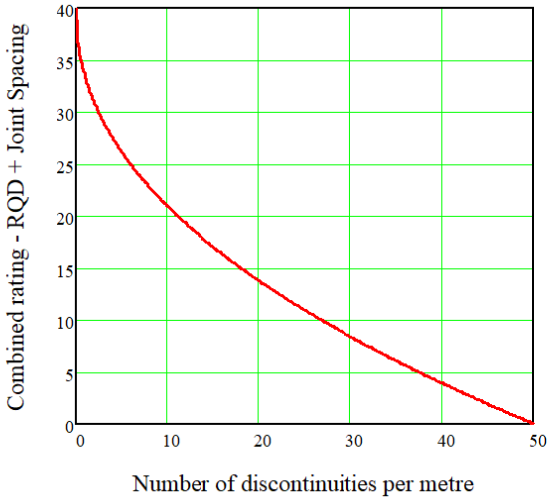


Fig. 7 Combined rating of the discontinuity density parameters: RQD plus discontinuity spacings.

4 Conclusions

The purpose of the brief summary presented here was to provide easy-to-use design charts and equations for the preliminary design of rock mass support based on RMR. The intention was to provide these guidelines in an easily accessible format, with some updates, and, where possible, along with the mathematical relationships, equations, and underlying logical and theoretical background used to construct the diagrams.

It must be emphasized that these charts are intended for preliminary design! This applies to all design diagrams and simplified methods presented here for the design of underground excavations. These are essentially expert systems developed to make design decisions similar to those an expert would make by following a set of rules.

It is also true, however, that while an expert may have deemed the rock support measures reasonable based on the information available before tunnelling begins, it may become apparent during construction that these initial plans need to be checked and modified. This may mean a greater or lesser need for rock support, or it may mean that, depending on the available equipment and the capabilities of the workforce, one type of rock support must be preferred over another. Rock mass conditions are inherently variable, and the stress state of the rock mass can differ dramatically from the "average" values assumed during design. Therefore, every contract must always provide for the ability to modify the preliminary rock support systems to suit the actual-real conditions.

All of the correlations and recommendations presented are based on the original RMR89 method for the analysis of drill cores, while for tunnels they are based on a modified version of RMR89 (omitting the RQD and spacing of discontinuities).

We excluded the use of RMR14 in our work, as decades of experience are available for the original RMR89; however, it should also be noted that, based on current international practice, RMR14 yields higher results than RMR89, so, particularly in the case of rock mass assessment, the balance of safety also leans toward a more conservative rock mass classification.

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