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Review

Review of Rock-Mass Rating and Tunneling Quality Index Systems for Tunnel Design: Development, Refinement, Application and Limitation

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Abstract: Although rock-mass rating (*RMR*) and tunneling quality index (*Q*) systems are used in different rock engineering projects as empirical design tools, their application in tunnel design is widely accepted as these systems were developed and updated for this purpose specifically. This paper reviews the work conducted by various researchers since the development of these two systems with respect to tunneling only. Compared to other empirical classification systems, these systems received international acceptance and are still used as empirical design tools in tunneling due to their continuous updates in the form of characterization and support. As the primary output of these systems is the initial support design for tunnel, however, their use in the calculation for rock-mass properties is an essential contribution of these systems in rock engineering design. Essential for the tunnel design, these rock-mass properties include the modulus of deformation, strength, Poisson's ratio, Mohr-Coulomb parameters and Hoek-Brown constants. Other application for tunneling include the stand-up time and rock load. The uses and limitations of these systems as empirical tunnel design tools are also included in this review article for better results. Research to date indicates that if the ground behavior is also taken into account, the application of these empirical systems will be more beneficial to the preliminary design of tunnels.

Keywords: *RMR*; *Q*; tunnel support; rock-mass properties; rock load; stand-up time; limitations

1. Introduction

In rock engineering design, the rock mass is considered as a complex natural geological building material and during tunnel construction, ground behavior is dependent on the ground condition and tunnel-related features [1]. The reason for the overall difficulty of modelling rock masses for engineering design is that the physical or engineering properties of rock mass must be established. This natural geological material is generally discontinuous, anisotropic, non-homogeneous and non-elastic [2] and produces uncertainties in modelling a rock mass. Modelling a rock-mass in a precise way allows scholars to decrease the ambiguity related with its characterization and replicate its inherent spatial inconsistency and heterogeneities [3]. For the purposes of rock engineering design, eight modelling methods have been categorized, including pre-existing standard methods, analytical methods, basic and extended numerical methods, precedent type analysis, empirical classification, the basic system approach and the combined system approach [4]. An important fact in rock mechanics is that having inadequate information for engineering design of structure in rock is a way of life, which is why the empirical rock-mass classification approaches have been established and are still essential for the tunnel construction.

The qualitative picture of rock masses through empirical classification systems and the succeeding correlation to design parameters has become one of the most challenging areas in rock mechanics. Since the first descriptive rock-mass classification system, rock load classification, was established in 1946 [5], numerous empirical systems have been suggested for rock masses with the consideration of a specific rock mass structure and purposes [6]. The empirical systems used in tunnels construction as design tool are summarized in Table 1. The rock load classification constituted the basis of modern rock classifications [7]. In their original form, the suitability of these rock mass classification systems, for the complex rock-mass conditions characterization is not always possible, which results in the development of new systems or the modification or extension of existing ones. Among the several empirical rock-mass classification systems developed so far, the *RMR* (rock-mass rating) system established by Bieniawski [8] and the Norwegian Geotechnical Institute (*NGI*) tunneling quality index (*Q*) system developed by Nick Barton and his co-researchers [9] are internationally accepted systems and are extensively used in tunneling. Additionally, these systems are also considered as the origin for launching other systems for the classification of rock-mass [7,10–12].

Table 1. The rock mass classification systems used in the design of tunnel.

S. No.	Classification System	Abbreviation	Applications	Year	Authors [References]
1	Rock load	-	Tunnels	1946	Terzaghi [5]
2	Stand-up time	-	Tunnels	1958	Lauffer [13]
3	Rock quality designation	*RQD*	General	1964	Deere [14,15]
4	Rock structure rating	*RSR*	Tunnels	1972	Wickham et al. [16]
5	Rock mass rating	*RMR*	tunnels	1973	Bieniawski [8]
6	Tunneling quality index	*Q*	Tunnels	1974	Barton et al. [9]
7	Geological strength Index	*GSI*	general	1995	Hoek et al. [12]
8	Rock mass Index	*RMi*	General	1995	Palmstrom [11]
9	Rock tunneling quality index by TBM excavation	Q_{TBM}	TBM tunnels	1999	Barton [17]
10	Continuous rock mass rating	*CRMR*	General	2003	Sen and Sadagah [18]
11	Rock mass excitability	*RME*	TBM tunnels	2006	Von Preinls et al. [19]
12	Rock mass quality rating	*RMQR*	General	2014	Aydan et al. [7]

The *RMR* and *Q* systems are admitted currently as a compulsory adjunct for evaluating rock-mass environments for engineering purposes. The two main objectives of these classification systems are easier communication between different users and as a decision-making tool [20]. The later objective is the remarkable practice of classification systems in rock engineering. The *RMR* system was suggested in 1973, as a jointed classification system for rock-mass [8]. This system was established on experience in underground projects in South Africa [21]. Afterwards, this system went through substantial variations. These variations are commonly due to ratings added or modified for ground-water, joint spacing and their condition, rock alterability, excavation method and stress–strain behavior. This system was numerously refined numerous times and main modifications in its characterization and structure were made in 1989 and 2014 [22,23]. The major changes in the structure of *Q* system were made in 2002 [24]. This system was established on tunneling case records for hard and jointed rock masses [25]. It was initially developed for the grouping of rock masses for assessing the need for supports in tunnels and caverns. Using a set of tables, for the characterization of rock mass, along with footnotes, the rating for various parameters can be recognized. The parameters rating can be based on the geological interpretations in tunnels during its construction, or by core logging during exploration. Through time, the *RMR* and *Q* systems have been updated so as to increase their trustworthiness for tunnel support. Along with tunnel support, these systems are also used to calculate the rock masses' geomechanical properties, the stand-up time for the tunnel and support load. Although these systems were developed, refined and updated for tunnel support design, these classification systems also have applications in various engineering projects like mining [26–32], slopes [10,33–37], dam foundations [38] and *TBM* (Tunnel Boring Machine) tunnels [17,19,39].

The *RMR* and *Q* systems have changed with time to reveal the apparent impact of several rock-mass factors on the stability of excavation. The focus of this review is on the historical progress and alteration of the two systems through rock-mass characterization and support recommendation with a particular emphasis on tunneling. The advanced versions of *RMR* and *Q*-system classification and the scope of its application in the field of tunneling are specified. The use of these systems in tunneling other than support recommendation is also discussed. The introduced changes have perhaps improved the applicability of *RMR* and *Q* systems in the field of tunneling but they have still limitations. The limitations of the two systems for use in the field of tunneling are also reviewed.

2. *RMR* and *Q* System Establishment

2.1. Development of the RMR System

To develop *RMR* system, the first three existing rock-mass classification systems of Table 1 were considered. Bienwiaski combined the best features of these available classification systems and supplemented them with the parameters that could be obtained from the data available from exploration, resulting in an *RMR* system, as shown in the Table 2 [8]. This *RMR* system, RMR_{73}, comprised eight parameters for classification of a jointed rock mass, in which each parameter has five important ratings. To rate the state of weathering, three different classifications were used: the Geological Society of London, Task committee of the American Society of Civil Engineers and the South African section of the Association of Engineering Geologists. The engineering classification of intact rock, proposed by Deere [15], was used in the *RMR* system with a slight modification. Classifications for joint spacing were made as per the suggestion by Deere [40]. Ground water and joint orientation were included as separate governing parameters in the *RMR* structure as per the study of Wickham, Tiedmann and Skinner [41]. Rating for all eight parameters were proposed based on this study. The summation of all the parameters determines the rock-mass quality. This system classifies the ground into five rock-masses.

Table 2. Characterization criteria in different versions of the rock-mass rating (RMR) system.

Parameter		*RMR*							
		1973 [8]	1974 [42]	1975 [43]	1979 [44]	1989 [22]	2011 [45]	2013 [46]	2014 [23]
Intact rock strength (MPa)		10–0	10–0	15–0	15–0	15–0	15–0	15–0	15–0
RQD (%)		16–3	20–3	20–3	20–3	20–3	20–0	-	-
Joint spacing (mm)		30–5	30–5	30–5	20–5	20–5	20–0	-	-
Discontinuity density (joints per meter)		-	-	-	-	-	-	40–0	40–0
Separation of joints (mm)		5–1	-	-	-	-	-	-	-
Continuity of joints (m)		5–0	-	-	-	-	-	-	-
Weathering		9–1	-	-	-	-	-	-	-
Condition of joints		-	15–0	25–0	30–0	30–0	30–0	30–0	20–0
Groundwater		10–2	10–2	10–0	15–0	15–0	15–0	15–0	15–0
Alterability (%)		-	-	-	-	-	-	-	10–0
Adjustment	F_0	15–3	15–3	0–(−12)	0–(−12)	0–(−12)	0–(−12)	0–(−12)	0–(−12)
	F_e	-	-	-	-	-	-	-	1.32–1
	F_s	-	-	-	-	-	-	-	1.3–1

2.2. Development of the Q System

This classification system was established in 1974 after the *RMR* system based on 212 tunnel cases. The development of this system was the result of the Seminar on Large Permanent Underground Openings held in Oslo in 1969. Two main breaches pointed out in the symposium were: (i) the acquisition of mechanical data and whether an excavation should be lined, or rock bolted, or unlined [47]; and (ii) the dilatant property of the rock had been unnoticed when designing rock-bolt spacing [48]. This system was independent from the *RMR* system but there are some parameters in common, that is, rock-quality designation (*RQD*) and groundwater. The system also included joint roughness, infilling of the joints and rock load, which were missed in the *RMR* system. The plot between *RQD* and the tunnel span for the tunnel case records [49] was the start of the development of support plot for the *Q* system [9]. The correlation was improved between *RQD* and the unsupported span by including the joint set number as an additional parameter. This modified *RQD* was further improved by including the joint roughness, joint alteration, water pressure and rock load. Afterward, an evaluation was done for case records in the literature, a clear picture was obtained between the rock-mass quality and excavation span. This trial-and-error and empiricism using more than 200 case records finally enabled an answer to the challenging question asked by the Norwegian State Power Board (Statkraft) from *NGI* in 1973: "Why are Norwegian powerhouses showing such a wide range of deformations?" [50,51]. This evaluation includes the tunnel span and purpose of excavation as additional parameters for the type and amount of rock support; however, they are not involved in the calculation of rock-mass quality as proposed for rock-mass classification [52]. The six parameters are united in the following manner to compute the rock-mass quality:

$$Q = \left(\frac{RQD}{J_n}\right) \times \left(\frac{J_r}{J_a}\right) \times \left(\frac{J_w}{SRF}\right) \quad (1)$$

where *RQD* is with a minimum score of 10, J_n symbolizes the score of the joint sets number, J_r is the score for the roughness of joint surface, J_a symbolizes the score for the clay filling or alteration degree, J_w symbolizes the groundwater pressure effects scores and the stress reduction factor (*SRF*) is the score for faulting, strength and stress ratios in hard rocks and squeezing or swelling. The system categorizes rock masses into nine classes.

3. Modification of the *RMR* and *Q* Systems Since Their Establishment

Both systems originated specifically for estimating rock tunnel support [53]. For this purpose, to estimate the adequate tunnel support measure, the quantitative valuation of the rock-mass quality is connected with an empirical rule for tunnel support design. For tunnel support design, these systems are refined and reviewed, in the form of support, characterization, or both.

3.1. Modification in Terms of Characterization

Empirical design approaches in rock tunnel construction are used because of their positive role and as an outcome of characterization and classification. The empirical classification systems include the main features affecting the rock mass to rate its class. These aspects are usually presumed to be independent and are called parameters, to which scores are assigned [54]. The different rock mass classification systems reflect different importance on various geological parameters. By description, characterization is the quantifying process of rock-mass for significant factors prevailing its behavior in tunneling, whereas their classification is the assessment method of rock-mass class according to a pre-defined classification system [55]. Ground conditions (rock mass along with stress and ground water) and project-related features (size and shape of the tunnel along with the excavation method) are liable for ground behavior [1]. The capacity of the characterization process to classify rock masses and the importance of containing as many factors as possible have been argued by Palmstorm and Stille [56].

3.1.1. *RMR* System

After its development, the *RMR* system has established many applications in several engineering projects, like tunnels, foundations, mines and slopes but utmost applications are in the field of tunneling [57]. This classification system was continuously refined [8,21–23,43–46] as more case records became available to approve it with global standard, as summarized in Table 2. In 1974, Bieniawski revised the system and presented its first alterations: weathering, aperture and persistence of joints were united within the term "discontinuity condition" with variations to the relative points [21]. Thus, the eight parameters creating the system was reduced to six. This revised *RMR* was again modified for intact rock strength (σ_c) and negative values were applied as adjustment parameters for the first time in 1975 [43]. The rating range for the first-class rock-mass (90–100) was changed to 81–100. The joint roughness was added as an additional parameter to the system and as a result, the assessment score was increased for the parameter "condition of joints." In 1979, modifications associated to the joint spacing, joint condition and ground water were made [44]. Main alterations to the *RMR* system were suggested in 1989 [22]. These comprised new graphs and tables for the score of σ_c, *RQD*, spacing and situation of discontinuities. The refined system is called RMR_{89}. In RMR_{89}, the basic *RMR* (RMR_b) score is the sum of the scores of five stated parameters. It comprises the score for σ_c (R_1), *RQD* (R_2), joint spacing (R_3), joint condition (R_4) and groundwater condition (R_5). The RMR_{89} system is the outcome once adjusting RMR_b for joint direction with respect to the tunnel alignment and direction of excavation score (R_6). The corresponding quality is achieved using Equation (2).

$$RMR_{89} = R_1 + R_2 + R_3 + R_4 + R_5 + R_6 \tag{2}$$

The minimum values of R_2 and R_3 for RMR_{73} version of *RMR* are 3 and 5 respectively, as listed in Table 2. A continuous-rating idea through equations was presented by Sen and Sadagah for R_2 and R_3 values of RMR_{73} system. According to RMR_{89}, the R_2 and R_3 values are from 0 to 20, after modification [45] (as shown in Table 2). The continuous-rating concept through equations, which was introduced by Sen and Sadagah for RMR_{73}, was extended for RMR_{89} to characterize R_2 and R_3 easily and precisely [58]. Since its development [59], *RQD* was used in *RMR* and *Q* system because of its historical background. Because of some intrinsic limitations and ignoring the basic definition [60–62], *RQD*, along with joints spacing, is substituted by joint frequency in the excavation face or rock core [46]. Koutsoftas [63] analyzed the arguments about the different definitions of *RQD* in different parts of the world and the use of rock exposures for determining *RQD* which are considered as the main sources of error. In fact, these are modifications made in the *RQD* for intended purposes and adopted wrongly. However, it cannot be considered as a system fault and is not sufficient reason to abandon the use of *RQD* in classification systems. The modified criterion shows that RMR_{89} can be stated in terms of five parameters and calculated using Equation (3):

$$RMR_{89} = R_1 + R_{2-3} + R_4 + R_5 + R_6 \tag{3}$$

where $R_{2\text{–}3}$ shows the rating for joint frequency.

After 25 years of use, an updated version of RMR_{89}, known as RMR_{14}, was proposed with new parameters, a revised rating and final structure [23]. The structure of *RMR* in RMR_{14} version is represented by Equation (4).

$$RMR_{14} = (RMR_b + F_0) \times F_s \times F_e, \tag{4}$$

where

RMR_b = *RMR* basic (without the effect of excavation),

F_0 = An adjustment factor like R_6 in RMR_{89},

F_e = An adjustment factor associated to the method of excavation, and

F_s = An adjustment factor associated to the stress-strain behavior at the tunnel face.

The discontinuity condition term is a function of discontinuity persistence, aperture, roughness, infilling and weathering. The corresponding score can be nominated for RMR_{89} using Table 3 but some situations are mutually exclusive; that is, in the case of infilling material, the roughness is overshadowed by the infilling influence [22]. The score for the discontinuity condition was reviewed in RMR_{14} and, because of the overshadowing effect, the aperture parameter is eliminated. The details are shown in Table 3.

Table 3. Rating comparison for joint condition in two different versions of *RMR*: RMR_{89} and RMR_{14}.

Parameter		Rating				
Persistence	Value	>20 m	20–10 m	10–3 m	3–1 m	<1 m
	RMR_{89}	0	1	2	4	6
	RMR_{14}	0	0	2	4	5
Aperture	Value	>5 mm	5–1 mm	1.0–0.1 mm	<0.1 mm	None
	RMR_{89}	0	1	4	5	6
	RMR_{14}	-	-	-	-	-
Roughness	Value	Slickenside	smooth	Slightly rough	rough	Very rough
	RMR_{89}	0	1	3	5	6
	RMR_{14}	0	1	-	3	5
Infilling	Value	Soft filling			Hard filling	
		>5 mm	<5 mm	>5 mm	<5 mm	None
	RMR_{89}	0	2	2	4	6
	RMR_{14}	0	2	2	5	-
Weathering	Value	Decomposed	Highly weathered	Moderately weathered	Slightly weathered	Not weathered
	RMR_{89}	0	1	3	5	6
	RMR_{14}	0	1	3	-	5

Additional parameters, namely the intact rock alterability, F_e and F_s, were also incorporated in RMR_{14}. The slake durability index is an important parameter to estimate qualitatively the durability of shale or other similar rock in the service environment and the intact rock alterability is rated per the results of the slake durability test, as defined in the standard (*ASTM D* 4644) [64]**.** The score of the rock alterability factor were proposed in tabular form in RMR_{14}. Mechanical excavation has a positive impact in tunneling [65] and this impact was introduced in the structure of the RMR_{14} system related to the excavation method. It can be obtained from Equation (5) [23].

$$F_e = 1 + 2 \times \left(\frac{RMR_{89}}{100}\right)^2, \ RMR_{89} < 40 \tag{5a}$$

$$F_e = 1.32 - \frac{\sqrt{(RMR_{89} - 40)}}{25}, \ RMR_{89} > 40 \tag{5b}$$

When the excavation is made by the drill and blast method, F_e = 1.

The *RMR* score at the tunnel face during excavation is significantly lesser than forecast during the design stage because of the yielding of the excavation periphery. To overcome this consequence, F_s is announced. This parameter was defined in terms of *ICE* (Índice de Comportamiento Elástico) [66], as stated in Equation (6). The Kirsch solution for stresses at the periphery of a circular tunnel was used for *ICE* calculation and has been prolonged to non-circular excavations using numerical modelling. For the derivation of Equation (6), Equation (7) [67] was used to calculate the rock-mass strength.

$$ICE = \frac{3704 \times \sigma_c \times e^{\frac{RMR_{89}-100}{24}}}{(3 - K_0) \times H} \times F, \ K_0 \ \leq 1 \tag{6a}$$

$$ICE = \frac{3704 \times \sigma_c \times e^{\frac{RMR_{89}-100}{24}}}{(3 \times K_0 - 1) \times H} \times F, \ K_0 \ \geq 1 \tag{6b}$$

$$\sigma_{cm} = \sigma_c \times e^{\frac{RMR_{89}-100}{24}}, \tag{7}$$

where

σ_c = intact rock uniaxial compressive strength (*UCS*) (MPa),
K_0 = virgin stress ratio,
H = tunnel depth (m), and
F = shape coefficient.

The value of *ICE* achieved from Equation (6) is used for the calculation of the adjustment factor, F_s, using Equation (8).

$$F_s = 1.3, \ ICE \ < 15 \tag{8a}$$

$$F_s = \frac{2.3\sqrt{100 - ICE}}{7.1 + \sqrt{100 - ICE}},\ 15\ < \text{ICE}\ < 70 \tag{8b}$$

$$F_s = 1,\ \text{ICE}\ > 70 \tag{8c}$$

3.1.2. *Q* System

Since the introduction, the *Q* system has not been tremendously updated like the *RMR* system in terms of characterization. The foremost modifications in characterization were suggested in relations of σ_c and *SRF* [68–70]. In the rock-mass properties, the role of σ_c is significant. Thus, a normalization factor is applied to Equation (1) for a modified Q_c:

$$Q_c = \left(\frac{RQD}{J_n}\right) \times \left(\frac{J_r}{J_a}\right) \times \left(\frac{J_w}{SRF}\right) \times \left(\frac{\sigma_c}{100}\right). \tag{9}$$

The initial maximum value of *SRF* for the condition of competent rock with a rock-stress problem was 20 [9]. The first relationship for *SRF* characterization was based on the principal field-stress ratio (*k*), cover depth of the tunnel (*H*) and σ_c [71]. The key changes were made for *SRF* characterization in hard massive rock in a high-stress environment and the rating for *SRF* increased from 20 to 400 [68]. These changes were based on the relation between RQD/J_n and *SRF* in hard rock under stress and no real stress problem was experienced for low values of RQD/J_n. The ratio RQD/J_n is the relative block size that is appropriate for differentiating massive, rock-burst-prone rock and the rock-burst-prone rock has RQD/J_n ratio from 25 to 200 whereas typical jointed rock mass has an RQD/J_n of 10 [24]. Tunneling in jointed rock under high stress is likely to be less hazardous than in massive rock because the rock-bursting phenomenon is less severe [72]. For Australian mining, where the high stresses are either due to mining depth or the advancing mining front acting on jointed rock, a relation was proposed by Peck for a low *SRF* (*SRF* = *f* (strength stress ratio)) value [73], which was confirmed by Barton [74]. The continuous rating of *SRF* based on tunneling data under a highly stressed jointed rock-mass environment was modified such that *SRF* is a function of relative block size (RQD/J_n), intact rock strength (σ_c) and strength-to-stress ratio (σ_c/σ_1) [70].

3.2. Development in Terms of Support

Keeping the basic purpose of the two systems, the *RMR* and *Q* systems were also updated in terms of support guidelines along with improved characterization for rock-quality determination.

3.2.1. *RMR* System

The initial guidelines were for drill and blast excavated tunnel support and were based on the rock-mass class in a tabulated form for a tunnel span from 5 to 12 m [8]. The length of fully resin-grouted rock bolts (25 mm in diameter) was purely based on the tunnel width, where the length of a rock bolt is half of the tunnel width. The other supporting materials were shotcrete, steel sets and wire mesh. In the 1976 version of *RMR*, the support guideline table was updated for a 10-m-wide tunnel with a horseshoe shape [75]. The rock bolts length and the thickness of shotcrete was reduced. In 1979, the length of the 20-mm-diameter fully grouted rock bolts was made a function of the *RMR* value [44]. The limit of vertical stresses was specified. Separate guidelines were suggested for support in walls and the crown of the tunnel. In 1989, the support chart remained unchanged and refinement of the system was made in characterization. Support technology upgraded for tunnels with time and thus, the designers used support for different tunnel spans [76–78]. The foremost update in support using *RMR* system was accomplished, after twenty four years of experience, in which the tunnel support is a function of the tunnel size and rock-mass score [46]. In this revision, the shotcrete thickness and rock bolt length (L_b) are functions of rock-mass score and tunnel span. The length of the rock bolt can be calculated using Equation (10).

$$span\ (m) = \frac{(L_b(m) + 2.5)^{\frac{RMR_{89}+25}{52}}}{3.6} \quad (10)$$

Rock-bolt spacing (S_b) is the function of RMR score only. When $RMR_{89} > 85$, spot bolting is required. In shotcrete design graph, the shotcrete thickness is greatly influenced by the tunnel size at low RMR_{89} score, whereas shotcrete thickness is greatly influenced by RMR_{89} for larger tunnel spans (Span > 10 m) [46]. The rock-bolt spacing can be obtained from Equation (11).

$$S_b(m) = 0.5 + 2.5 \times \frac{RMR_{89} - 20}{65},\ 20\ < RMR_{89}\ \leq\ 85 \quad (11a)$$

$$S_b(m) = 0.25 + \frac{(RMR_{89} - 10)^{1.5}}{140},\ (10\ < RMR_{89}\ \leq\ 20 \quad (11b)$$

$$S_b(m) = 0.25,\ RMR_{89}\ \leq\ 10 \quad (11c)$$

3.2.2. *Q* System

To classify a rock mass for its quality, *Q* system was updated by continuous re-analyzing of the tunnel case record until a reliable connection was achieved between *Q* value, the equivalent dimension (D_e) and the actual support used for the stability of the tunnel [9]. These three variables are organized by means of a support chart in the *Q* system of rock-mass classification. The D_e is a function of the excavation size and its purpose and this purpose of the excavation was expressed in terms of the excavation-support ratio (*ESR*). The conclusion of the support recommendation chart was 38 box categories of permanent support. This support was for the crown of the tunnel. For the tunnel wall support, *Q* was increased up to 5 times and then the support was recommended. The recommended supports were bolting (spot or systematic), grouting, shotcrete (plain or mesh-reinforced) and cast-concrete arches (plain or steel-reinforced). In the 1970s, the wet process of shotcrete along with fiber reinforcement was a revolution in tunnel support. The innovation in shotcrete technology was summarized by Barrett and McCreath [79]. In 1993, mesh-reinforced shotcrete was replaced by fiber-reinforced shotcrete [68]. The 38 box categories are simplified to the graphical method (9 reinforcement categories) based on 1050 more tunnel case records. Rock-bolt spacing was correlated to the rock-mass quality value in both shotcrete and non-shotcrete area. Out of the 9 reinforced categories, reinforced ribs of shotcrete were also added for one category. Based on numerical modelling [80,81] and empiricism, the *Q*-system support chart was again updated by Grimstad for reinforced ribs of shotcrete [82]. The thickness of shotcrete was increased due to the widespread application of shotcrete in tunneling. The diameter of the rock bolt was specified as 20–25 mm in diameter. In the latest support chart [25], the bolts spacing is quantified for a 20-mm diameter and detailed adjustments are included for the tunnel wall support. It is also mentioned that the shotcrete thickness for excavation with D_e less than 3 m is just supposed and is not empirically based. As in the *RMR* system, bolt spacing is a function of *Q* value and thickness of shotcrete is function of the D_e along with the *Q* value.

4. Application in Determination of Rock-Mass Properties

The determination of rock mass mechanical properties is one of the essential parts of rock engineering design. To consider the scale effect for rock masses, field tests are desired, as laboratory testing on rock masses is not easy and is very unwieldly. However, field tests to obtain these properties are time-consuming, difficult to perform and expensive. Therefore, correlation between the *RMR* and *Q* with mechanical properties of rock masses have been made. The mechanical properties include not only the deformation modulus (E_m) and *UCS* (σ_{cm}) of rock masses but also Poisson's ratio (ν_m), frictional angle (ϕ_m), cohesion (c_m), Hoek-Brown constant and tensile strength (σ_{tm}), among the others, which are critical for tunnel design. Correlations between empirical rock-mass classification systems and mechanical rock-mass properties are a common practice observed in rock engineering, as they create an easy approach for assessing the rock-mass conditions.

4.1. Deformation Modulus of Rock Masses

The uncertainties are associated to in-situ E_m calculations due to blast damage, experiment method and procedure and therefore, a good rock mass characterization may give equivalent, or possibly better E_m values [83]. In the literature, several correlations have been suggested for the E_m and the rock-mass quality (*RMR* and *Q*) values since the first empirical equation between *RMR* and E_m by Bienwiaski [84] and the first empirical equation between *Q* and E_m by Barton [85]. In these equations, the intact rock properties were missing. Later, it was determined that, for the calculation of rock-mass mechanical properties (E_m, σ_{cm}, ν_m, ϕ_m, c_m and σ_{tm}), the intact rock properties (E_i, σ_c, ν_i, ϕ_i, c_i and σ_t respectively) and rock-mass classification indexes must be involved [86]. The proposed correlations between the deformation modulus of rock mass and the *RMR* or *Q* values can be divided into two groups:

- The E_m is calculated independently from the E_i, Table 4.
- The E_m is calculated dependently from the E_i, Table 5.

Table 4. Equations for calculation of E_m without E_i.

S. No.	Equation	Equation No.	Reference
1	$E_m = 2 \times RMR_{76} - 100(\text{GPa})$, $RMR > 50$	(12)	[84]
2	$E_m = 10^{\frac{RMR_{76}-10}{40}}(\text{GPa})$	(13)	[87]
3	$E_m = 0.3 \times H^{\alpha} \times 10^{\left(\frac{RMR_{79}-20}{38}\right)}(\text{GPa})$, H > 50 m	(14)	[88]
4	$E_m = e^{(4.407+0.081\times RMR_{89})}(\text{GPa})$	(15)	[89]
5	$E_m = 0.1 \times \left(\frac{RMR_{89}}{10}\right)^3(\text{GPa})$	(16)	[90]
6	$E_m = 0.0097 \times RMR_{89}^{3.54}(\text{MPa})$	(17)	[86]
7	$E_m = 0.0876 \times RMR_{89}(\text{GPa})$, $RMR \le 50$ $E_m = 0.0876 \times RMR_{89} + 1.056 \times (RMR_{89} - 50) + 0.015 \times (RMR_{89} - 50)^2$, $RMR > 50$	(18a) (18b)	[91]
8	$E_{rm} = 1.35 \times e^{0.047 RMR_{89}}(\text{GPa})$	(19)	[92]
9	$E_m = c \times \log Q(\text{GPa})$, $Q > 1$	(20)	[85]
11	$E_m = 10 \times Q^{\frac{1}{3}}(\text{GPa})$	(21)	[69]
12	$E_m = 10 \times \left(Q \times \frac{\sigma_{ci}}{100}\right)^{\frac{1}{3}} = 10 \times Q_c^{\frac{1}{3}}(\text{GPa})$	(22)	[24]
13	$E_m = 10^{(15\times\log Q+40)/40}$, $Q < 1.0$ and $RMR < 50$	(23)	[24]

Where α is a constant in Equation (14) and its value varies from 0.16 to 0.3. This equation is suitable for dry conditions. c in Equation (20) is a general constant for $Q > 1.0$ that ranges from 10 to 40 with a mean value of 25 [68,93]. The applications of Equation (12) and (13) are extended for RMR_{89}. RMR_{76} and RMR_{79} are the 1976 and 1979 version of *RMR*, respectively.

Table 5. Equations for calculation of E_m with E_i.

S. No.	Equation	Equation No.	Reference
1	$\frac{E_m}{E_i} = \left(0.9 \times e^{\frac{RMR_{79}}{22.82}} + 0.0028 \times RMR_{79}^2\right) \times \frac{1}{100}$	(24)	[94]
2	$\frac{E_m}{E_i} = 0.05\left(1 - \cos \pi \frac{RMR_{89}}{100}\right)$	(25)	[95]
2	$\frac{E_m}{E_i} = \frac{RMR_{89}}{RMR_{89}+\beta(100-RMR_{89})}$, $\beta = 6.0$	(26)	[96]
4	$\frac{E_m}{E_i} = \frac{e^{(RMR_{89}-100)}}{22.94}$	(27)	[93]
5	$\frac{E_m}{E_i} = e^{-0.0035(5(100-RMR_{89})}$	(28)	[97]
6	$\frac{E_m}{E_i} = e^{(RMR_{76}-100)/36}$	(29)	[91]
7	$\frac{E_m}{E_i} = 10^{[(RMR_{89}-100)^2/4000\times e^{(-RMR_{89}/100)}]}$	(30)	[98]
9	$\frac{E_m}{E_i} = e^{[-0.0035[250(1-0.3\times\log Q)]]}$	(31)	[97]

4.2. Strength of Rock Masses

Construction materials used in mining and civil construction are categorized according to their strength properties. This basic quality information of the material is used in engineering and design for various construction purposes. Although rock is considered a construction material, in rock engineering, no such precise strength characterization of the rock mass is applied. Although the various utilizations of rocks and rock masses have different purposes and are subjected to various problems, the suitability and quality of the rock mass depends largely on its strength properties. Reliable determination of rock-mass strength is crucial for underground excavation [11,57,99,100]. The development [12,101] and update of the *GSI* (geological strength index) system [102] and its incorporation in the Hoek-Brown empirical failure criterion [103] is mainly for the reliable calculation of σ_{cm}. Like the empirical failure criteria, the use of empirical classification systems for the calculation of σ_{cm} is a common practice in rock engineering through correlation. The proposed correlations between the σ_{cm} and the *RMR* or *Q* values can also be divided into two groups:

- The σ_{cm} calculated independently of the σ_c, Table 6.
- The σ_{cm} calculated dependently of σ_c, Table 7.

Table 6. Equations for calculation of σ_{cm} without σ_c.

S. No.	Equation	Equation No.	Reference
1	$\sigma_{cm} = 7 \times \gamma \times Q^{1/3}(\text{MPa}), \sigma_c > 2\text{ MPa}, Q < 10, J_w = 1$	(32)	[104]
2	$\sigma_{cm} = 0.0016R \times MR_{89}^{2.5}(\text{MPa})$	(33)	[86]
3	$\sigma_{cm} = 5 \times \gamma \times Q_c^{1/3}(\text{MPa})$	(34)	[24]
4	$\sigma_{cm} = 0.5 \times e^{0.06RMR_{76}}(\text{MPa})$	(35)	[105]

Where γ is the unit weight in t/m^3.

Table 7. Equations for calculation of σ_{cm} including σ_c.

S. No.	Equation	Equation No.	Reference
1	$\frac{\sigma_{cm}}{\sigma_c} = \sqrt{e^{\left(\frac{RMR_{76}-100}{9}\right)}}$	(36)	[99]
3	$\frac{\sigma_{cm}}{\sigma_c} = e^{\left(\frac{RMR_{74}-100}{18.75}\right)}$	(37)	[106]
4	$\frac{\sigma_{cm}}{\sigma_c} = e^{\left(\frac{RMR_{89}-100}{24}\right)}$	(38)	[67]
5	$\frac{\sigma_{cm}}{\sigma_c} = \frac{7}{100}\gamma Q^{1/3}, \gamma \text{ in t/m}^3$	(39)	[107]
6	$\frac{\sigma_{cm}}{\sigma_c} = e^{\left(\frac{RMR_{76}-100}{20}\right)}$	(40)	[108]
7	$\frac{\sigma_{cm}}{\sigma_c} = \frac{RMR_{89}}{RMR_{89}+6(100-RMR_{89})}$	(41)	[109]

4.3. Poisson's Ratio Value

Compared to the other basic mechanical properties of the rock mass, Poisson's ratio is an elastic constant whose importance is generally underrated [110]. The behavior of rock masses is influenced by the mechanical behavior and properties of discontinuities and of the intact rock bounded by discontinuities. The *UCS* test on a rock mass (σ_{cm}) indicated that ν_m has an inverse relation with the σ_{cm} [111], as expressed in Equation (42). As *RMR* and *Q* systems are used for calculation of σ_{cm}, these systems can be used indirectly for the calculation of ν_m.

$$\nu_m = 0.25\left(1 + e^{-0.2\sigma_{cm}}\right) \tag{42}$$

A direct method of determining ν_m from the *RMR* value was proposed [93,112] and is expressed in Equation (43).

$$\nu_m = 0.5 - 0.2\frac{RMR_{89}}{RMR_{89} + 0.2(100 - RMR_{89})} \tag{43}$$

This equation was modified and according to this modification, the ratio of ν_m and ν_i is a function of the corresponding *RMR* value [93], as shown in Equation (44).

$$\frac{\nu_m}{\nu_i} = 2.5 - 1.5 \times \frac{RMR_{89}}{RMR_{89} + (100 - RMR_{89})} \tag{44}$$

4.4. Mohr-Coulomb Parameters

Mohr-Coulomb model is an elastoplastic model based on the Mohr-Coulomb failure criterion and is the most common model in the context of geomaterials. When using this model, it is essential to estimate the c_m and the ϕ_m. Bienwiaski correlated the rock-mass class with corresponding c_m and ϕ_m in a discrete manner [44]. This discrete approach is used in literature for the calculation of the cohesion and internal friction angle [78,113,114]. The correlation between the σ_{cm} and the ϕ_m were made through the best-fit analysis, which shows that the ϕ_m tends to converge to a value of about 50° as σ_{cm} increases [111]. As the *RMR* and *Q* values are used for calculation of σ_{cm}, these systems can be used indirectly for the calculation of the ϕ_m using Equation (45).

$$\phi_m = 20 \times \sigma_{cm}^{0.25} \tag{45}$$

Later, a direct relation between the rock-mass internal friction angle and the *RMR* value was developed [115], as shown in Equation (46), taking the intact rock friction angle (ϕ_i) into consideration:

$$\frac{\phi_m}{\phi_i} = 0.3 + 0.7 \times \frac{RMR_{89}}{RMR_{89} + \beta(100 - RMR_{89})}. \tag{46}$$

β is a constant and its value varies between 0.1 and 3. The revised value for β is 1.0 and Equation (46) is simplified to Equation (47) [112].

$$\frac{\phi_m}{\phi_i} = 0.3 + 0.7 \times \frac{RMR_{89}}{100} \tag{47}$$

From the σ_{cm} and ϕ_m, Aydan and Kawamoto suggested Equation (48) for the calculation of rock-mass cohesion [116] as cited by [117].

$$c_m = \frac{\sigma_{cm}}{2} \times \frac{1 - \sin\phi_m}{\cos\phi_m} \tag{48}$$

As *RMR* and *Q* values are used for the estimation of σ_{cm} and ϕ_m, Equation (48) indirectly uses the empirical classification systems for the rock-mass cohesion (c_m). Later, a direct relation between c_m and the *RMR* value was developed and, according to this relation, the c_m is a function of intact rock cohesion (c_i) and *RMR* [112], as expressed in Equation (49).

$$c_m = c_i \times \frac{RMR_{89}}{RMR_{89} + 6(100 - RMR_{89})} \tag{49}$$

Sen and Sadagah also suggested a continuous system for the calculation of cohesion and the frictional angle of rock masses [18] using Equations (50) and (51).

$$c_m = 3.625 \times RMR_{73} \tag{50}$$

$$\phi_m = 25(1 + 0.01RMR_{73}),\ \mathrm{RMR}_{73} \geq 20 \tag{51a}$$

$$\phi_m = 1.5 \times RMR_{73},\ \mathrm{RMR}_{73} < 20 \tag{51b}$$

Barton split the modified *Q* system from Equation (9) into two components and designated them the c_m and ϕ_m [24], as shown in Equations (52) and (53), respectively.

$$c_m = \left(\frac{RQD}{J_n} \times \frac{1}{SRF} \times \frac{\sigma_{ci}}{100}\right) \tag{52}$$

$$\phi_m = \tan^{-1}\left(\frac{J_r}{J_a} \times \frac{J_w}{1}\right) \tag{53}$$

4.5. Hoek-Brown Constants and Tensile Strength of Rock Mass

For the subsurface excavations design in hard rock, Hoek and Brown introduced their empirical failure criterion, known Hoek-Brown failure criterion [99,118]. The criterion for rock masses is accepted widely in a huge number of excavation projects worldwide [103]. The development history of this empirical failure criterion [119] shows that *RMR* served this criteria before replacing it with the *GSI* system [12] for the calculation of the rock-mass Hoek-Brown constants *m* and *s*. The original Hoek-Brown failure criterion is shown in Equation (54) for intact rock and the generalized Hoek-Brown failure criterion for rock mass is expressed in Equation (55) [103].

$$\sigma_1' = \sigma_3' + \sqrt{m_i \times \sigma_3' \times \sigma_c + \sigma_{ci}^2} \tag{54}$$

$$\sigma_1{'} = \sigma_3' + \sigma_{ci} \times \left(m_b \times \frac{\sigma_3'}{\sigma_c} + s\right)^a, \tag{55}$$

where m_b in the above equations is a reduced value for the rock mass of the material constant m_i and *s* and *a* are constants for the rock mass. The rock-mass constants m_b and *s* were expressed in terms of the basic *RMR* before the introduction of *GSI* in the Hoek-Brown criterion [22,120], as shown in Equations (56)–(59).

Disturbed rock masses:

$$\frac{m_b}{m_i} = \exp\left(\frac{RMR_b - 100}{14}\right) \tag{56}$$

$$s = \exp\left(\frac{RMR_b - 100}{6}\right) \tag{57}$$

Undisturbed or interlocking rock masses:

$$\frac{m_b}{m_i} = \exp\left(\frac{RMR_b - 100}{28}\right) \tag{58}$$

$$s = \exp\left(\frac{RMR_b - 100}{9}\right) \tag{59}$$

These rock-mass constants were also expressed in *Q*′ (*Q* without *SRF* and J_w) value, as expressed in Equations (60) and (61) [104].

$$\frac{m_b}{m_i} = 0.135 \times (Q')^{\frac{1}{3}} \tag{60}$$

$$s = 0.002 \times Q' \tag{61}$$

By substituting σ_1=0 in the original Hoek-Brown Equation (54), the resultant equation, Equation (62), determines the σ_t.

$$\sigma_t = \frac{\sigma_c}{2} \times \left(m_b - \sqrt{m_b^2 + 4 \times s}\right) \tag{62}$$

This equation indirectly yields the σ_{tm} from the *RMR* and *Q* values, as m_b and s are functions of these systems.

The direct method for the calculation of the σ_{tm} from the *RMR* value is given in Equation (63) [121].

$$\frac{\sigma_{tm}}{\sigma_t} = \frac{RMR}{RMR + 6(100 - RMR)} \tag{63}$$

5. Other Applications

5.1. Stand-Up Time

In subsurface openings, time-dependent loss of the rock mass is normal, causing either accelerated damage or slow damage [122]. A practical consequence of time dependency in tunneling is the amount of time the excavated rock mass can remain unsupported before instabilities occur, resulting in failure and collapse. This period of time to collapse is known as the stand-up time. The stand-up time was established as an empirical classification system [13]. According to this classification, the rock mass was classified into seven classes and a relation between the rock-mass classes, stand-up time and active span was established. The stand-up time classification was one of the three classification systems that were considered during the development of the *RMR* system and a chart was developed to correlate the predicted stand-up time for the unsupported span of the tunnel and the *RMR* score [8]. Along with the update of the *RMR* system for tunneling, the stand-up time chart was also updated for drill and blast tunnel [22] and was also revised for *TBM* tunnel [123].

In the *Q* system, instead of providing the stand-up time, the system suggests the span of the unsupported tunnel based on the tunnel-quality index value, that is, *Q*. The unsupported span can be calculated using Equation (64) [9].

$$D_e = 2 \times Q^{0.4} \tag{64}$$

The general condition requirement for the unsupported span was expressed for different ESR values [85]. Based on the correlation between the *RMR* and *Q* systems, the stand-up time for an unsupported span was extended for the *Q* system [53], as shown in Figure 1.

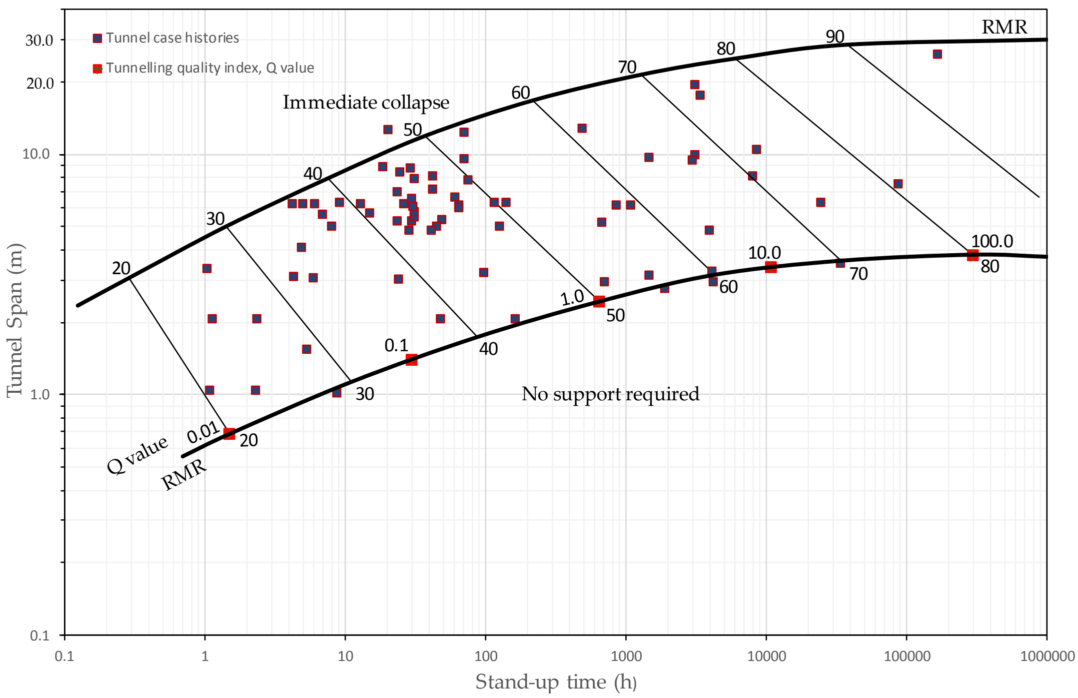

Figure 1. Stand-up time versus unsupported span of the tunnel for different *RMR* and *Q* values.

5.2. Rock Load

In underground excavations, generally, the load on installed support is known as rock load. It indicates the rock pressure that results from the rock-load height above the underground excavation. Reliable forecast of rock load is one of the toughest jobs in rock engineering. Terzagi was the first whose rock-mass classification system,—that is, rock-load classification—was used to estimate the rock load to be carried by the steel arches installed to support a tunnel [5], as shown in Figure 2.

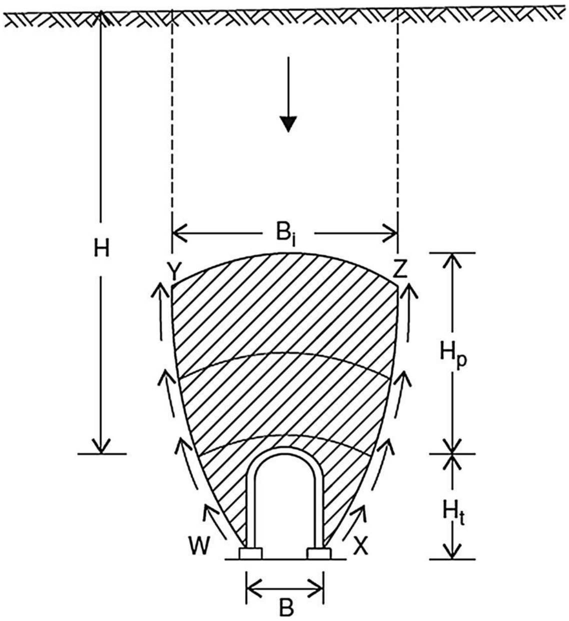

Figure 2. Development of loosened ground zone in response to tunnel construction. B and H_t are the width and height of the tunnel, respectively, H_p and B_i are the height and width of the arching zone, respectively and H is the depth of the tunnel from the ground surface.

Since then, different methods, including empirical, analytical, numerical and experimental approaches, were used for prediction of the rock load [9,124–126]. According to Barton [9] and supported by Verman [88], the rock load is free of the tunnel size in rock and Barton expressed this using Equation (65).

$$P = \frac{2 \times J_n^{1/2} \times Q^{-1/3}}{3 \times J_r} \ (\mathrm{kg/cm^2}) \tag{65}$$

The correlation of Equation (65) has proven useful, except in cases of squeezing-ground conditions. For squeezing ground, three adjustment factors were applied to Barton's original equation, including the tunnel depth factor, tunnel radius factor and correction factor for time [127,128]. The application of the *RMR* system was introduced for the support load by Unal [124] based on the old version of *RMR* and extended for RMR_{89} using Equation (66). He defined the rock load as unstable zone height above the excavation crown, which tends to collapse ultimately.

$$P = \frac{100 - RMR_{89}}{100} \gamma B, \tag{66}$$

where p is the support load (kN), B is the tunnel width (m) and γ is the rock density (kg/m^3).

Since the above equation is based on a coal mine roadway, the evaluation of this equation for rock tunnel applications shows that the estimated support pressure is unsafe in squeezing conditions [128]. Furthermore, the estimates for the non-squeezing condition are unsafe in the case of a small tunnel and are overestimated for a large tunnel. The following correlations (Equations (67) and (68)) have been proposed for a rock tunnel, taking the tunnel depth and tunnel radius as two additional parameters with *RMR* for non-squeezing and squeezing conditions [128].

For non-squeezing ground:

$$P = (2.32 - 0.035 \times RMR_{89} + 0.001 \times H + 0.03 \times a)\,,\, MPa \tag{67}$$

For squeezing ground:

$$P = (f(RMR_{89})/12) \times 10^{\left(1.8H^{0.4} \times a^{0.1} / RMR_{89}{}^{1.2}\right)}\,,\, MPa, \tag{68}$$

where P is the ultimate support pressure, $f(RMR_{89})$ is the correction factor for the tunnel closure, H is the tunnel depth (m) and a is the radius of the tunnel (m).

Due to the widespread acceptance of *GSI* in rock engineering, Equation (66) is considered the core origin for the Equation (69), which is based on *GSI* for the calculation of support pressure [129].

$$P = \frac{100 - \left[\left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_{cr}}{100}GSI}\right]}{100} C_s S_q \gamma D_e, \tag{69}$$

GSI defines the quality of rock-mass, D is the disturbance factor, σ_{cr} is the residual compressive strength of the rock mass in the broken zone around the tunnel, D_e is the equivalent diameter of the excavation, γ is the unit weight of rock mass, C_s is the correction factor for the horizontal-to-vertical field stress ratio and S_q is the correction factor for the squeezing-ground condition.

6. Recommended Procedure for Using *RMR* and *Q* Systems and their Correlation

After *RMR* and *Q* systems had been used in the field of tunneling worldwide, Nick Barton and Z. T. Bieniawski provided the "ten commandments" for the proper use of *RMR* and *Q* systems [53] as follows:

- Use standard procedure of measurement for the parameters instead of only description.
- Determine the ranges and average values of *RMR* and *Q* per their classification procedure.
- Use *RMR* and *Q* systems and check them with published correlation.
- Estimate the support and rock-reinforcement requirements.
- For preliminary modelling, determine the stand-up time and modulus of rock-mass.
- For checking, perform numerical modelling.
- If the available information is inadequate, extend the exploration work.
- Consider the construction process.
- Include the characterization information of rock-mass along with the design procedure, specifications and assumptions in the Geotechnical Baseline Report.
- In the construction phase, calculate *RMR* and *Q* for design verification and modification.

A back-analysis methodology for *RMR* and *Q* calculation, as shown in Figure 3, was proposed using the installed support and used to evaluate and extend the application of these classification in the field of tunneling [70]. The variation of rock-mass quality in underground excavation is always significant. However, the forecast of *RMR* ahead of the excavation face in tunneling, up to certain distance, is reliable using the rock-mass quality of the current working face of the tunnel [130]. It is worth emphasizing that use at least two empirical classification systems in the tunnel-support design process [131]. When using both systems, they should be checked through correlation [53]. After the development of *RMR* and *Q* systems, a linear correlation based on the regression analysis of *RMR* and *Q* values was presented by Bienwiaski [75], as shown in Equation (70), where A = 9 and B = 44.

$$RMR = A \times \ln Q + B \tag{70}$$

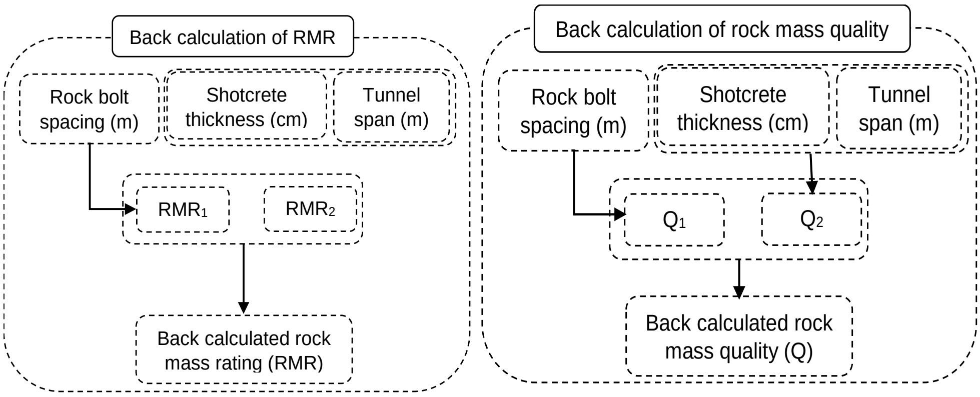

Figure 3. Back-calculation technique for *RMR* (left) and *Q* system (right) using tunnel span along with rock-bolt spacing and shotcrete thickness [58].

Since then, several researchers have presented other correlations but there is no scientific basis to assume a universally valid regression between the two systems. The limitations in correlation is due to the differences between the rock-mass characterizations and rating rules of *RMR* and *Q* system [132]. Due to this constraint of correlation, some scholars propose local, project-based, or geological-based correlation between the two systems [132–136]. Another suggested approach for the correlation is between the truncated *RMR* and *Q* values [54,128]. The latest version of *RMR*, that is, RMR_{14}, is not as popular as RMR_{89}; however, an excellent correlation between the two versions exists [23,58,137], as shown in Figure 4.

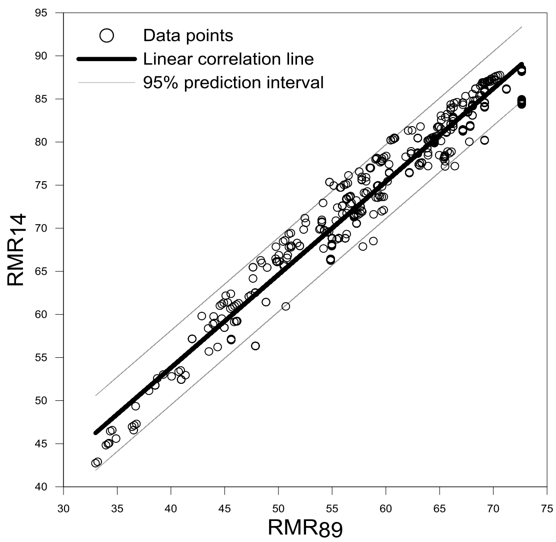

Figure 4. Correlation between the two versions of *RMR* [58].

7. Limitations of *RMR* and *Q* Systems

Rock-mass classification systems are frequently used in rock engineering and design and have gained wide attention in this field; however, these classification systems have limitations. Nevertheless, if applied appropriately, they are valuable design tools [138]. One of the many reasons for updating the two systems is to point out their limitations. These updates kept the two systems viable to date and also expanded their applications in rock engineering. Some limitations found were inherent to the two systems, which have been proven from their use over time, whereas the others were overcome with improvements. The inherent limitations of these systems formed basis for new classification systems. Stille and Palmstorm pointed out six necessities for a true rock mass classification system and, according to them, none of these systems fulfill these requirements [20]. However, Hand distinguished between the two main types of classification, that is, unsupervised and supervised classification [139]. The *RMR* and *Q* systems belong to the supervised classification because the rock-mass class structure is logical and the principles of separation are expressed so as to allow one to distribute objects to their suitable class [20].

Many empirical systems in rock engineering have been suggested with the respect of a specific rock-mass structure and/or particular purposes. Rock mass related professionals incline to the empirical method over the theoretical and numerical methods due its simplicity [128]. However, these empirical approaches are unable to characterize complex rock-masses. This is probably one of the causes why rock engineers have continued to develop new systems or modify and extend current ones [7]. Each classification has its own capabilities and therefore they are used in parallel as much as possible. The limitations can be broadly divided into three types, as described in the following sections.

7.1. Limitations Related to RMR System

The key benefit of the *RMR* classification is that it is easy to practice. However, this system is comparatively insensitive to slight changes in *RMR* value. Taking ground behavior in consideration, adequate support measure be selected with clear understanding in tunnel design [1]. Based on ground behavior and the available tools for underground excavation in rock engineering, the application of *RMR* systems are the lowest among the classification systems [56], as shown in Table 8. This system is based on a comparatively small case record, which makes the *RMR* system less appropriate as an empirical design technique for rock-mass support, particularly when in-situ stresses or time-dependent rock mass properties are of importance for the rock engineering design [20]. In the *RMR* system, the rock-mass quality is divided into five classes but these classes are scarcely comparable due to the wide variation of rock-mass quality within a single class [140]. As the classification and design systems develop, the old forms of rock-mass classification systems are not always well-matched with new design approaches. Presently, usual mistakes of *RMR* system applications primarily deal with the use of old versions of the system [141], for example, the use of RMR_{76} for hard rock-stability analysis for a span in entry-type excavation [142]. Although this system is improved tremendously in terms of characterization and support, it is noticeable that the application of RMR_{89}, RMR_{14} systems for tunnel-support design in high-stress locations is still a limitation of the system [58]. Although the positive impact of mechanical excavation is incorporated in the latest version of *RMR*, that is, RMR_{14} [23], the mechanical excavation impact is negative in high-stress environments and drill and blast excavation has a positive impact against rock bursting [143,144].

Table 8. The fitness of empirical rock-mass classification systems for different ground behavior during tunnel excavation.

Ground Behavior	Empirical Rock-Mass Classification System		
	RMR	***Q***	***RMi***
Stable	2	2	1–2
Fragment (s) or block (s) fall	1–2	1–2	1–2
Cave-in	3	2–3	2
Running ground	4	4	4
Buckling	4	3	3
Rupturing from stress	4	3	3
Slabbing, spalling	4	2	2
Rock burst	4	3–4	2
Plastic behavior (initial)	4	3–4	3
Squeezing ground	4	3	3
Raveling from slaking or friability	4	4	4
Swelling ground	4	3	3
Flowing ground	4	4	4
Water ingress	4	4	4

Fitness rating of different systems: 1, suitable; 2, fair; 3, poor; and 4, not applicable. *RMi* is the rock-mass index system for rock engineering purposes [11].

7.2. Limitations Related to Q System

Empirical design methods have been accepted for a long time, owing to the lack of appropriate design methods and undeveloped theory, which show quite adverse results during construction [145]. An important benefit of the *Q* system is that this system is based on well-documented case records for its original development. However, compared to *RMR*, this system is not covering new fields in its use. Therefore, the support chart of *Q* system gives only a suggestion of the support to be applied and this support system should be tempered by practical and sound engineering judgement [138]. In this system, RQD/J_n is an indication of relative block size, however, this is not understandable [146], which was supported by Palmstorm [60]. The effect of water in the *Q* system was discussed by Palmstorm through example, which shows that in exceptionally high-flow cases, the *Q* system is unable to recommend suitable support [138].

The application of *SRF* is unclear for buckling, rock-burst, and/or squeezing conditions, or for weakness zones [56]. The *SRF* values were adjusted in 1993, in overstressed massive brittle rocks based on limited data [68]. This increase is from a value of 20 to 400 based on the strength–stress ratio but no criteria have been selected for the rating selection of *SRF* even though *SRF* was a fine tuning parameter [9]. In case of squeezing rock, Barton selected the range of *SRF* from 5 to 20 [9]; however, this *SRF* range does not effectively characterize the squeezing influence due to the deficiency of enough case records and resulting in unsafe tunnel practice when using the *Q* system for large underground excavations in squeezing ground [128]. The true nature of the rock-mass is not explicitly reflected in the *Q* system, although it is crucial for the prediction of the support measures (e.g., popping ground, squeezing, or swelling) [20]. According to Loset [147], the *Q* system is not applicable in weak rock and weak zones, which was also confirmed by comparison of *Q*-system support with that of *NATM* (New Austrian Tunneling Method) support in swelling rock [148]. Bhawani Singh and his co-researchers [127] have presented an empirical approach (Equation (71)) for critical overburden calculation when squeezing may happen.

$$H > 350Q^{1/3} \tag{71}$$

In the calculation of H expressed above, the method of calculating *Q* is uncertain. This was also pointed out by Palmstorm [138].

In the *Q*-system support chart, the determination of support is based on *Q* value and D_e. The D_e is the function tunnel span and ESR. The ESR value should be based on the safety of the working team according to the country's safety criteria because each country has its own criteria [138]. Application of the *Q* system is more prevalent in jointed rock without overstressing [138] and displays optimal performance for a tunnel with D_e between 2.5 and 30 and *Q* values between 0.1 and 40 [149].

7.3. Limitations Related to the Two Systems

The different excavation (size, shape and purpose) and corresponding rock-supporting practices and procedures in different countries, as well as the permanent support requirements, result in approaches and amounts of support that fluctuate widely from tunnel to tunnel. The users of empirical support charts ignore on which averaged and imprecise basis the rock-support given in the support chart is based. This feature is common for quantitative rock mass classification systems used for tunnel support. It is the opinion of the experts in tunneling that rock-mass classifications as a designing tool on their own should be used for preliminary design of underground openings, that is, for planning purposes and not for final tunnel support [99,150,151].

The two systems have duplications of some parameters, such as *RQD*, joint number and their spacing, which doubles the effect of the joints on the final score. Furthermore, the influence of water on clay-bearing rocks plays a significant role in reducing rock-mass strength, however, this influence is not effectively reflected in these classification systems [7]. *RQD* is used in both systems as a mandatory parameter but their correlation with other jointing measurement parameters is very difficult [60].

The difficulty in correlation is that *RQD* is a one-dimensional measurement based only on core pieces longer than 0.1 m, which means that the application of *RQD* in rock engineering may lead to inaccuracy. The incorporation of this parameter within the two systems was a matter of historical development and its use in these classification system is no longer essential [61]. In rock-mass classification systems, parameters and ratings are applied irrespective of the project, specific characteristics and failure mechanisms [151].

Three of the basic limitations in using these systems are the risk of over-simplification by summarizing a complex rock-mass with a single digit, the scale effect and accounting for anisotropy and heterogeneity in tunnel and cavern designs [151–153]. The same rock mass quality value (*RMR* or *Q*) can be attained by a number of groupings of parameters score, even though the behavior of rock-mass could be dissimilar [151]. In rock-mass classification systems, there is no way of assessing the safety margin of a certain support under certain conditions, as the classification was assigned based on a rule and not on the engineering analysis [153]. These systems do not allow the user to quantify the degree of safety achieved by the design [56]. According to Pell and Bertuzzi [154], rock-mass classification systems include a significant degree of interpretation, as they are based on a particular structure at a particular depth.

One concern with empirical systems is that the true class of the rock mass is unidentified, which results in additional uncertainties and therefore the risk of misclassification [20]. The hazards and associated consequences due to misclassification are not taken into consideration. Thus, they will have a limited understanding of the mechanical processes associated with tunneling [153]. When excavated, a rock mass is the location of complex phenomena that rely upon mechanics, physics, thermodynamics, chemistry and so forth. Therefore, although it is comparatively easy to quantify the rock matrix properties. However, the same is not true for the rock mass at the scale of our works and a single score of this rock mass can clearly not pretend to achieve a description of the large variety properties.

In rock engineering, with time, original rules and temporary clarifications become rooted as exercise and the original rules are hardly reconsidered or questioned [155]. These systems are misused continuously due to the limitations used and assumptions made in developing them have been ignored by users. For this reason, empirical classification systems contain a hidden critical risk [156]. The assigned parameters are neither independent nor directly connected with rock behavior [157]. Therefore, the users only consider the geological characterization of the rock mass followed by ground support recommendation without reflection of the probable ground behavior. It is essential to examine the available rock-mass behavior information whether they are addressed by the empirical systems or not. A number of potential modes of failure are shown in Table 8 which reveal that every failure mode are not covered by the empirical methods and they must be considered individually [151]. As these systems are based empirically for the design and no division is made between durability, serviceability and structural resistance, this is a severe deficiency, particularly when linked to the design codes [56]. In heterogeneous and poor ground conditions, these systems may deliver false results, whilst their other inadequacies comprise a deficiency of consideration for different rock-mass failure modes and for the ground-support interaction [151].

8. Conclusions

Classification systems of rock masses have been developed significantly and serve as transmitters of their characteristics. Specifically, with respect to tunneling, *RMR* and *Q* systems describe and divide the ground into different classes and are also part of the engineering design methodologies. These *RMR* and *Q* systems are numerical classifications and are established on jointed rock-mass case records. These have a wide-range of application in rock mechanics but are established for tunnel-support and for this particular purpose they are updated empirically. The key purpose behind their worldwide recognition is that these systems have been continuously updated both in the form of characterization and support. These systems on their own can be used for the preliminary design and planning of tunnel supports. They are unreliable during tunnel construction for rock-support, as local geometry of the underground excavation and geological features may dominate the rock-mass quality defined by these systems. Along with preliminary design of tunnel support, these systems are also good alternatives for the estimation of rock masses geomechanical properties (deformation modulus, strength, Mohr-Coulomb parameters and Hoek-Brown constant), stand-up time for the tunnel and support load. Along with the wide use of these systems in rock engineering in general and tunneling in particular, there are limitations of using *RMR* and *Q* systems that will be encountered when using them in the engineering applications. The classification system accuracy and the hazard resulting from misclassification must be weighed. It is concluded that both systems are suitable for jointed rock-mass where the ground behavior is led by rock fall. However, other types of behavior, like swelling, squeezing, raveling, or popping ground, are not effectively covered by these systems. These systems should be applied with great care for such conditions and, preferably, should be supported by other design tools. Rock-mass classifications were never suggested as the decisive solution to design problems but only a way towards this end. For a true classification, these empirical design methods need further improvements, as it is difficult to define their accuracy. There are a number of pitfalls in using these classification systems in the design and construction of tunnels, which must be considered by the users. Considering the limitations of the said systems and when used along with other suitable design tools, the *RMR* and *Q* systems are valuable tools for tunnel design and also serve as a kind of checklist.

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