Performance of the empirical method with rock mass classification systems to derive optimal rock support design in poor rock mass conditions

Jorge Terron-Almenara
Norwegian University of Science and Technology NTNU, Trondheim, Norway

Charlie Chunlin Li
Norwegian University of Science and Technology NTNU, Trondheim, Norway

ABSTRACT: The empirical method has been traditionally used for the design of permanent rock support in hard rock tunnels. Engineering classification systems have typically assisted empirical design with the description and classification of rock masses, together with recommendation of permanent rock support. In this sense, rock mass classifications have generally performed well, including poor ground conditions. However, research and studies during the last decades together with recent case records have shown that there is room for improvement when the empirical classifications are utilized in poor ground conditions. Some of the mentioned limitations are addressed in this article, through the study of a database with more than one hundred case records of monitored tunnels with empirical support design and excavated in poor ground conditions. The results indicate that a convenient design approach in poor ground conditions should involve the study of the ground behavior in the more traditional empirical method.

Keywords: Empirical method, rock mass classification, poor ground conditions, rock support, ground behavior.

1 INTRODUCTION

Rock support design in hard rock tunnelling has been normally done with the assistance of empirical rock mass classifications like the Q-system. After nearly half a century of use, it has become a rather extended design method for tunnel rock support, even in poor ground conditions.

Although in most situations rock masses and permanent support design are well addressed in rock mass classification systems, there have been instances where more elaborated design methods had to be included when facing poor ground conditions like weakness zones, weak and deformable ground, and anisotropic rock masses, among others. It is in principle inevitable that the complexity of the ground increases with decreasing rock mass quality, which naturally invites more complex failure mechanisms that often need to be addressed separately. It this sense, some research has been done, like Palmstrom & Broch (2006) and Høien et al. (2019). Besides the known premises and limitations of classification systems, this paper investigates now the performance of classification systems for support design from the perspective of real case records of tunnels with support designed empirically.

2 EMPIRICAL DESIGN OF ROCK SUPPORT IN POOR ROCK MASS CONDITIONS

The empirical design of tunnel rock support has been traditionally assisted by the application of rock mass classification systems like the RMR of Beniawski (1973) or the rock mass quality Q of Barton et al. (1974). Later, the empirical Geological Strength Index (GSI) of Hoek (1994) to provide geological input into the "Generalized Hoek-Brown" failure criterion (Hoek 1994 and Hoek et al. 2002) was introduced. It provided a new methodology to describe rock masses and estimate rock mass properties needed for continuum and elasto-plastic analyses of weak, deformable rock masses.

In the case of the empirical Q-system, it has now been used for nearly half a century for rock mass classification and permanent rock support recommendation in rock tunnels. This has included design through a wide variety of geological and ground conditions from *hard* bedrock to poor, weak rocks. The gained experience since then and the *a posteriori* analysis of case records have therefore contributed to the development of the method through updates and publications. Among others, Grimstad & Barton (1993) and Grimstad et al. (2002). The Q-system similarly employs the design principles of hard rock tunnelling. That is, the utilization and conservation (or improvement) of the self-bearing capacity of the rock mass by implementing rock reinforcement and supportive measures. As such, the developers of the Q-system found six rock mass parameters judged to describe best the rock mechanical link between the fundamental causes of tunnel instability (or stability) and the tunnel stabilization provided by both the self-bearing rock and the rock support. A Q-design chart was then derived with prescribed rock mass classes, rock support classes and support designs.

In poor and deformable ground conditions represented by low Q-values, similar design principles are used. But in addition to the typical rock reinforcement provided by combinations of fiber reinforced sprayed concrete and pattern bolting, load-bearing support in the form of Ribs of Reinforced Sprayed concrete (RRS) are applied (NGI 2015). Basically, bolted and steel reinforced shotcrete arches disposed circumferentially around the tunnel contour as seen in Figure 1e.

Although description of rock masses and rock support recommendation are well addressed in most of situations by classification systems, there have been instances where consideration of the actual rock mass response (or ground behavior) should have been made as well. This is not only in line with the known premises (or limitations) of classification systems as reported by Palmstrom & Broch (2006), but also supported by the research of Høien et al. (2019) and Elmo & Stead (2021). In this context, identification of ground behavior types as presented in Figure 1 may be a beneficial tool in the design process of rock support in poor rock mass conditions. The support design could then be adjusted properly when combining the effect of tunnelling to the given rock mass properties like rock mass structure, rock strength, and in situ stresses, among others (Terron-Almenara 2022).

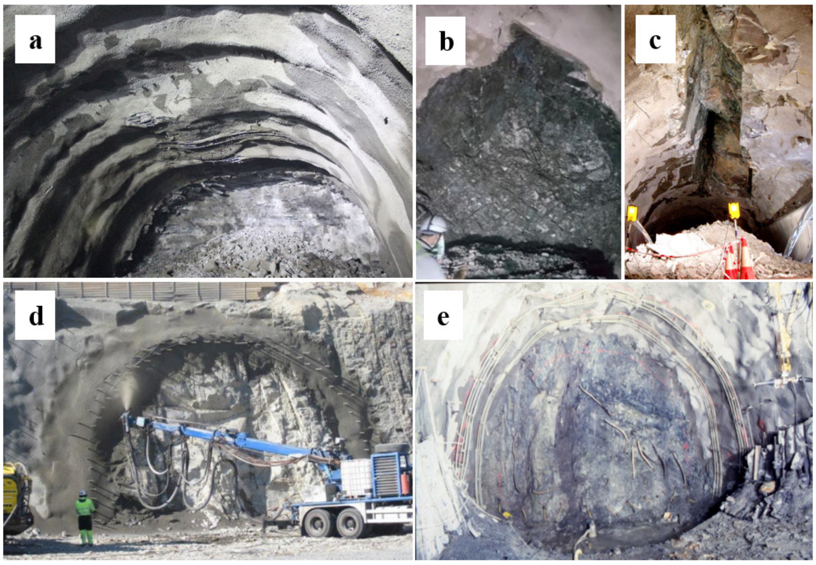

Figure 1. Ground behavior types in hard rock tunnels. a) geological structure-controlled roof delamination (Photo: Skanska), b) stress-induced and structurally controlled overbreak in fractured rock, c) weakness zone failure, d) very low in-situ stresses e) weak rock in fault zone (Photo: K.G. Holter).

3 PERFORMANCE OF ROCK MASS CLASSIFICATION SYSTEMS

3.1 Rock mass mapping

Rock mass classification is based on the visual inspection, mapping and rating of individual rock mass parameters defined in engineering rock mass classifications. The performance of the mapping and classification is therefore subject to not only the limitations of empirical classifications, but also to the inherent cognitive biases involved along the process of rock mass mapping.

Cognitive biases in the application of engineering rock mass classifications have been well addressed by Elmo & Stead (2021). The authors discuss, among others, the role of uncertainty and variability and its effect on the characterization and quantification of rock parameters in classification systems. One way to reduce uncertainty is by increasing knowledge. Knowledge could be increased by a progressive collection of more data in a project, then reducing, but not eliminating, uncertainty. However, other types of uncertainty like parameter uncertainty (i.e., scale effect and spatial variability), model uncertainty (i.e., limitations of the method) and human uncertainty (i.e., measurement errors and different engineering judgement) would remain unchanged regardless the data availability. A similar situation or bias may be found when assuming that the accumulated experience and the engineering judgement may feed knowledge. This is because engineering judgement can be predisposed by the very own experience that created it (Elmo & Stead 2021).

The latter is illustrated in Figure 2. The graph represents more than six hundred tunnel *face mappings* with registration of RQD and Q-values, recorded in a tunnel project with different geologists in charge of rock mass mapping. It is interesting that a rock mass with quality Q 1-2 (Poor rock) may have so distant RQD from 30% to 70%. This might not only tell about the wide variability of RQD in the mapped poor rock masses, but also about the limitations of classification systems to describe two rather different rock masses ("X" and "Y" in Figure 2) with the same mapped Q-value. A similar example is observed for the Q-interval 0.8-60 (Very Poor to Very Good rock) with a mapped RQD 60%. The ground conditions remain not well described as two very different ground conditions (represented as "V" and "W") can be described by the same RQD (60%). For the same example, it should be also questioned whether the *mapped* conditions of rock mass "V" (RQD 60%, Q 60) are *geologically* compatible. That is, RQD 60% represents a relatively high joint count (13-19 joints/m^3) as suggested in NGI (2015), which would hardly define a massive rock mass with Q 60.

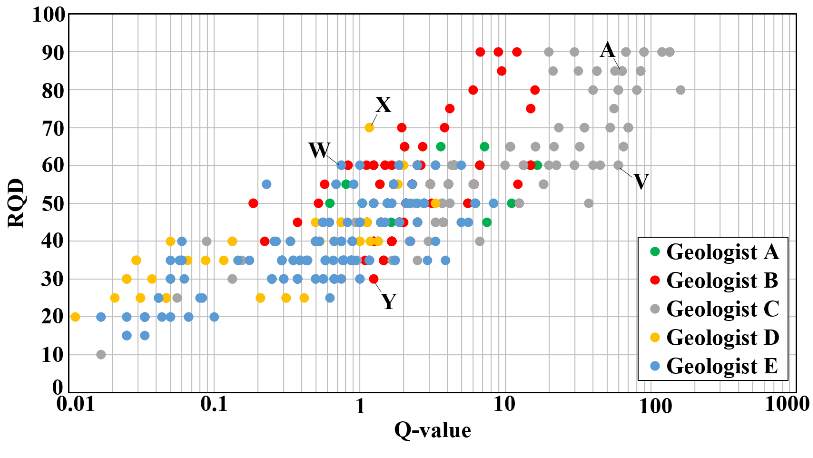

Figure 2. Mapped RQD and Q by different geologists in poor ground conditions. Tunnel span 6-12.5 m.

Another common bias when using rock mass classifications is the "anchoring" (Elmo & Stead 2021). In other words, the propensity to associate rock mass quality indices (or ratings) to already built impressions of what the rock mass should look like. This is partly observed in Figure 2. For example, the geologist C has "learned" that for a particular set of geological conditions, i.e., defined by the rock masses between "A" and "V", a wide range of RQD (likely combined with different values of other Q-parameters) can suit the ground conditions, defined with Q 60.

Other well-known empirical systems like the GSI (Hoek 1994, Marinos & Hoek 2000) are not exempt of limitations. Mapping of GSI is based on the qualitative assessment of only two rock mass parameters: rock mass structure and the quality of the joint surfaces. In other words, the whole complexity and nature of a rock mass is reduced to only two parameters measured qualitatively. For example, a rock mass with a mapped value of GSI 40 may behave quite differently if the "quality" of the joints varies from "Fair" to "Poor" but keeping the same "geological" rating (40). If this input is used in the Hoek-Brown failure criterion to estimate rock mass strength (Hoek et al. 2002) and the output used further for design calculations, it would be obvious that the destabilizing effect of joints with poor quality is missed in the process.

The graph in Figure 3 represents the relationship between mapped and back-calculated GSI-values for 99 case records belonging to deformation monitored tunnels, mostly from Norway. They represent hard rock tunnelling through poor ground conditions as these shown in Figure 1. The back-calculated GSI-values were derived from numerical analyses utilizing actual rock mass properties from site and laboratory characterization and calibration against measured ground behavior. It is observed that for the poorer rock masses (Q<0.1), GSI tends to be conservatively mapped. Possibly, due to the inherent conservativism involved in describing qualitatively the "interlocking" and the "joint surfaces" of complex and strongly jointed rocks. In turn, the mapped GSI becomes aligned or even optimistic compared to the back-calculated GSI in the more frictional rock masses (Q>0.1). Likely, due to the overestimation of intact rock- and/or joint -properties in the more competent rocks.

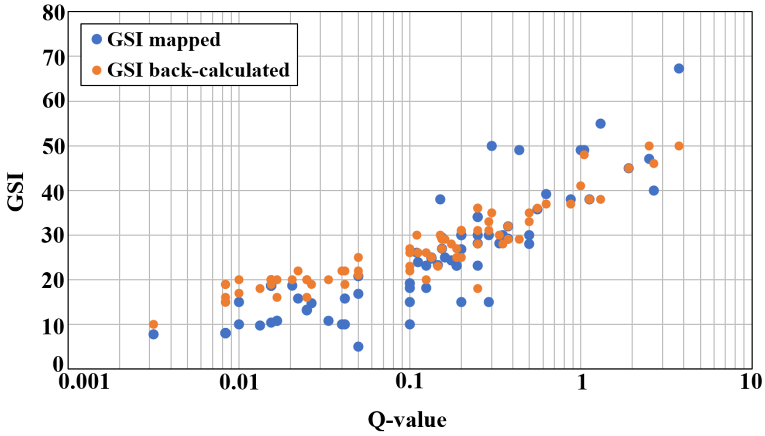

Figure 3. Comparison of mapped and back-calculated GSI in poor ground conditions.

3.2 Prediction of ground behavior

Rock mass classification systems can also be used to make predictions of ground behavior. Two different methodologies have been studied and the results plotted in Figure 4. The study contains over one hundred case records and is based on the comparison of calculated tunnel strain ($\mathcal{E}$, ratio of tunnel closure to tunnel span) obtained from the application of empirical correlations (i.e., Barton et al. 1994) and the back-calculated ground behavior using deformation monitoring and numerical analysis. The study has similarly served to evaluate the performance of the two methods in anisotropic rock masses (nearly 30 case records in foliated, layered hard rock). A notable feature in Figure 4 is the de-branched distribution of estimated tunnel strain, where $\mathcal{E}$>10% is predicted by the Q-system and up to $\mathcal{E}$=1% in the monitored and back-calculated tunnel sections when rock mass quality Q≈0.01. It seems that the empirical prediction of tunnel strain for Q>0.1 had been projected over to the trend for Q<0.1. This gives significantly high values of deformation which are more in line with collapsed tunnels than with the actual deformational response of the ground. The latter is represented by two failure cases, Skarvberg (Figure 1a) and Hanekleiv (Figure 1c) tunnels. The deformation in these two case records was not measured but estimated empirically. About 30 cm of vertical roof deflection for Skarvberg, and about 1 m (collapse) in Hanekleiv. Both failures seem to match with the empirical prediction. On the other hand, it is also noticeable in Figure 4 that the empirical prediction does not make distinction between isotropic and anisotropic rock masses, whilst the different responses provided by isotropic and anisotropic rock masses are catched with numerical analyses assisted by deformation monitoring.

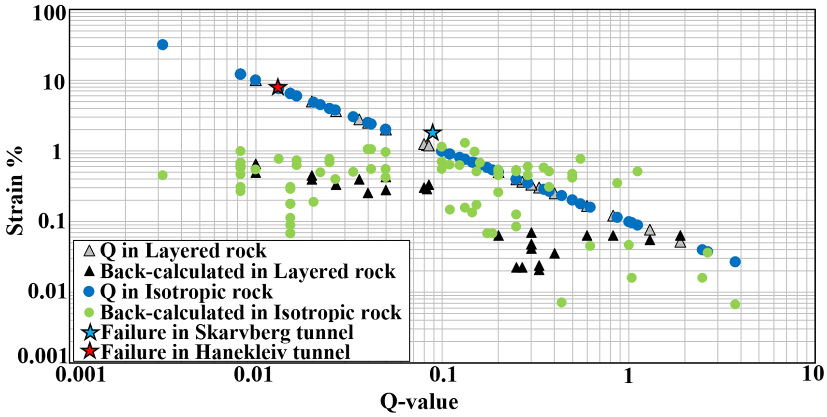

Figure 4. Performance of the Q-system and back-calculations to predict tunnel strain in supported tunnels.

3.3 Current design experience

Figure 5 is a support pressure to Q-value diagram modified from the original Barton et al. (1974) chart. The shaded envelope represents the best estimation of support pressure needed to withstand the loads endorsed by the ground (Barton et al. 1974). The calculated support pressure representing the "current experience" from Norwegian tunnels (about hundred case records) have been plotted over the original Barton et al. (1974) chart and split into ground behavior types as suggested in Figure 1. The two failure cases of Skarvberg and Hanekleiv tunnels are included as well.

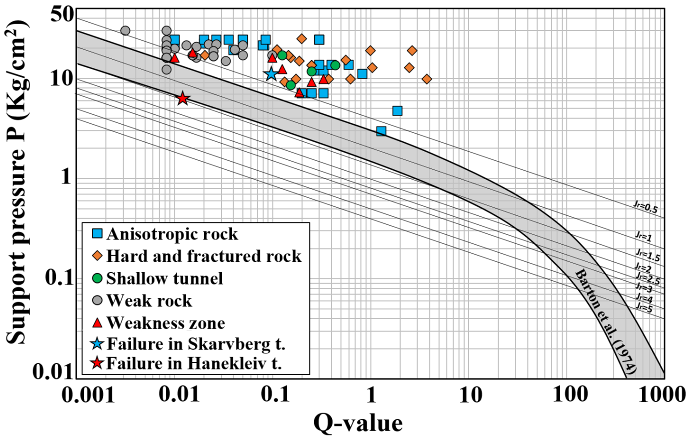

Figure 5. Support pressure utilized in recent case records in poor ground conditions in Norwegian tunnels (of span 9-12.5 m). Modified from Barton et al. (1974) and published in Terron-Almenara (2022).

A general observation in Figure 5 is that most of the points lie above the shaded envelope of support pressure, which indicates a general design conservativism during the last years in Norway. The conservativism may have been introduced by the local design guidelines used in Norway, by the own limitations of the (empirical) method in poor ground conditions, or a combination as discussed in Terron-Almenara (2022). For example, the flat distribution of points representing both ground categories "anisotropic rock masses" and "weak rocks" at the interval Q 0.1-0.01 with a nearly constant support pressure of ca. 20-30 kg/cm^2 illustrates how insensitive the empirical approach might be to variations of in-situ rock mass quality and rock properties. By looking at the ground category "weak rock", it is also interesting that for Q 0.008 the support pressure varies as much as 11 to 30 kg/cm^2. Several conditions may have caused such wide variation in support pressure (design) like rock strength, in-situ stresses, and/or jointing conditions. However, the same rock mass quality

Q is still mapped, 0.008. This results in the application of the same prescribed rock support class (design) for two different ground conditions that empirically demand support pressures of up to 3 times difference. In looking to the failure in Hanekleiv tunnel, it sits at the lower boundary of the shaded envelope, which may in part explain the collapse. However, the direct reason for the failure occurred after ca. 10 years of tunnel operation was the weakening and swelling activation over time of the clayey infillings of a zone crossing the tunnel (Nilsen et al. 2007). This questions about how long term changes in the ground are described in empirical classifications. A similar challenge, as anticipated in Figure 4, is the account of the rock mass structure in empirical classifications, for example the failure in Skarvberg tunnel. Although it sits well above the empirically recommended ground support pressure (Figure 5), failure took place as shown in Figure 1a. Among other reasons, caused by an unfavorable combination of low confining stress, subhorizontally laminated rock mass structure, and likely insufficient rock reinforcement to enhance the needed arching in the rock mass.

4 CONCLUSION

Engineering rock mass classifications like the Q-system and GSI are well-proven and valuable design tools in rock engineering. However, some limitations have being reported, especially in poor ground conditions. A database with more than hundred case records of monitored tunnels has been studied to evaluate the *a posteriori* performance and limitations of classification systems, from mapping to actual design. The results have indicated that classification systems perform well for *classification* and *recommendation* of rock support, which obviously requires further *engineering* to arrive to a final, optimal design. For example, with the identification and study of ground behavior.

REFERENCES

Barton, N., Lien, R. & Lunde J. 1974. Engineering classification of rock masses for the design of tunnel support. *Rock Mechanics* 6, pp. 189-236.

Barton, N., By, T.L., Chryssanthakis, P., Tunbridge, L., Krisiansen, J., Løset, F., Bhasin, R., Westerdhal, H. & Vik, G. 1994. Predicted and measured performance of the 62 m span Norwegian Olympic ice hockey cavern at Gjøvik. *Int. Journal Rock Mech. Min. Sci. Geomech. Abstr.* 31 (6), pp. 617-641

Beniawski, Z.T. 1973. Engineering classification of jointed rock masses. *Transaction of the South African Institution of Civil Engineers* 15, pp. 335–344.

Elmo D. & Stead D. 2021. The role of behavioural factors and cognitive biases in rock engineering. *Rock Mechanics and Rock Engineering* 54, pp. 2109-2128.

Grimstad, E. & Barton, N. 1993. Updating the Q-system for NMT. In: *Proc. Int. Symp. Sprayed Concrete for Underground support*, Fargenes. Norwegian Concrete Association, Oslo, pp. 46-66.

Grimstad, E., Kankes, K., Bhasin, R., Magnussen, A.W. & Kaynia, A. 2002. Rock mass quality Q used in designing reinforced ribs of sprayed concrete and energy absorption. In: *Proc. 4th Int. Symp. on Sprayed Concrete*, Davos. pp. 18.

Hoek, E. 1994. Strength of rock and rock masses. *ISRM News Journal* 2 (2), pp. 4-16.

Hoek, E., Carranza-Torres, C. & Corkum, B. 2002. Hoek-Brown failure criterion- 2002 edition. In: *Proceedings NARMS-TAC Conference*, Toronto, 1, pp. 267-273.

Høien, A.H., Nilsen, B. & Olsson, R. 2019. Main aspects of deformation and rock support in Norwegian road tunnels. *Tunnelling and Underground Space Technology* 86, pp. 262-278.

Marinos, P. & Hoek, E. 2000. GSI: A geologically friendly tool for rock mass strength estimation, *GEOENG 2000*, Melbourne, Australia.

NGI 2015. Using the Q-system. *Handbook*, p.56.

Nilsen, B., Bollingmo, P. & Nordgulen, Ø. 2007. Raset i Hanekleivtunnelen 25 desember 2006. Rapport fra Undersøkelsesgruppen. *Statens vegvesen*. (In Norwegian).

Palmstrom A. & Broch E. 2006. Use and misuse of rock mass classifications systems with particular reference to the Q-system. *Tunnelling and Underground Space Technology* 21, pp. 575-593.

Terron-Almenara, J. 2022. Empirical design of rock support in Norwegian tunnels excavated in poor rock mass conditions. Challenges and opportunities. In: *Proc. Norwegian Annual Tunnel Conf.*, Oslo, pp. 34.1-34.15.