Prediction of squeezing potential of rocks in tunnelling through a combination of an analytical method and rock mass classifications

Ömer Aydan*, Süleyman Dalgiç**, T. Kawamoto***

Summary

This paper is concerned with the prediction of squeezing potential of rocks in tunnelling and of the deformation of tunnels in such rocks. The authors present a procedure how to combine an analytical method proposed earlier by the authors (ÖA and TK) for predicting the deformation of tunnels in squeezing rocks with rock mass classifications. Then the proposed procedure is applied to Bolu tunnels (Ibrkey), which are currently under construction, and predictions are compared with actual measurements and observations.

1. Introduction

The deformational behaviour of tunnels, which underwent large deformations, so-called squeezing, have been recently receiving great attention in the field of rock mechanics and tunnelling. The first author proposed an analytical method to predict squeezing potential of rocks in tunnelling. This method is based on the utilisation of classic wave velocity of rock mass to estimate its uniaxial strength and related mechanical properties. Although the elastic wave velocity method is commonly used in Japan, it is very rare to see the use of this method in other countries. Instead, rock classifications are usually used in rock mass characterization. In this paper, the authors briefly present a procedure for combining the analytical method proposed by Aydan *et al.* [1993] with rock mass classifications. Then an application of the proposed method to predict the squeezing potentials of rocks along the alignement of Bolu Tunnels in Turkey, which are currently under construction, is given, and predictions are compared with observations.

2. Assessing mechanical properties of squeezing rocks

2.1. *Rock Mass Uniaxial Strength*

Although there are several rock classifications used in many countries, it seems that RMR and Q-systems are the most widely known rock classifications [Bieniawski 1974; Barton *et al.* 1974]. Hoek and Brown [1980] tried to establish a relation between their empirical yield criterion and RMR. For uniaxial case, rock mass strength is related to intact rock strength by

$$\frac{\sigma_{cm}}{\sigma_{ci}} = \sqrt{s} \tag{1}$$

Using the experimental results on granulated Panguna Andesite, they suggested the following formula:

$$s = e^{(RMR - 100)/B} \tag{2}$$

where B is a constant. Hoek and Brown used the following relation between RMR and Q-value, proposed by Bieniawski [1978] to compute relation between s and Q-value.

$$RMR = 9\log Q + 44 \tag{3}$$

Recently Barton [1995] suggested the following relation between *RMR* and *Q*-Value

$$RMR = 15 \log Q + 50 \tag{4}$$

As discussed by Aydan *et al.* [1997], the estimations from these methods are generally conservative when one uses the values for constant B suggested by Hoek and Brown. For example, when the values of RMR are less than 40 and Q-values are less than 1, rock mass strength becomes very small. Since the value of RMR and Q-system for squeezing rocks are generally below these threshold values, the strength of squeezing rocks will be usually underestimated. Probably for these reasons, Singh [1993] proposed the following direct relation between Q-value and rock mass strength

$$\sigma_{cm} = 0.7\gamma Q^{1/3} \tag{5}$$

* Tokai University, Shimizu, Japan.
** Istanbul University, Istanbul, Turkey.
*** Alchi Institute of Technology, Toyota, Japan.

BARTON [1995] suggested the unit weight of rocks 26 kN/m^3 for engineering applications of the relation above. Thus the above equation is re-written in the following form

$$\sigma_{cm} = 18.2\, Q^{1/3} \tag{6}$$

The unit of uniaxial strength is MPa. AYDAN *et al.* [1997] also recently proposed the following relation between RMR and rock mass strength using in-situ test data on rock mass strength in Japan.

$$\sigma_{cm} = 0.0016\, RMR^{2.5} \tag{7}$$

The unit of uniaxial strength is MPa. It should be also noted that this relation is derived for Basic RMR values. The relations given above are compared with the data of in-situ tests gathered from several construction sites in Japan in Fig. 1.

When rock mass is weak and jointing is negligible, direct relations giveñ above may overestimate the strength of rock masses. Under such circumstances, it is recommended to use reduction coefficients related to rock classification values. Besides HOEK-BROWN's proposal, KALAMARAS and BIENIAWSKI [1995] also suggested the following formula between the uniaxial strength of intact rock and that of rock mass

$$\sigma_{cm} = 0.5\, \frac{(RMR - 15)}{85}\, \sigma_{ci} \tag{8}$$

Reanalysing the data shown in figure 1, the authors will propose the following relation for strength reduction coefficient:

$$\sigma_{cm} = \left(\frac{RMR}{100}\right)^{1/3} \sigma_{ci} \tag{9}$$

The relation given above is compared with that of HOEK-BROWN as well as with in-situ data in Fig. 2.

2.2. *Other related mechanical properties*

From an extensive survey of data, AYDAN *et al.* [1993] developed a data-base system, called *SQR-PROP* and plotted various physical and mechanical properties of rocks as a function of the uniaxial strength of rocks and established relationships between these properties and the uniaxial strength of squeezing rocks. The relationships between physical and mechanical properties and uniaxial strength of squeezing rocks are summarised as follow:

i-) Physical Properties: unit weight $\gamma(kN/m^3)$ porosity $n(\%)$, elastic wave velocity $V_p(km/s)$

$$\gamma = 10(1 + 0.8\sigma_c^{0.15}), \quad n = 60e^{-0.10\sigma_c}, \quad V_p = 1.4 + 0{,}2\sigma_c^{0{,}7} \tag{10}$$

ii-) Elastic constants: elastic modulus *E (MPa)*, Poisson's ratio ν

$$E = 80\sigma_c^{1{,}4}, \quad \gamma = 0{,}25(1+e^{-0.2\sigma_c}) \tag{11}$$

iii-) Plastic constants: peak friction angle ϕ_p(°), residual uniaxial strength σ_p^r *(MPa)*, residual friction angle ϕ_r(°), peak plastic Poisson's ratio f_p, softening plastic Poisson's ratio f_s, residual plastic Poisson's ratio f_r

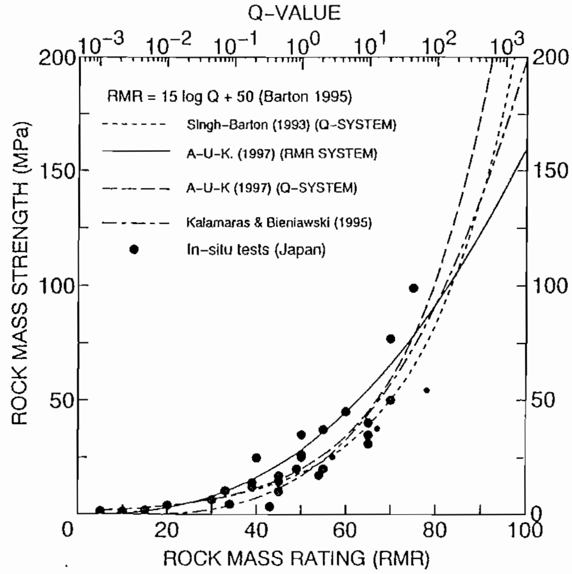

Fig. 1 – Relation between rock classifications and rock mass strength.
Fig. 1 – Relazione tra le classificazioni dell'ammasso roccioso e la sua resistenza a compressione monoassiale.

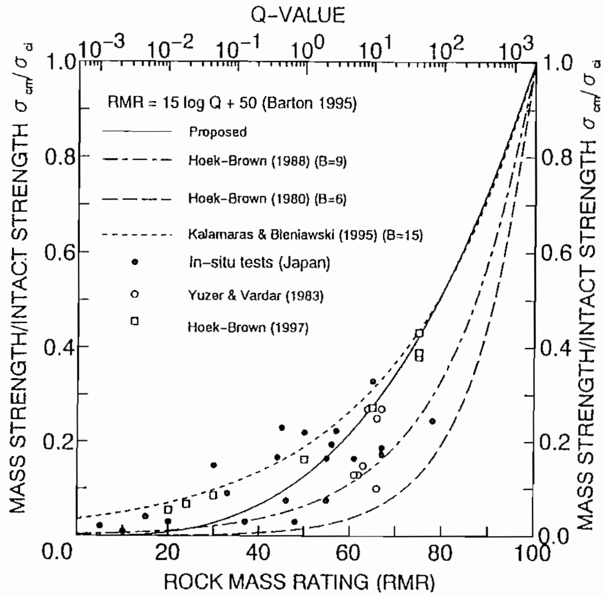

Fig. 2 – Relation between rock classifications and strength reduction coefficient.
Fig. 2 – Relazione tra le classificazioni dell'ammasso roccioso ed il coefficiente di riduzione della resistenza a compressione monoassiale della roccia intatta.

$$\phi_p = 20\sigma_c^{0.25}, \quad \frac{\sigma_c^r}{\sigma_c} = e^{-0.3\sigma_c},$$

$$\frac{\phi_r}{\phi_p} = 1.3 - 0.3e^{-0.3\sigma_c} \quad f_p = 0.5 + 0.25\sigma_c^{0.8}, \quad (12)$$

$$f_s = 1.0 + 0.25\sigma_c^{0.7}, \quad f_s = 0.5 + 0.25\sigma_c^{0.4},$$

It should be noted the unit of uniaxial strength is *MPa* in the above relations.

3. An analytical method for predicting squeezing potential of rocks

An analytical solution as been already developed by AYDAN *et al.* [1993] for predicting strain and stress fields and the level of squeezing of circular tunnels in squeezing rocks under a hydrostatic state of stress. Therefore, the final expressions for tangential strains for various states are only given herein:

1-) Elastic state

Tangential strain and stress at tunnel wall can be obtained as:

$$\varepsilon_\theta^a = \frac{1+\nu}{E}(p_0 - p_i), \quad \sigma_\theta^a = 2p_0 - p_i \quad (13)$$

If the tunnel is strained to its elastic limit, then $\sigma_\theta^a = \sigma_c$, with $p_i = 0$. Thus, we have the elastic strain limit as:

$$\varepsilon_\theta^a = \frac{1+\nu}{E}\frac{\sigma_c}{2} \quad (14)$$

Using the above relation in Eq. (13), one obtains the normalised tunnel wall strain as:

$$\xi = \frac{\varepsilon_\theta^a}{\varepsilon_\theta^e} = 2\frac{1+\beta}{\alpha} \leq 1 \beta = \frac{p_i}{p_o}, \ \alpha \ \frac{\sigma_c}{p_o} \quad (15)$$

2-) Perfectly plastic state

Tangential strain at tunnel wall can be obtained as

$$\varepsilon_\theta^a = \frac{1+\nu}{E}(p_0 - \sigma_{rp})\frac{R_{pp}^{f+1}}{a} \quad (16)$$

elastic limit is given as:

$$\varepsilon_\theta^a = \frac{1+\nu}{E}(p_0 - \sigma_{rp}) \quad (17)$$

Using the above relation in Eq. (16), one obtains the normalised tunnel wall strain as:

$$\xi = \frac{\varepsilon_\theta^a}{\varepsilon_\theta^e} = \left[\frac{2}{q+1}\left\{\frac{(q-1)+\alpha}{(q+1)\beta+\alpha}\right\}\right]^{\frac{f+1}{q-1}} \quad (18)$$

3-) Residual plastic state

Tangential strain at tunnel wall can be obtained as

$$\varepsilon_\theta^a = \frac{1+\nu}{E}(p_0 - \sigma_{rp})\eta_{sf}\frac{R_{pp}^{f+1}}{a} \quad (19)$$

Using Eqs. (17) & (19), one obtains the normalised tunnel wall strain as:

$$\xi = \eta_{sf}\left[\frac{\frac{2}{q+1}\left\{\frac{(q-1)+\alpha}{(q+1)\beta+\alpha}\right\}\eta_{sf}^{-\frac{q-1}{f+1}} - \frac{\alpha}{q-1} + \frac{\alpha^*}{q^*-1}}{\beta + \frac{\alpha^*}{q^*-1}}\right]^n \quad (20)$$

where

$$\frac{\varepsilon_\theta^a}{\varepsilon_\theta^e} = \alpha^* = \frac{\sigma_c^*}{p_0}, \quad n = \frac{f^*+1}{q^*-1}$$

4. Applications to Bolu tunnels

Turkish Roadway Authority (Türkiye Karayollari Genel Müdürlügü) is now constructing the Anatolian Express Motorway which extends from the European borders of Türkiye to the Asian borders. This express motorway passes through the famous fault zones such the North Anatolian Fault Zone, East Anatolian Fault Zone, which are tectonically very active.

The Bolu tunnels have a cross-section of about 170-200 m^2, which is approximately the half of typical cross sections of underground power houses, and are 3250 m long; still under construction, the excepted date of completion is year 2000.

The Bolu tunnels pass through the North Anatolian Fault Zone (NAFZ) and the rock masses consist of various sedimentary rocks as well as igneous rocks (Fig. 3). Faulting along the tunnels is of strike-slip or thrust types. The excavation through igneous rocks was not problematic. Once the tunnels have entered the sedimentary rocks which are heavily faulted and folded, very excessive deformation of the tunnels was encountered. In some cross-sections deformation was more than 1500 mm.

The second author (SD) has been involved with these tunnels since preliminary site investigations and has experienced the difficulty of assessing the characteristics of faulted sedimentary rock masses from outcrop surveying in heavily forested areas and vertical borings [DALGIÇ, 1994; DALGIÇ and GÖZÜBOL, 1996].

In this article, the authors attempt to predict the squeezing potential of rocks and the measured deformation responses of the Asarsuyu section of the left

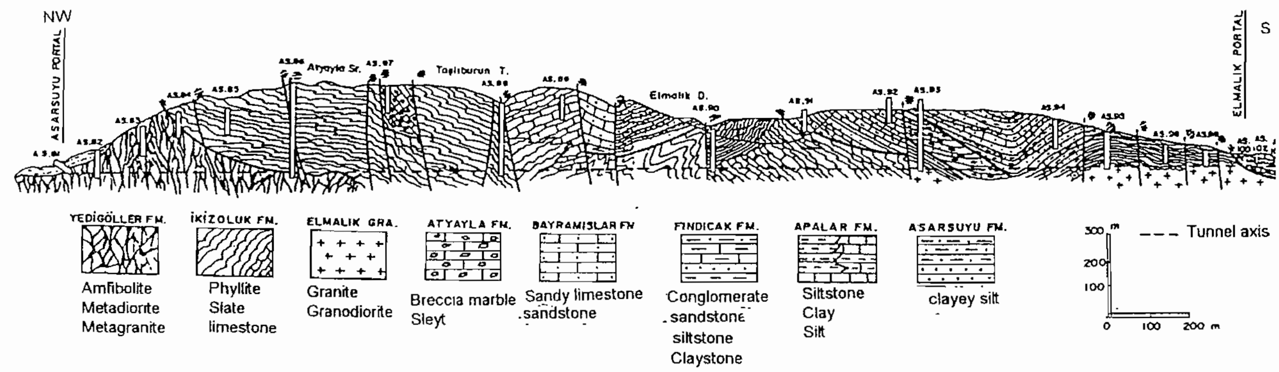

Fig. 3 – Longitudinal geology of Bolu Tunnels.
Fig. 3 – Profilo geologico delle gallerie Bolu.

tube of Bolu Tunnels, the excavation of which is almost completed. Rock masses have been assessed by the second author (SD) using both Bieniawski's RMR and NGI's Q-System and their strength has been estimated from the formula of AYDAN *et al.* [1997] for RMR system and SINGH [1993] for Q-system. Fig. 4 shows the variation of overburden and of rock mass strength estimated from each formula. Surprisingly, the strength estimated from both formula are very similar to each other. After estimating the strength of rock masses, other mechanical properties needed in computations are determined from the empirical formulas given in Subsection 2.2 and the deformation behaviour of the tunnels is predicted by using the method of AYDAN *et al.* [1993]. Fig. 5 compares the prediction for Asarsuyu section of the left tube of Bolu Tunnels for unsupported case. As expected, the deformations predicted for unsupported case should be larger than those measured since the actual tunnel

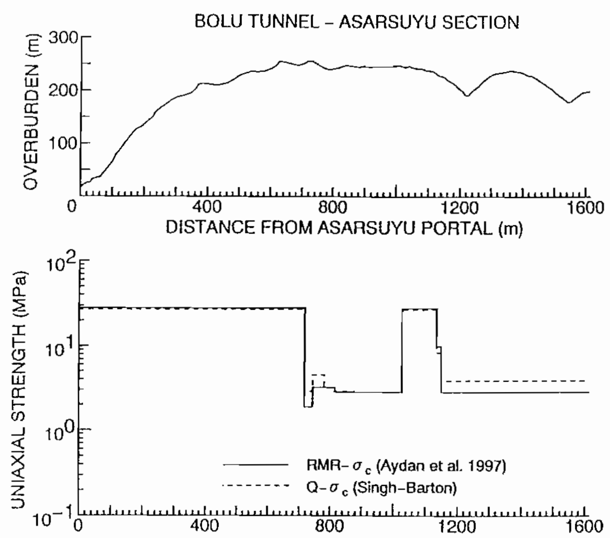

Fig. 4 – Variation of overburden and estimated uniaxial strength of rock mass along tunnel alignement.
Fig. 4 – Variazione della copertura e stima della resistenza a compressione monoassiale dell'ammasso roccioso lungo lo sviluppo della galleria.

is supported. The predictions show that when squeezing takes place, it will range from fair squeezing (FS) to very heavy squeezing (VHS). These predictions are consistent with both observations and deformation measurements. Although there exist some differences between predictions and observations, these may be solved through more detailed rock classifications along the tunnel alignement and the consideration of actual support system.

5. Conclusions

In the present study, the authors combined analytical method proposed by AYDAN *et al.* [1993] and rock mass classifications. The rock masses were as-

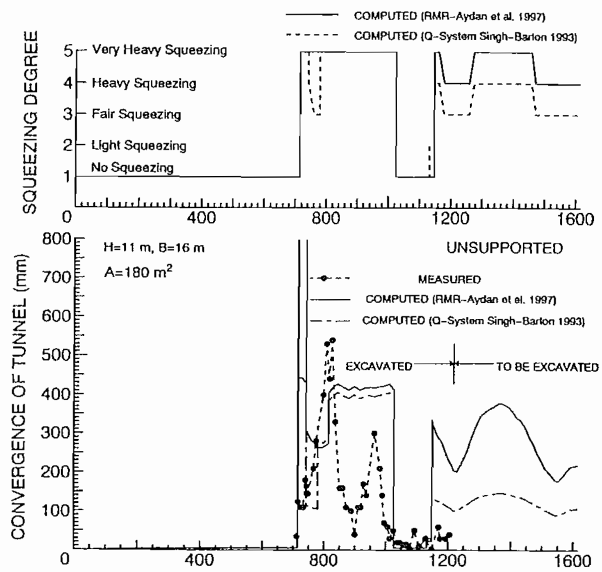

Fig. 5 - Variation of squeezing degree and comparison of predicted deformation of tunnel with measurements along tunnel alignement.
Fig. 5 – Variazione del livello di comportamento spingente e previsione delle deformazioni della galleria poste a confronto con le misure lungo l'asse longitudinale.

sessed using both Bieniawski's RMR and NGI's Q-System and their strength was estimated from the formulas given by AYDAN *et al.* [1997] for RMR system and SINGH [1993] for Q-system. Once the strength of rock masses is determined, other mechanical properties for computations are determined from the empirical formula of AYDAN *et al.* [1993], and the behaviour of the tunnels is predicted by the method of AYDAN *et al.* [1993]. The prediction for Asarsuyu section of the Bolu Tunnels are compared with observations. The applicability and validity of the proposed procedure have been checked by comparing the predictions with actual observations. It is found that the predictions by the proposed procedure well agree with observations and it can be a usefull tool for engineers to predict not only the squeezing potential of rocks but also the magnitude of deformation of tunnels in squeezing rocks.

6. Acknowledgements

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Previsioni del potenziale comportamento spingente nello scavo di gallerie mediante metodi analitici e di classificazione dell'ammasso roccioso

Sommario

La nota descrive la previsione del potenziale comportamento spingente nello scavo di gallerie e della conseguente deformazione indotta. Gli autori presentano un procedimento che consiste nell'uso di un metodo analitico, proposto in precedenza dagli autori (ÖA e TK), per la previsione della deformazione di gallerie in condizioni spingenti, congiuntamente ai metodi di classificazione degli ammassi rocciosi. Il procedimento proposto viene applicato alle gallerie Bolu (Ibrkey), attualmente in costruzione, e le previsioni formulate sono poste a confronto con i risultati delle misure e delle osservazioni.