Engineering Geology 107 (2009) 154–166

Contents lists available at ScienceDirect

Engineering Geology

journal homepage: www.elsevier.com/locate/enggeo

Technical Note

An empirical method for design of grouted bolts in rock tunnels based on the Geological Strength Index (GSI)

Reza R. Osgoui [a,*], Erdal Ünal [b]

[a] GEODATA SpA., Corso Duca degli Abruzzi, 48/E, 10129, Turin, Italy
[b] Mining Engineering Department, Middle East Technical University, 06531/Ankara, Turkey

ARTICLE INFO

Article history:
Received 14 April 2008
Received in revised form 4 May 2009
Accepted 16 May 2009
Available online 27 May 2009

Keywords:
Empirical design method
Grouted bolt
Bolt density
Geological Strength Index (GSI)
Malatya railroad tunnel

ABSTRACT

The procedure presented in this paper has been developed for the design of grouted rock bolts in rock tunnels during preliminary design stage. The proposed approach provides a step-by-step procedure to set up a series of practical guidelines for optimum pattern of rock bolting in a variety of rock mass qualities. For this purpose, a new formula for the estimation of the rock load (support pressure) is recommended. Due to its wide-spread acceptance in the field of rock engineering, the Geological Strength Index (GSI) is adopted in support pressure equation. For poor and very poor rock mass where the GSI < 27, the use of Modified-GSI is, instead, recommended. The supporting action is assumed to be provided by rock bolts carrying a total load defined by the rock load height. The mechanism of bolting is assumed to rely on roof arch forming and suspension principle. Integrated with support pressure function, the bolt density parameter is modified in order to provide an optimized bolt pattern for any shape of tunnel. The modified bolt density can also be used in analysis of a reinforced tunnel in terms of Ground Reaction Curve (GRC) in such a way as to evaluate the reinforced rock mass and the tunnel convergence. By doing so, the effectiveness of the bolting pattern is well evaluated. The proposed approach based on GSI is believed to overcome constrains and limitations of existing empirical bolt design methods based on RMR or Q-system, which are doubtful in poor rock mass usage. The applicability of the proposed method is illustrated by the stability analysis and bolt design of a rail-road tunnel in Turkey.

1. Introduction

Currently, rock reinforcement technique (rock-bolting) is used in almost all types of underground structures due to its performance, cost-effectiveness, and safety. The structures reinforced by bolts are, in general, very reliable and long lasting. The main objective of rock bolts should be to assist the rock mass in supporting itself by building a ground arch and by increasing the inherent strength of the rock mass. One of such type of bolts is the grouted bolts that develop load as the rock mass deforms. Small displacements are normally sufficient to mobilize axial bolt tension by shear stress transmission from the rock mass to the bolt surface. Grouted bolts have been successfully applied in a wide range of rock mass qualities especially in poor rock mass and found to be often more economical and more effective than mechanical rock bolts. Owing to their grouting effect on improvement of rock mass, grouted rock-bolts have been widely used in tunnelling under difficult ground condition. They are also widely used in mining for the stabilization of roadways, intersections, and permanent tunnels in preliminary design stage. Simplicity of installation, versatility and lower cost of rebars are the further benefits of grouted bolts in comparison to their alternative counterparts.

Broadly speaking, the empirical design methods based on rock mass classification systems (Bieniawski, 1973; Barton et al., 1974; Ünal, 1983; Bieniawski, 1989; Ünal, 1992; Grimstad and Barton, 1993; Palmström, 1996, 2000; Mark, 2000), the methods dependent on laboratory and field tests (Bawden et al., 1992; Hyett et al., 1996; Kilic et al., 2003; Karanam and Pasyapu, 2005), the performance assessment methods (Freeman, 1978; Ward et al., 1983; Kaiser et al., 1992; Signer, 2000; Mark et al., 2000), the analytical methods based on rock-support interaction theory and convergence-confinement approach (Hoek and Brown, 1980; Aydan, 1989; Stille et al., 1989; Oreste and Peila, 1996; Labiouse, 1996; Li and Stillborg, 1999; Carranza-Torres and Fairhurst, 2000; Oreste, 2003; Cai et al., 2004a,b; Wong et al., 2006; Guan et al., 2007) or based on equivalent material concept (Grasso et al., 1989; Indraratna and Kaiser, 1990; Osgoui, 2007; Osgoui and Oreste, 2007), and the numerical techniques (Brady and Loring, 1988; Duan, 1991; Swoboda and Marence, 1991; Chen et al., 2004)are the methods utilized in designing an effective rock-bolt system.

Generally, analytical and numerical methods are not directly used for dimensioning bolts in preliminary stage of design. However, they are used to evaluate the effectiveness of bolt system in order to modify the bolt pattern if necessary. The laboratory and field tests, on the

* Corresponding author. Tel.: +39 011 581 0628.
E-mail address: ros@geodata.it (R.R. Osgoui).

0013-7952/$ – see front matter © 2009 Elsevier B.V. All rights reserved.
doi:10.1016/j.enggeo.2009.05.003

other hand, are adopted to verify the performance of the bolting system.

Empirical methods, based on rock mass classifications, such as those mentioned earlier are the only ways to dimension the bolt system. Although a great number of such methods have been developed so far, they suffer from many limitations. For instance, their bolting patterns are qualitative, rather than quantitative and they are independent of calculations. They cannot correlate the necessary bolt length and thickness of failure zone around the tunnel. They are less flexible in terms of change in rock mass properties and field stress. Not all empirical methods focus on the grouted bolts in wide range of rock mass qualities. Furthermore, in weak rock mass, the available empirical methods do not provide a sufficiently sensitive guide for bolt design (Indraratna and Kaiser, 1990). It is not only due to the uncertainties in frictional behaviour of such bolts but also their effect on rock mass improvement has not intuitively understood.

As the bolting pattern suggested by RMR (Rock Mass Rating) and Q-system depends only on the rock mass quality, some significant critics have recently arisen. Palmström and Broch (2006) and Pells and Bertuzzi (2008) have agreed that the "well-known Q-support chart gives only indication of the support to be applied, and it should be tempered by sound and practical engineering judgement". Furthermore, the results of many experiences by Pells and Bertuzzi (2008) put forward this statement: "the classification systems should not solely be used as the primary tool for the design of primary support".

The proposed approach is intended to alleviate such limitations and constrains. Integrated with existing elasto-plastic solutions, the proposed approach makes it possible to set up several applicable bolting patterns for any rock mass condition and tunnel size. With modification of the bolt density parameter that exhibits the frictional behaviour of bolt and its link with the proposed support pressure function, a new approach in depicting the bolting pattern for any shape of tunnel is achieved. The key base of the proposed approach falls in the estimation of the support pressure by using the Geological Strength Index (GSI) or the Modified-GSI (Osgoui, 2007) because of its successful acceptance in characterization of a broad range of rock mass qualities. In addition, the proposed support pressure function is applicable in squeezing ground condition and anisotropic field stress. Consequently, the rock load which bolts should carry is more realistic.

The bolt density parameter obtained through the support pressure function can also be used in evaluating the reinforcement degree of the rock mass around the tunnel by means of GRC (Indraratna and Kaiser, 1990; Osgoui, 2007; Osgoui and Oreste, 2007). This reinforcement effect of the grouted bolts helps in reducing ultimate support pressure.

2. Definition of support pressure (rock load) based on the Geological Strength Index (GSI)

One of the most important steps in dimensioning the bolt system of a tunnel is that of determining the support pressure that bolts should carry since miscalculations of support pressure may lead to a failure in bolt system.

In general, the load that acts on a support system is referred to as the support pressure. It denotes the rock pressure that results from the rock-load height above the tunnel excavation. In this case, bolts are expected to provide the support pressure as a resistance force required to carry the weight of the failed rock above the tunnel. The support pressure function implicitly depends on the parameters indicated below:

$$P \approx f\left(\mathrm{GSI}, D, \sigma_{cr}, D_e, \gamma, C_s, S_q\right) \tag{1}$$

where GSI is the Geological Strength Index that defines the quality of the rock mass; D the disturbance factor indicating the method of excavation; σ_{cr} the residual compressive strength of the rock in the broken zone around the tunnel; D_e the equivalent diameter of the excavation; γ the unit weight of rock mass; C_s the correction factor for the horizontal to vertical field stress ratio (k), and S_q the correction factor for the squeezing ground condition.

Similar to its previous counterpart, developed by Ünal (1983, 1992, 1996), the main advantage of the newly proposed approach is that the quality of the rock mass is considered as the GSI. Due to its accepted applicability in a broad range of rock mass qualities, the GSI was chosen to signify the rock mass quality in the proposed support pressure formula. This makes it possible to estimate the support pressure (support load) for tunnels in various rock mass qualities provided that the GSI has initially been determined. The Modified-GSI has to be used for very poor or poor rock masses where the GSI $<$ 27, instead of the GSI, for support pressure estimation (Osgoui and Ünal, 2005). It is, therefore, suggested that the new approach be applied to a wide spectrum of rock masses, with qualities ranging from very good to very poor. The steps that were followed to define the support load function were:

I. The original support load function previously developed by Ünal (1983, 1992) is considered to be the main basis for the new equation because it uses Bieniawski's RMR system (1973), which quantitatively evaluates the quality of the rock mass. The original Ünal load equation is (Ünal 1983, 1992):

$$P = \frac{100 - \mathrm{RMR}}{100}\gamma B \tag{2}$$

where B is the longest span of the opening.

II. The new support pressure function is defined in such a way that it does not contradict Ünal's equation (1983, 1992), whose applicability has been widely accepted in the field of mining and tunnelling.

III. The importance of the two parameters (i.e. method of excavation and residual strength of the rock), which are directly related to the damage extension in the rock mass around the tunnel, was inspired by the rock mass deformation equation introduced by Hoek et al. (2002). This deformation modulus is a modified version which previously proposed by Serafim and Pereira (1983) for $\sigma_{ci} < 100$ MPa:

$$E_m = \left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}}10^{\frac{\mathrm{GSI}-10}{40}} \tag{3}$$

The above equation can be re-written as:

$$\frac{E_m}{10^{\frac{\mathrm{GSI}-10}{40}}} = \left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}} \text{ or } \frac{E_{m(\mathrm{Hoek.})}}{E_{m(\mathrm{Serafim\,\&\,Pereira})}} = \left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}} \tag{4}$$

Looking at Eqs. (3) and (4), it can be found that the right hand of the equality is a reduction factor for the estimation of the deformation modulus in weak rock masses. This reduction factor can play a significant role in support pressure function as a sensible way in considering the effect of the rock mass disturbance and intact rock strength.

IV. The definitions of squeezing ground conditions and their correction factors were adopted through descriptions originally introduced by Singh et al. (1992, 1997) and Hoek and Marinos (2000).

V. The effect of the anisotropy in field stress was taken into account similar to the definition given by Ünal (1992).

VI. Since the effect of the horizontal to the vertical field stress (k) was studied through a 2-D numerical plane strain analysis (Osgoui, 2006), so the proposed support function is valid only for cases where the horizontal stresses are equal in each direction.

VII. The proposed support pressure function applies for $\sigma_{ci}<$ 100 MPa. The maximum value of σ_{ci} that should be used in the proposed equation must be 100 MPa even if $\sigma_{ci}>100$ MPa. σ_{ci} is the uniaxial compressive strength of the intact rock.

Considering the original Ünal load Eq. (2), substituting the GSI for RMR, and taking into consideration the reduction factor for rock mass quality through disturbance and strength factors (Eq. (4)), the new equation to estimate the support pressure was proposed in such a way as to keep its original perception:

$$P=\frac{100-\left[\left(1-\frac{D}{2}\right)\sqrt{\frac{\sigma_{cr}}{100}}\mathrm{GSI}\right]}{100}C_sS_q\gamma D_e \tag{5}$$

where $\sigma_{cr}=S_r\cdot\sigma_{ci}$, $0<S_r<1$, S_r = post-peak strength reduction factor, characterizing the brittleness of the rock material as explained later on.

It should be noted that in the aforementioned equations, D_e is the equivalent diameter of the excavation and it is used for any tunnel shape. It can easily be obtained from:

$$D_e=\sqrt{\frac{4A}{\pi}} \tag{6}$$

where A is the cross-section area of the excavation.

Taking into account Ünal's rock-load height concept (Ünal, 1983, 1992), the support pressure function can be dependent on the parameters specified in the following expressions:

$$P\approx f\left(h_t,\gamma,C_s,S_q\right) \tag{7}$$

where h_t is the rock-load height. It is defined as the height of the potential instability zone, above the tunnel crown, which will eventually fall if not supported properly.

Considering support pressure functions (1) and (7) and Eq. (5), the rock-load height, instead, can be expressed as:

$$h_t=\frac{100-\left[\left(1-\frac{D}{2}\right)\sqrt{\frac{\sigma_{cr}}{100}}\mathrm{GSI}\right]}{100}C_sS_qD_e \tag{8}$$

The most common form of the support pressure expression can be written as below when $S_r=1$:

$$P=\frac{100-\left[\left(1-\frac{D}{2}\right)\sqrt{\frac{\sigma_{cr}}{100}}\mathrm{GSI}\right]}{100}C_sS_q\gamma D_e=\gamma h_t \tag{9}$$

2.1. Parameters used in support pressure

2.1.1. Geological Strength Index (GSI) estimation

In view of the fact that the Geological Strength Index (GSI) plays the most dominant role in determining the support pressure, it is of paramount importance that the GSI of a rock mass be estimated accurately. GSI accounts for a large percentage of the support pressure value since it directly reflects the quality of the rock mass around the tunnel. Hence, a distinction to estimate GSI for either fair to good quality rock mass or poor to very poor rock mass must be applied. The boundary that initiates the threshold of poor rock mass is defined as RMR = 30. For fair to good quality rock mass if RMR > 30 then GSI = RMR. Consequently, either qualitative GSI charts (Hoek and Brown, 1997; Hoek, 1999; Hoek and Marinos, 2000; Marinos et al., 2005) or quantitative GSI charts (Sönmez and Ulusay, 1999, 2002; Cai et al., 2004c) can readily be used.

Due to obvious deficiency of GSI in characterizing poor and very poor rock mass where RMR falls below 30 (Hoek, 1994), the Modified-GSI should, instead, be directly or indirectly determined (Osgoui, 2007). The original and the existing GSI charts found in literature are not capable of characterizing poor and very poor rock mass as denoted by N/A in the relevant parts. By adding measurable quantitative input in N/A parts of existing GSI charts, they will be enhanced in characterizing poor rock mass while maintaining its overall simplicity. Further, the new Modified-GSI chart is considered as a supplementary means for its counterparts (Fig. 1). The modified-GSI chart is valid for poor and very poor rock mass with GSI ranging between 6 and 27. In the case of GSI greater than 27, the existing GSI charts mentioned earlier should be used.

To set up the Modified-GSI chart (left side of Fig. 1), two indicators of poor rock mass; namely, Broken Structure Domain (BSTR) and Joint Condition Index (I_{JC}) are defined. The latter is adopted and modified from the Modified-RMR (Özkan, 1995; Ulusay et al., 1995; Ünal, 1996) in order to use in Modified-GSI. For this purpose, a block in the matrix of 2×2 of GSI chart is selected in terms of two axes signifying the rock mass blockiness (interlocking) and joint surface conditions.

As shown in the left side of Fig. 1, the vertical axis of the matrix presents quantitatively the degree of jointing in terms of BSTR. Generally speaking, broken drill-core zones recovered from a very weak rock mass having a length greater than 25 cm are defined as BSTR. Various types of BSTR domains can be categorized into 5 groups based on their size and composition (Osgoui and Ünal, 2005; Osgoui, 2007). Furthermore, the Structure Rating (SR) suggested by Sönmez and Ulusay (1999, 2002) was integrated with Modified-GSI to define the blockiness of rock mass. The original intervals of SR were adjusted to be compatible with BSTR types in Modified-GSI.

The horizontal axis, on the other hand, is assigned for the joint condition rating. In order to determine the Joint Condition Index (I_{JC}), BSTR type, Intact Core Recovery (ICR), and filling and weathering conditions should be known as given in right hand of Fig. 1. ICR is defined as the total length of the cylindrical core pieces greater than 2 cm divided by the total length of the structural region or drill-run. The ICR for poor and very poor rock masses is considered to be less than 25 to satisfy the Modified-GSI requirements. For joint condition rating, the upper part of modified-GSI chart is divided into 2 categories; namely, poor and very poor. For ICR < 25%, the total rating of joint condition index varies between 0 and 16. A simple way for determining joint condition index "I_{JC}"is presented in the right side of Fig. 1.

In the absence of the necessary parameters for GSI determination, the following exponential equation provides a correlation between the Rock Mass Rating (RMR) and the Geological Strength Index (GSI) for a poor rock mass (Osgoui and Ünal, 2005):

$$\mathrm{GSI}=6e^{0.05\mathrm{RMR}}\ \text{if RMR}<30 \tag{10}$$

This correlation has proved to be compatible with those ascertained from many case studies and calibrated with the Modified-GSI chart.

2.1.2. The effect of excavation method

The tunnelling method has a significant influence on the support pressure. Conventional excavation methods (Drill & Blast) cause damage to the rock mass whereas controlled blasting and mechanized tunnelling with Tunnel Boring Machine, TBM, leave the rock mass undisturbed. Singh et al. (1992, 1997) declared that the support pressure could be decreased to 20% for such cases. Moreover, the effects of rock interlocking and stress change (relaxation) as a result of the ground unloading cause a disturbance in the rock mass.

The value of RMR used in the original support pressure equation (Ünal 1983, 1992) included the adjustment for the effects of the blasting damage, change in in-situ stress, and major faults and

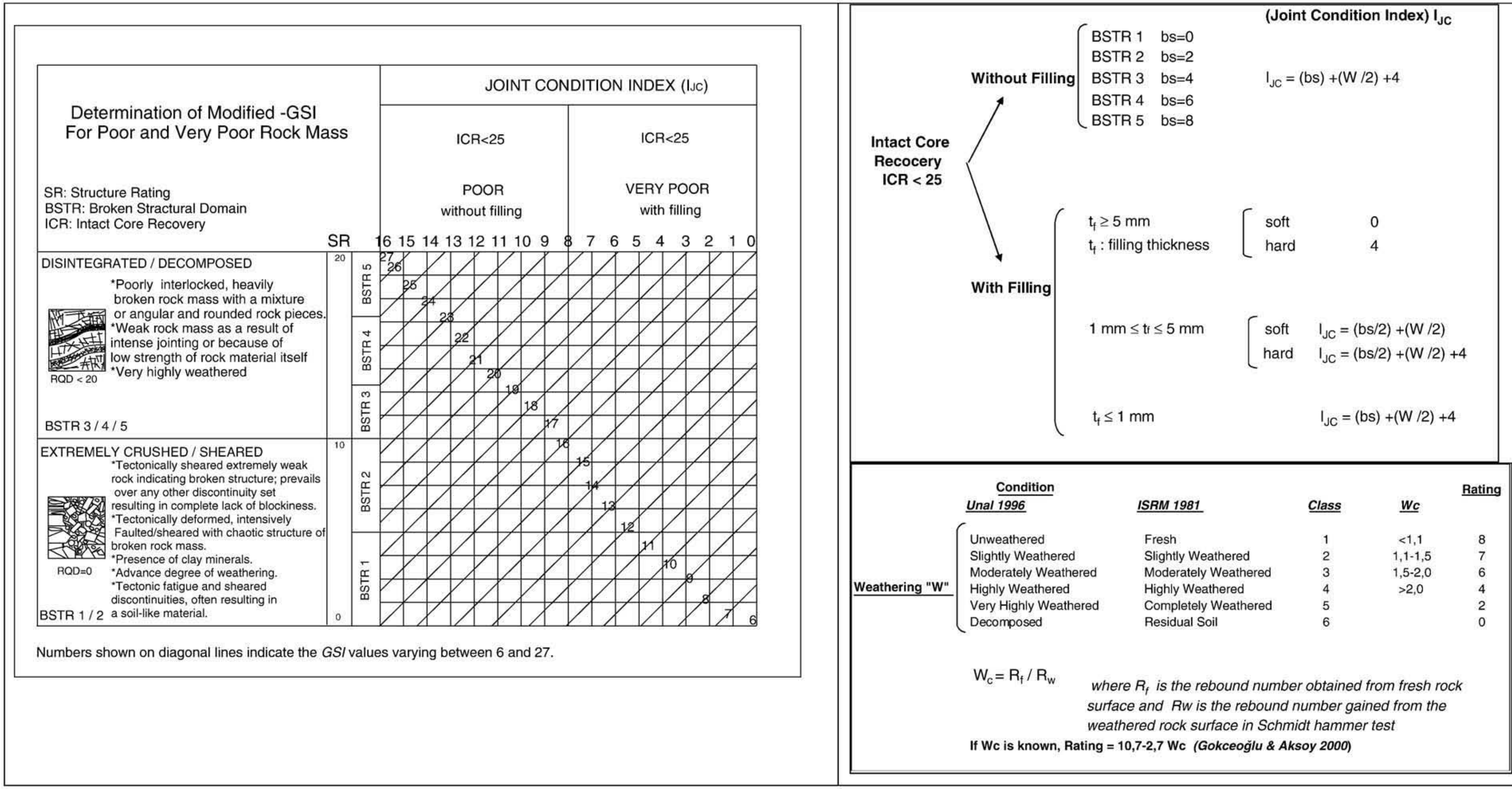

Fig. 1. Modified-GSI chart suggested to be used in proposed approach (GSI < 27: poor to very poor rock mass).

fractures (Bieniawski 1984, 1989). Since GSI is substituted for RMR in proposed approach, those factors should, similarly, be incorporated with the proposed equation. The disturbance factor (D) originally recommended by Hoek et al. (2002) is taken into account to adjust the value of GSI. This factor ranges from $D=0$ for undisturbed rock masses, such as those excavated by a tunnel boring machine, to $D=1$ for extremely disturbed rock masses. This factor also allows the disruption of the interlocking of the individual rock pieces within rock masses as a result of the discontinuity pattern (Marinos et al., 2005). For the same properties of rock mass and tunnel, the support pressure increases as the disturbance factor increases from 0 to 1. Indications for choosing the disturbance factor are given in Table 1.

2.1.3. The effect of residual strength of intact rock

Since the broken zone extension around an underground opening depends on the strength parameters of the rock, it is suggested that the compressive strength of the rock material as an influential parameter in estimating the thickness of the broken zone (rock-load height) and support pressure be taken into account. In the majority of sophisticated closed-form solutions for tunnels, the residual strength parameters are allowed for in calculations in accordance with the post failure behaviour of the rock. It is also substantiated that the extension of the broken zone relies on the residual value of the intact rock strength (Hoek and Brown, 1980; Brown et al., 1983; Indraratna and Kaiser, 1990; Carranza-Torres, 2004).

Hence, the effect of the compressive strength of a rock material must be included in the form of the residual value because it loses its initial value due to stress relief or an increase in the strain. A stress reduction scale should, therefore, be considered as:

$$\sigma_{cr} = S_r \cdot \sigma_{ci} \tag{11}$$

where S_r refers to the strength loss parameter that quantifies the jump in strength from the intact state to the residual condition. The parameter S_r characterizes the brittleness of the rock material: ductile, softening, or brittle. By definition, S_r will fall within the range $0<S_r<1$, where $S_r=1$ implies no loss of strength and the rock material is ductile, or perfectly plastic. In contrast, if S_r tends to 0, the rock is brittle (elastic–brittle plastic) with the minimum possible value for the residual strength as highlighted in Fig. 2. As a first guess in proposed support pressure equation (Eq. (5)), $S_r=1$ is taken into account for the poor and very poor rock masses with GSI<27 because their post-failure behaviour is perfectly-plastic (Hoek and Brown, 1997). For average and good quality rock masses, on the other hand, the exact value of the residual strength for the intact rock can be determined from stress–strain response of rock in laboratory tests, so the value of S_r can readily be obtained (Aydan et al., 1996; Cundall et al., 2003).

Table 1
Modified guideline for estimating disturbance factor (D), which initially suggested by Hoek et al. (2002).

Description of rock mass	Suggested value for D
Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel.	$D=0$
Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass.	$D=0$
Usual blasting that causes local damages.	$D=0.5$
In mechanical excavation where squeezing problems result in significant floor heave unless a proper invert is placed.	$D=0.5$
Very poor quality blasting in tunnel results in severe damages, extending 2 or 3 m, in the surrounding rock mass.	$D=0.8$
Very poor quality blasting along with a intensive squeezing ground condition in tunnel — unexpectedly heavy blasting in caverns leading to significant cracks propagation on roof and walls.	$D=1$

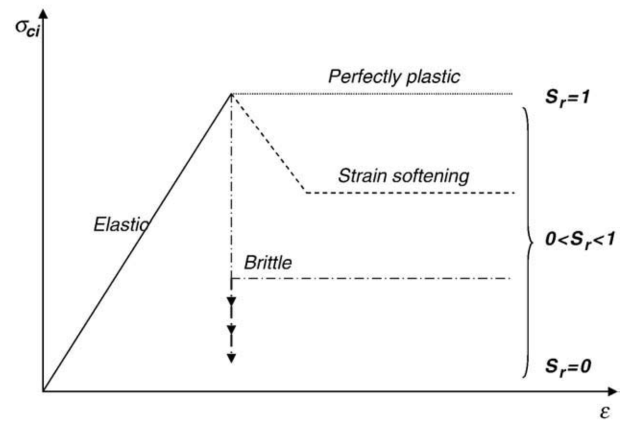

Fig. 2. Different post-peak strength models of rocks.

2.1.4. The effect of squeezing ground condition

In view of the fact that almost all deep tunnelling works in poor rock masses undergo squeezing ground, it is of paramount importance to take this effect into consideration in precisely estimating the support pressure. The squeezing degree is expressed in terms of tunnel convergence or closure (Indraratna and Kaiser, 1990; Singh et al., 1992, 1997), strength factor (Bhasin and Grimstad, 1996; Hoek and Marinos, 2000), or critical strain concept (Hoek and Marinos, 2000; Lunardi, 2000). Since tunnel convergence (closure) is an important indicator of tunnel stability, the squeezing behaviour has been evaluated in terms of tunnel convergence in the current study. The squeezing correction factor used in the proposed approach were adopted and modified from the results of Singh et al. (1992, 1997) and Hoek and Marinos (2000), as outlined in Table 2.

2.1.5. The effect of anisotropy in field stress

Numerical analysis of the broken zone around the tunnel implied that the extension of failure heights above tunnels and consequently the support pressure depend upon the magnitude of the stress ratio (k) For arch-shaped and rectangular tunnels, the extent of the failure zone decreases as the value of k changes from 0.3 to 0.5; conversely, the height of the failure zone starts to increase again as the value of k approaches 2.5 (Osgoui, 2006).

The failure height (obtained from numerical methods) and rock-load height (determined by the proposed formula) ratio yields a value called the stress correction factor (C_s). This correction value should be applied when using Eqs. (5), (8), and (9). Therefore, a multiplier (C_s) is required to correct the stress ratio. For the reason of reliability, the minimum value of C_s is suggested as 1.0 for $k=0.5$. Fig. 3 aims at choosing the stress correction factor for proposed approach.

3. Bolt density parameter

The dimensionless bolt density parameter, firstly defined by Indraratna and Kaiser (1990), can be written as follows to adopt any tunnel shape:

$$\beta = \frac{\pi d \lambda}{S_L \theta} = \frac{\pi d \lambda r_e}{S_L S_T} \tag{12}$$

where d is the bolt diameter; λ the friction factor for bolt–grout interface that relates the mobilized shear stress acting on the grouted bolt to the stress acting normal to the bolt; r_e the equivalent radius of the tunnel opening; S_T the transversal bolt spacing around the tunnel; S_L the longitudinal bolt spacing along the tunnel axis; θ the angle between tow adjacent bolts (i.e. $S_T=r_e\times\theta$) in an axi-symmetrical problem and considering identical bolt with equal spacing along the tunnel axis.

Table 2
Suggested values for squeezing ground condition correction factors (S_q) used in empirical approach (adopted and modified from Hoek and Marinos, 2000; Singh et al., 1997).

Strains %(tunnel closure or convergence/tunnel diameter) *100	Rock mass strength/ in-situ stress (σ_{cm}/P_o)	Comments	Suggested correction factor (S_q)for squeezing ground condition
<1% No squeezing	>0.5 Mostly (>1.0)	The strength of the rock mass exceeds the stress level at the face and mostly around the cavity. The ground behaviour is elastic. Instabilities are associated with rock block and rock wedge failures.	1.0
1–2.5% Minor squeezing	0.3–0.5	The magnitude of stress at the face approaches the strength of the rock mass. The behaviour is elastic-plastic. The deformability gradient at the face is low. On the periphery of the cavity the stresses exceed the strength of the rock mass, SR<1, resulting in the formation of a plastic zone around the excavation.	1.5
2.5–5% Severe squeezing	0.2–0.3	The magnitude of stress at the face exceeds the strength of the rock mass. Although face is in plastic zone, the deformation gradient is low for typical advance rate; therefore, immediate collapse of the face is prevented. The plastic state at the face in conjunction with the development of the plastic zone around the tunnel results in severe overall stability.	0.8
5–10% Very severe squeezing	0.15–0.2	At tunnel face and periphery, the stress-to-strength state results in high deformation gradient and critical conditions for stability.	1.6
>10% Extreme squeezing	<0.15	Immediate collapse (very short stand-up-time) of the face during excavation. This behaviour is associated with non-cohesive soils and very poor rock mass rock masses such as these found in sheared zones.	1.8

The bolt density parameter reflects the relative density of bolts with respect to the tunnel perimeter and takes into consideration the shear stresses on the bolt surface, which oppose the rock mass displacements near the tunnel wall.

The value of β varies between 0.05 and 0.20 for most cases. For tunnels excavated in very poor rock mass analyzed by Indraratna and Kaiser (1990) very high values for β (in excess of 0.4) were reached by very intensive bolting patterns. The friction factor λ, being analogous to the coefficient of friction, relates the mean mobilized shear stress to the stress applied normal to the bolt surface. Indraratna and Kaiser (1990) suggested that the magnitude of λ for smooth rebars falls in the range $\tan(\varphi_g/2)<\lambda<\tan(2\varphi_g/3)$ and for shaped-rebars approaches $\tan\phi_g$, depending on the degree of adhesion (bond strength) at the bolt–grout interface.

The bolt density parameter has been used to introduce the equivalent strength parameters in terms of either Mohr–Coulomb (c^*, ϕ^*) or Hoek–Brown (s^*, m^*, σ_{ci}^*) failure criteria (Grasso et al., 1989; Indraratna and Kaiser, 1990; Osgoui, 2007; Osgoui and Oreste, 2007). The equivalent strength parameters, marked with *, belong to the reinforced plastic zone around the tunnel. Consequently, the concept of equivalent plastic zone has been introduced to describe the extent of yielding reinforced with grouted bolts.

As grouted bolts effectively improve the apparent strength of the rock mass, the behaviour of the improved ground around a reinforced tunnel can ideally be represented by Ground Reaction Curve (GRC), using elasto-plastic solutions found in the literature, for example: Brown et al. (1983), Panet (1993), Ducan Fama (1993), Labiouse (1996), Carranza-Torres and Fairhurst (2000), Carranza-Torres (2004), Osgoui (2007), and Osgoui and Oreste (2007).

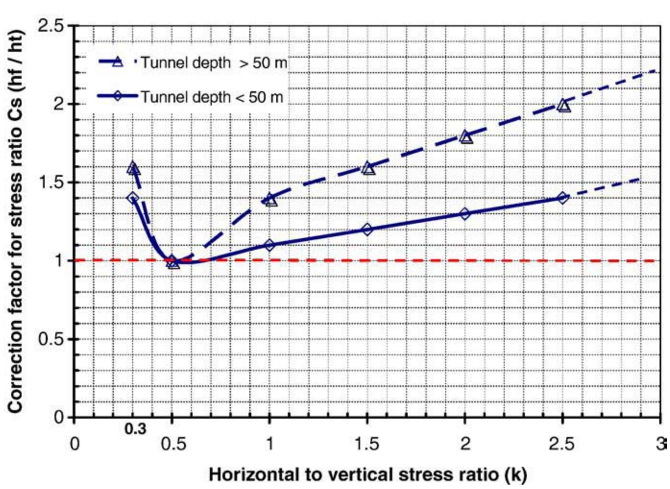

Fig. 3. Suggested values for stress correction factor (Cs) used in proposed formula (hf: failure height by numerical method, ht; rock-load height by empirical approach).

4. Bolt design

Since during initial stage of the bolt design, the pattern (bolt spacing) is not known, it is not advisable to determine the bolt density parameter through guessed pattern. Instead, the combination of the bolt density parameter and support load function is supposed to provide a practical way to obtain more realistic value for the bolt density parameter.

The supporting action is assumed to be provided by rock bolts carrying a total support load defined by the rock- load height (Eq. (8)). Hence, if the effective area of each bolt is defined as the area of longitudinal and transversal spacing (see Fig. 4), the following equation relates the acting support load (Eq. (5)) to the bolt capacity (C_b) in a condition of equilibrium:

$$C_b = P \cdot S_T \cdot S_L \tag{13}$$

Defining a factor of safety (FOS), which is defined as the resistant force to the imposing force, Eq. (13) can be re-written as:

$$\mathrm{FOS} = \frac{C_b}{P \cdot S_T \cdot S_L} \tag{14}$$

A value for FOS is suggested only for mining applications as suggested by Bieniawski (1984). By substituting Eq. (14) into Eq. (12) and equating FOS = 1(limiting equilibrium state), the correlation between the rock bolt density (β) and the support load (P) will be:

$$\beta = \frac{P\pi d\lambda r_e}{C_b} \tag{15}$$

The main application of the Eq. (15) is that of determining the bolt density parameter that considers the support pressure. Simultaneously, the already determined bolt density parameter can then be used as the first estimate of β in analysis of the reinforced tunnel in terms of the Ground Reaction Curve (GRC) so as to evaluate the reinforced rock mass and the tunnel convergence. By doing so, the effectiveness of the bolting pattern is consequently evaluated. For this purpose, the equivalent strength parameters of the reinforced rock mass around the tunnel can be determined in terms of either Mohr–Coulomb or Hoek–Brown failure criteria.

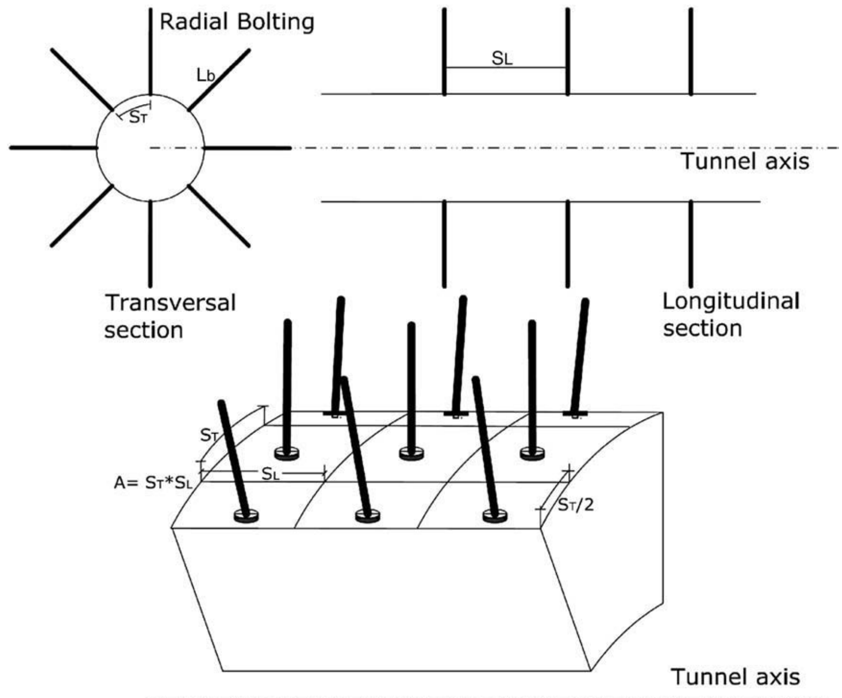

Fig. 4. Bolting pattern (transversal and longitudinal) for a circular tunnel.

For Mohr–Coulomb material (Indraratna and Kaiser, 1990):

$$\sigma_1 = C_0^* + k_p^* \sigma_3 \tag{16a}$$

$$k_p^* = k_p(1+\beta) \tag{16b}$$

$$C_0^* = C_0(1+\beta) \tag{16c}$$

$$k_p = \tan^2\left(45 + \frac{\phi}{2}\right) = \frac{1+\sin\phi}{1-\sin\phi} \tag{16d}$$

$$C_0 = 2c\tan\left(45 + \frac{\phi}{2}\right) = \frac{2c\cos\phi}{1-\sin\phi} \tag{16e}$$

For Hoek–Brown material (Osgoui, 2007; Osgoui and Oreste, 2007):

$$\sigma_1 = \sigma_3 + \sigma_{ci}^*\left(m_b^*\frac{\sigma_3}{\sigma_{ci}^*} + s^*\right)^a \tag{17a}$$

$$m_b^* = (1+\beta)\cdot m_b \tag{17b}$$

$$s^* = (1+\beta)\cdot s \tag{17c}$$

$$\sigma_{ci}^* = (1+\beta)\cdot\sigma_{ci} \tag{17d}$$

where σ_1 and σ_3 are the maximum and the minimum principal stresses; C_0 the uniaxial compressive strength of rock mass; ϕ the friction angle of rock mass, c the cohesion of rock mass, m_b, s, a the strength constants of rock mass, and σ_{ci} the uniaxial compressive strength of the intact rock. The superscript * stands for the material with properties equivalent to those of a reinforced rock mass.

Substituting the equivalent strength parameters (Eqs. (16) and (17)) for the strength parameters of the original rock mass in elasto-plastic solution makes it possible to produce the GRC of a reinforced rock mass as illustrated in Fig. 5.

Furthermore, the advantage of using the Eq. (15) is attributed to the fact that the first approximate of β can easily be obtained through a real characterization of rock mass because two significant indicators of rock mass characterization have been incorporated, i.e. the support load and GSI or Modified-GSI.

For an equal-spaced bolting pattern, the bolts spacing is calculated by Eq. (18) which is derived from Eqs. (13) and (15):

$$S_s = \sqrt{\frac{\pi d \lambda r_e}{\beta}} \tag{18}$$

It is interesting, at this point, to note that the influence of bolt–ground interaction, a very important design parameter, was included in the proposed method. The friction factor, λ, integrated in bolt density parameter is a characteristic parameter for the bolt–ground interaction. Based on the results of the experimental tests (Indraratna and Kaiser, 1990) and of the numerical analyses (Osgoui, 2007), the value of λ = 0.6 provides more realistic result for shaped-rebars.

The variation of the bolt density parameter with the support pressure for a tunnel with the span of 5 m reinforced with the

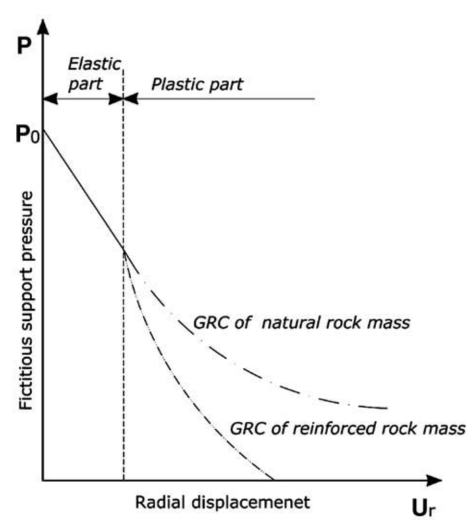

Fig. 5. Typical Ground Reaction Curve (GRC) in presence of rock reinforcement. Key: P: fictitious support pressure; u_r: radial displacement of the tunnel wall.

different types of grouted rock-bolts (MAI-bolts, Atlas Copco, 2004) is represented in Fig. 6, indicating that with increasing support load; more severe bolt density is required to satisfy the minimum stability condition of the tunnel.

Fig. 7 illustrates the correlation of the bolt density parameter β and support load for a tunnel of 2.5 m diameter which is excavated in relatively poor to very poor rock mass (GSI < 40). For the different range of support loads provided at GSI < 40, the variations of bolt density β change between 0.10 and 0.25 depending on the types of applied grouted bolts.

With reference to Fig. 7, it can be inferred that the appropriate choice of bolt density parameter in design, regardless of grouted bolt types, can lead to the same results for different support pressure magnitudes. To illustrate, for a support pressure of 0.296 MPa associated with Modified-GSI = 23, the primary stability of the tunnel can be achieved by installation of arbitrary pattern of grouted bolts of 25 mm, 32 mm, and 51 mm in diameters corresponding to bolt densities of 0.23, 0.16, and 0.11, respectively. It should be noted that the usage of the thicker and longer grouted bolts having higher yield capacity is preferably suggested in poor rock masses.

Since tunnel convergence is the main indicator of tunnel stability, the bolting performance is best evaluated in terms of its effect of the tunnel convergence. This technique has plentifully addressed in literature (Indraratna and Kaiser, 1990; Oreste, 2003; Osgoui and Oreste, 2007).

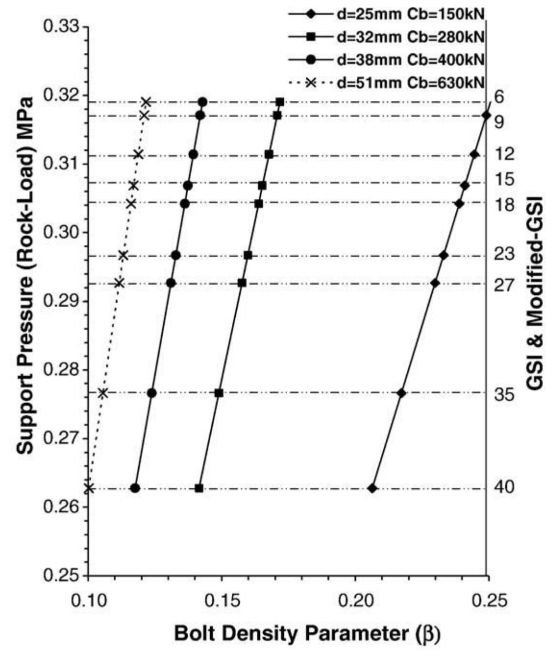

Fig. 7. Correlation between the bolt density parameter and support pressure for a tunnel of 2.5 m radius excavated in relatively poor to very poor rock mass (GSI < 40) and undergoing a very high squeezing, $\frac{\sigma_h}{\sigma_v} = 1.5$.

4.1. Bolt length

Although many empirical approaches have been suggested, the only appropriate criterion for bolt length is to know the thickness of the plastic zone around the tunnel using existing elasto-plastic solutions. To have a successful bolting pattern, the anchor length of the grouted bolt should exceed the thickness of the plastic zone, taking into account the radius of reinforced plastic zone in presence of bolts (r^*_{pe}) and the radius of the tunnel (r_e) as shown in Fig. 8. In this way, the systematic bolts around the tunnel perform properly, particularly in reducing the convergence of the tunnel.

Substituting the equivalent strength parameters (Eqs. (16) and (17)) for the strength parameters of the original rock mass in elasto-plastic solution makes it possible to determine the radius of reinforced plastic zone.

Based on results of numerous elasto-plastic analyses carried out by Oreste (2003) and Osgoui (2007), a simple criterion that defines the grouted bolt length (L_b) has been recommended as:

$$r^*_{pe} < (r_e + a \cdot L_b), \text{where } a = 0.5 - 0.75 \qquad (19)$$

where r^*_{pe} is the reinforced plastic zone radius. In fact, this criterion provides the prevention of the plastic zone radius further than the anchor length of the bolt.

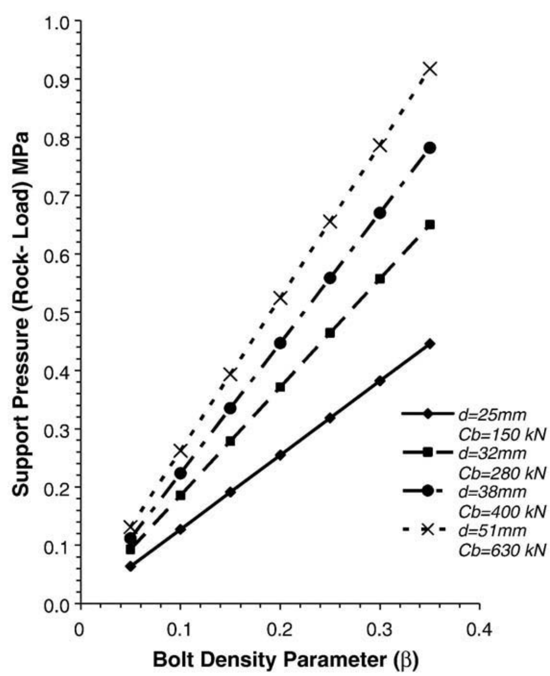

Fig. 6. Variation of the bolt density parameter with support pressure (rock load) for different types of grouted bolts in terms of their yield capacity and diameter.

5. Comparison with alternative empirical design methods

The RMR system of Bieniawski (1973, 1989) has been acknowledged to be applicable to fully grouted rock bolts in all type of rock

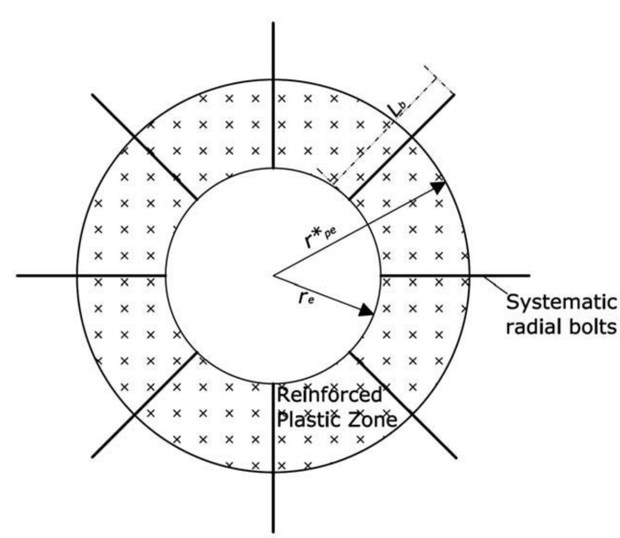

Fig. 8. The extension of reinforced plastic zone with respect to bolt length in a systematic bolt pattern around a circular tunnel.

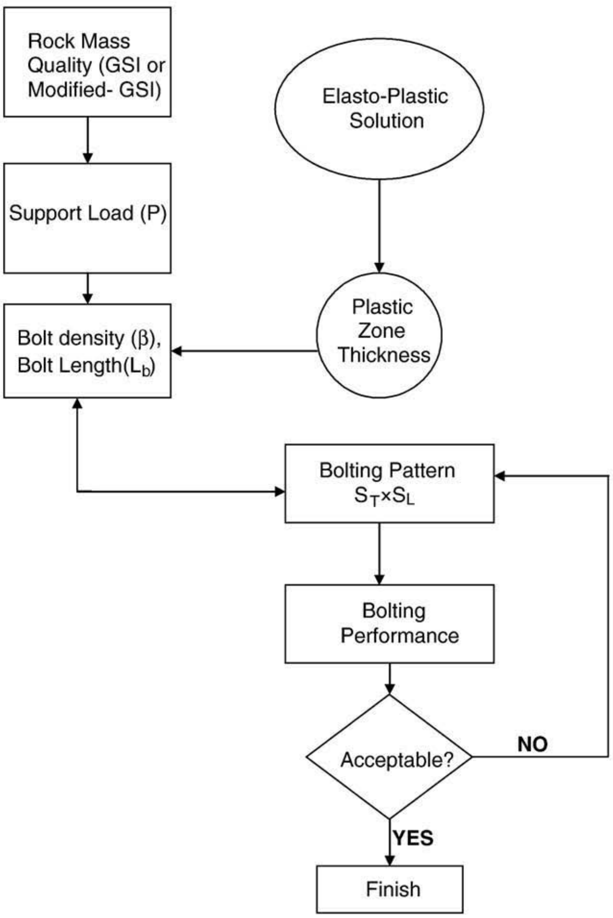

Fig. 9. Computational steps of the proposed methodology for design of grouted bolt.

masses. The design tables and recommendations proposed by Bieniawski are intended for tunnels in the order of 10 m width, excavated by the drill and blast method at depths of less than 1000 m and reinforced by 20 mm diameter grouted bolts. Supplemental support by shotcrete, wire mesh and steel sets are also suggested for poorer ground. Conversely, in not all types of rock mass qualities, the use of the grouted bolts has been recommended in Q-system (Barton et al., 1974; Grimstad and Barton, 1993; Barton, 2002). Most recently, Palmström and Broch (2006) have also pointed out that " the suggested support systems by Q-system work best in a limited domain between 0.1 and 40 and outside this range, supplementary methods and calculations should be applied".

With the help of the proposed method, it is possible to set up several informative tables, like that of Bieniawski (1973, 1989) and Barton et al. (1974), for any rock mass condition and tunnel size provided that the rock mass quality is accurately defined and the bolt density parameter and the strength parameters of reinforced rock mass are estimated accurately. For this purpose, the proposed methodology can be applied as shown in the flowchart of Fig. 9. An example of such informative tables that recommends the bolt configurations is represented in Table 3.

Fig. 10. Huge collapse as e result of remarkable amount of convergence in squeezing ground condition in Malatya Railroad Tunnel.

It should be noted that only a change in bolt length for displacement control in the poor rock mass is not enough. For instance, a bolt density parameter (β) more than doubles is needed as the spacing is decreased from 1.5 m to 1.0 m when the quality of rock mass diminishes from fair to poor. Hence, a further reduction of the bolt spacing for the poorest rock class would provide a sufficiently high magnitude for β to curtail displacements more effectively than by increasing the bolt length. This is supported by Laubscher (1977) who proposed a bolt spacing less than 0.75 m for poor ground at RMR < 30.

Since RMR or Q-system may not provide a sufficiently sensitive guide to properly design the grouted bolts in relatively poor to very poor rock mass (for rock classes RMR < 40) as also reported by Indraratna and Kaiser (1990), the proposed alternative design method provides a sound basis for effective bolt design in such rock masses as indicated in Table 3.

6. Application of the proposed empirical approach: a case study from Turkey

The 537 m long Malatya railroad tunnel being 5 m wide, situated in the South-Eastern part of Turkey, was excavated in 1930 through a toe of a paleo-landslide material. This sheared zone of rock-mass around the tunnel consists of metavolcanics, schist, fractured limestone blocks, antigorite and radiolarite in patches. The matrix material consists of clay and schist with low swelling potential. Limestone blocks are heavily jointed and highly fractured and weathered. There are also voids within the rock mass (Osgoui and Ünal, 2005). Ever since 1930, this horseshoe shape tunnel has struggled with severe

Table 3
Recommended grouted bolt densities for a reinforced tunnel of 2.5 m radius for different rock classes (field stress = 15 MPa, grouted bolt diameter = 32 mm), key: the use of bolt couplings for installation of long bolts in small diameter tunnel must be applied.

GSI	Definition	L_b (m)	S_T and S_L (m) squared pattern assumed	β/λ	β at $\lambda = 0.6$	Possible yielding around the tunnel
81–100	Very good		No support required		0.0	No yielding
61–80	Good	2 to 3	2.0–2.5	0.06–0.04	0.038–0.024	Minimal
41–60	Fair	3 to 5	1.5–2.0	0.11–0.06	0.067–0.038	Minimal–major
31–40	Relatively poor	5 to 6	1.0–1.25	0.25–0.11	0.151–0.067	Major
21–30	Poor	≥6	1.0	0.25	0.151	Major–excessive
<20	Very poor	≥6	0.8	0.39	0.236	Excessive

Table 4
Parameters used in calculating Modified-GSI and support pressure.

Calculation of Modified-GSI (Fig. 1)		Estimation of the support pressure	
Prevailing BSTR type = 2	Completely loss of blockiness	Rock mass unit weight (γ)	0.025 MN/m^3
Intact core recovery (ICR)	<25%	Disturbance factor (d)	0.5 (Table 1), usual blasting with local damage
Joint condition index (I_{JC}) = 7	ICR < 25%, bs = 2, completely weathered W = 2, without filling	Correction factor for squeezing ground condition (S_q)	1.8 (Table 2), extreme degree of squeezing, considering $\left(\frac{\sigma_{cm}}{p_o} = 0.101\right)$
Modified-GSI = 13–15	Very poor rock mass	Correction factor for stress field (C_s)	1.4 (Fig. 3), best assumption for relatively shallow tunnel in very poor rock mass is $k = \frac{\sigma_h}{\sigma_v} = 1.0$
		σ_{cr} = 5 MPa	σ_{ci} = 5 MPa, S_r = 1(Eq. (11)), for perfectly plastic material that should be used for very poor rock mass.

stability problems. These problems are associated with the existence of a very poor rock mass around the tunnel, underground water or seepage pressure, and considerable amount of convergence (squeezing phenomenon). A large amount of deformation developing through many years and leading to misalignment of the tunnel was observed. This excessive deformation is mainly attributed to the squeezing

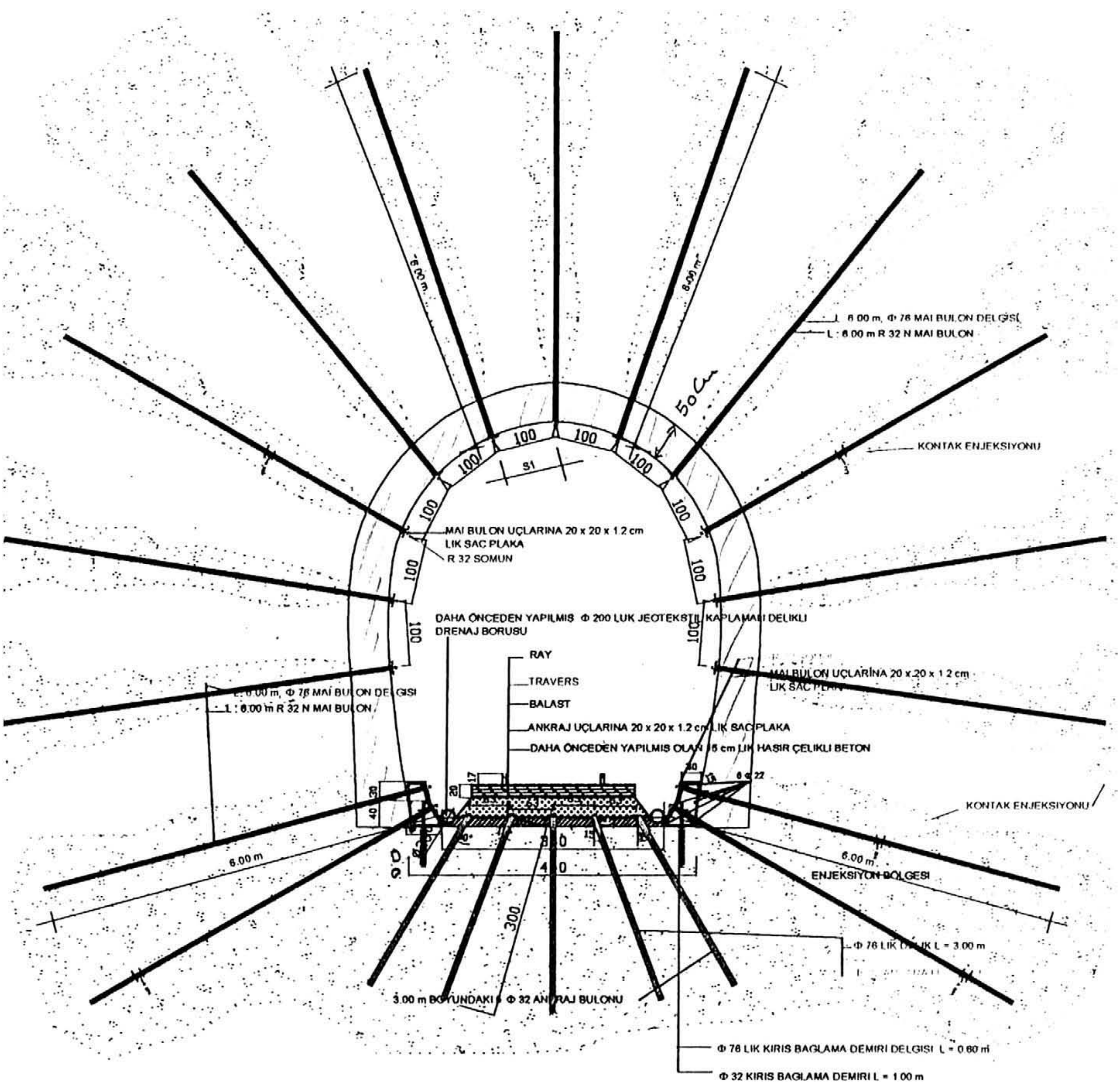

Fig. 11. Systematic bolt pattern around Malatya Railroad Tunnel.

ground condition in which the rock stress exceeds its strength in the passage of time.

As for squeezing ground condition, the in-situ stress was assumed to equal to product of the depth below surface and the unit weight of the rock mass. Considering that the vertical in-situ stress and the rock mass strength were 1.54 MPa and 0.156 MPa, the rock mass strength to in-situ stress ratio is 0.101. Hence, even from this criterion, a huge amount of convergence would have been anticipated. In 2002, a collapse occurred inside the tunnel followed by a failure in support system. This phenomenon was ascribed, to a great extent, to the severe squeezing ground condition along with remarkable amount of uncontrolled convergence as shown in Fig. 10.

Following a comprehensive geological and geomechanical investigation, the overall Modified-GSI of rock mass was approximately calculated 13–15 as, in detail, outlined in Table 4.

Having determined the Modified-GSI, the expected support load was estimated 0.31 MPa using Eq. (5). The value of parameters used in estimating the support pressure is given in Table 4.

The primary support system prior to rehabilitation of the Malatya Tunnel was comprised of steel-sets, concrete lining, and final bricking. In some parts of tunnel withstood a severe squeezing condition, the support system failed. In order to re-design the support system, the use of grouted bolts was recommended as an active primary support element to control the convergence, followed by a convergence measurement program. The final lining was comprised of the combination of the steel-sets and concrete lining to guarantee the long term stability of the tunnel.

In order to understand the short term behaviour of the tunnel and to evaluate its stability before rock bolts applications, a series of convergence measurements were carried out. A total of 15 monitoring stations were set up inside the tunnel, 11 of which were installed in the deformed section and 4 of which were set up in the non-deformed part. The period of convergence monitoring of the tunnel was around 100 days. A maximum horizontal displacement of 8.78 mm was recorded in 3 stations inside the deformed part of the tunnel, indicating that there were still horizontal movements.

For poor rock mass surrounding the Malatya tunnel, MAI-bolts (Atlas Copco, 2004), which are self-driving full column cement-grouted bolts, were preferred to be the most suitable because drill holes usually close before the bolt has been installed, and the injection operation associated with rock-bolting make the ground improved in terms of engineering parameters. Therefore, it was expected that with the use of systematically grouted bolts the extent of the yielding and convergence decreased. The MAI-bolts, like ordinary grouted bolts, develop load as the rock mass deforms. Relatively small displacements are normally sufficient to mobilize axial bolt tension by shear stress transmission from the rock to the bolt surface (Indraratna and Kaiser, 1990).

For rock reinforcement design, the squeezed-section of the tunnel that should be supported was divided into three groups namely; A1, A2, and B. Using MAI-bolts with diameter of 32 mm and yield capacity of 280 kN, the bolt density (Eq. (15)) was obtained $\beta = 0.17$. Then, the bolt spacing was calculated 0.94 m, using Eq. (18), to create a systematic bolting pattern of 1.0 m × 0.9 m (i.e. $S_T = 1.0$ m × $S_L = 0.9$ m, S_T,S_L = transversal and longitudinal spacing, respectively).

Depending on group types of the tunnel section, the bolt length (L_b) of 6 m and 9 m were preferred to satisfy the criterion considered in Section 4.1. These defined lengths prevented the plastic zone thickness from exceeding the anchor length of the bolts. A total of 15 rock bolts were installed around the tunnel except for invert. Floor was supported by installing 5 grouted bolts with the length of 5 m. The bolt pattern applied for Malatya tunnel is illustrated in Fig. 11.

It is interesting to note that based on the convergence–confinement analysis, the equilibrium support load (pressure) of 0.31 MPa for 1.12% strain ($\frac{u_r}{r_i}\% = \frac{0.028}{2.5} \times 100$) is achieved through a combination of two support elements as illustrated in Fig. 12. This value of the support pressure is equal to that which estimated from the empirical equation. TH type of steel-sets, spaced at 1 m, embedded in a 20 cm thick concrete will produce maximum and equilibrium support pressures of 2.6 MPa and 2.3 MPa, respectively. Alternatively, a 1.0 m × 1.0 m pattern of 34 mm diameter mechanical rock bolts together with a 20 cm concrete lining will provide the required support pressure. However, in view of the uncertainty associated with the reliability of the anchorage in this poor rock mass, the use of mechanical rock bolts should be avoided.

7. Conclusions

An empirical-based method for design of grouted bolts in tunnels has been developed. The proposed approach provides a step-by-step procedure to design the grouted bolt and to set up the practical guidelines for optimum pattern of rock bolting. Incorporating the bolt density parameter and support pressure function, a practical means to depict the optimized bolting pattern for any shape of tunnel has been

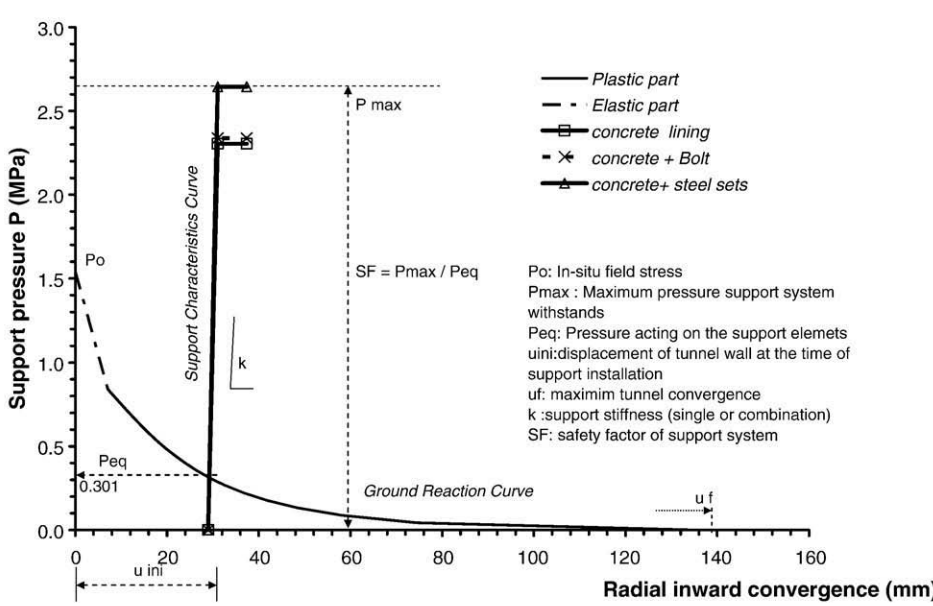

Fig. 12. Required support pressure obtained by convergence–confinement analysis for Malatya tunnel.

introduced. The obtained bolt density parameter can also be used in analysis of the reinforced tunnel in terms of GRC, whereby the reduction in tunnel convergence can be more comprehensible. Therefore, the effectiveness of a bolting pattern can be best evaluated in terms of tunnel convergence which is the significant indicator of tunnel stability. The GSI or Modified-GSI included in the support pressure function makes it possible to effectively design the grouted bolts for a wide range of rock mass qualities. Since RMR or Q-system may not provide a sufficiently sensitive guide to properly design the grouted bolts in relatively poor to very poor rock mass, the proposed alternative design method based on GSI provides a sound basis for effective bolt design in such rock masses as in turn applied in Malatya railroad tunnel in Turkey.

References

Atlas Copco, 2004. MAI, International GmbH, Werkstraße 17, A-9710 Feistritz/ Drau. Austria. http://www.mai.co.at/ankere/anker_frame.htm.

Aydan, Ö. 1989. The Stabilisation of Rock Engineering Structures by Rockbolts. Ph.D thesis, Nagoya University, Japan.

Aydan, Ö., Akagi, T., Kawamoto, T., 1996. The squeezing potential of rock around tunnels: theory and prediction with examples taken from Japan. Rock Mech. Rock Eng. 29 (3), 125–143.

Barton, N., 2002. Some new Q value correlations to assist in site characterization and tunnel design. Int. J. Rock Mech. Min. Sci. 39, 185–216.

Barton, N., Lien, R., Lunde, J., 1974. Engineering classification of rock masses for the design of tunnel support. Rock Mech. Rock Eng. 6 (4), 189–239.

Bawden, W.F., Hyett, A.J., Lausch, P., 1992. An experimental procedure for the in situ testing of cable bolts. Int. J. Rock Mech. Min. Sci. Geomech. Abstr. 29 (5), 525–533.

Bhasin, R., Grimstad, E., 1996. The use of stress–strength relationship in the assessment of tunnel stability. Proc. Recent Advances in Tunnelling Technology, New Delhi, India.

Bieniawski, Z.T., 1973. Engineering classification of jointed rock masses. Trans. S. Afr. Inst. Civ. Eng. 15, 335–344.

Bieniawski, Z.T., 1984. Rock Mechanics Design in Mining and Tunneling. A.A. Balkema. 272 pp.

Bieniawski, Z.T., 1989. Engineering Rock Mass Classification. John Wiley & Sons, New York. 251 pp.

Brady, B.H.G., Loring, L., 1988. Analysis of rock reinforcement using finite difference method. Comput. Geotech. 5, 123–149.

Brown, E.T., Bray, J.W., Landanyi, B., Hoek, E., 1983. Ground response curves for rock tunnels. J. Geotech. Eng. 109, 15–39.

Cai, Y., Esaki, T., Jiang, Y., 2004a. A rock bolt and rock mass interaction model. Int. J. Rock Mech. Min. Sci. 41, 1055–1067.

Cai, Y., Esaki, T., Jiang, Y., 2004b. An analytical model to predict axial load in grouted rock bolt for soft rock tunnelling. Tunnel. Underground Space Technol. 18, 347–363.

Cai, M., Kaiser, P.K., Uno, H., Tasaka, Y., Minami, M., 2004c. Estimation of rock mass deformation modulus and strength of jointed hard rock masses using the GSI System. Int. J. Rock Mech. Min. Sci. 41 (1), 3–19.

Carranza-Torres, C., 2004. Elasto-plastic solution of tunnel problems using the generalized form of the Hoek–Brown failure criterion. Int. J. Rock Mech. Min. Sci. 4 (supplement 1), 629–639.

Carranza-Torres, C., Fairhurst, C., 2000. Application of convergence–confinement method of tunnel design to rock masses that satisfy the Hoek–Brown failure criterion. Tunnel. Underground Space Technol. 15, 187–213.

Chen, S., Qiang, S., Chen, S., Egger, P., 2004. Composite element model of the fully-grouted rock bolt. Rock Mech. Rock Eng. 37 (3), 193–212.

Cundall, P., Carranza-Torres, C., Hart, R., 2003. A new constitutive model based on the Hoek–Brown failure criterion. Proc. 3rd Int. FLAC Symposium. FLAC and numerical modelling in geomechanics, Sudbury, Canada.

Duan, F. 1991. Numerical modelling of cable bolt support systems. Ph. D thesis. The University of Utah. Utah.

Ducan Fama, M.E., 1993. Numerical modelling of yield zones in weak rocks. In: Hudson, J.A. (Ed.), Comprehensive rock engineering, vol. 2. Pergamon, Oxford, pp. 49–75.

Freeman, T.J., 1978. The behaviour of fully-bonded rock bolts in the Kielder experimental tunnel. Tunn. Tunn. 10, 37–40.

Grasso, P., Mahtab, A., Pelizza, S., 1989. Reinforcing a rock zone for stabilizing a tunnel in complex formations. Proc. Int. Cong. Progress and Innovation in Tunnelling, vol. 2, pp. 671–678. Toronto.

Grimstad, E., Barton, N., 1993. Updating the Q-system for NMT. Int. Symp. Sprayed Concrete, Fagernes, Norway, pp. 46–66.

Guan, Z., Jiang, Y., Tanabasi, Y., Huang, H., 2007. Reinforcement mechanics of passive bolts in conventional tunnelling. Int. J. Rock Mech. Min. Sci. 44, 625–636.

Hoek, E., 1994. Strength of rock and rock masses. ISRM News J. 2 (2), 4–16.

Hoek, E., 1999. Putting numbers to geology — an engineer's viewpoint. Q. J. Eng. Geol. 32 (1), 1–19.

Hoek, E., Brown, E.T., 1980. Underground Excavations in Rock. Institution of Mining and Metallurgy, London. 527 pp.

Hoek, E., Brown, E.T., 1997. Practical estimates of rock mass strength. Int. J. Rock Mech. Min. Sci. 34 (8), 1165–1186.

Hoek, E., Marinos, P., 2000. Predicting Tunnel Squeezing. Tunnels and Tunnelling International. Part 1 — November Issue. 45–51, Part 2 — December, pp. 34–36.

Hoek, E., Carranza-Torres, C.T., Corkum, B., 2002. Hoek–Brown failure criterion—2002 edition. Proc. 5th North American Rock Mechanics Symposium, Toronto, Canada, vol. 1, pp. 267–273.

Hyett, A.J., Moosavi, M., Bawden, W.F., 1996. Load distribution along fully grouted bolts, with emphasis on cable bolt reinforcement. Int. J. Numer. Anal. Methods Geomech. 20, 517–544.

Indraratna, B., Kaiser, P.K., 1990. Design for grouted rock bolts based on the convergence control method. Int. J. Rock Mech. Min. Sci. Geomech. Abstr. 27, 269–281.

Kaiser, P.K., Diederichs, M., Yazici, S., 1992. Cable bolt performance during mining induced stress change—a case study. In: Kaiser, P.K., McCreath, D.R. (Eds.), Proc. Int. Symposium on Rock Support, pp. 367–376. Sudbury, Canada.

Karanam, U.M.R., Pasyapu, S.K., 2005. Experimental and numerical investigations of stresses in a fully grouted rock bolts. Geotech. Geolog. Eng. 23, 297–308.

Kilic, A., Ysar, E., Atis, C., 2003. Effect of bar shape on the pullout capability of fully-grouted rockbolts. Tunnel. Underground Space Technol. 18, 1–6.

Labiouse, V., 1996. Ground response curves for rock excavations supported by ungrouted tensioned rockbolts. Rock Mech. Rock Eng. 29 (2), 19–38.

Laubscher, D.H., 1977. Geomechanics classification of jointed rock masses—mining applications. Trans. Inst. Min. Metall. 86, A–1–A-7.

Li, C., Stillborg, B., 1999. Analytical models for rock bolts. Int. J. Rock Mech. Min. Sci. 36, 1013–1029.

Lunardi, P., 2000. The design and construction of tunnels using the approach based on the analysis of controlled deformation in rocks and soils. TT Int. (special supplement.).

Marinos, V., Marinos, P., Hoek, E., 2005. The geological strength index: applications and limitations. Bull. Eng. Geol. Environ. 64 (1), 55–65.

Mark, C., 2000. Design of roof bolt system. Proc. National Institute for Occupational Safety and Health (NIOSH).

Mark, C., Dolinar, D., Mucho, T.P., 2000. Summary of field measurements of roof bolt performance. Proc. National Institute for Occupational Safety and Health (NIOSH).

Oreste, P.P., 2003. Analysis of structural interaction in tunnels using the covergence–confinement approach. Tunnel. Underground Space Technol. 18, 347–363.

Oreste, P.P., Peila, D., 1996. Radial passive rockbolting in tunnelling design with a new convergence–confinement model. Int. J. Rock Mech. Min. Sci. Geomech. Abstr. 33, 443–454.

Osgoui, R. 2007. Development of an elasto-plastic analytical model for design of grouted rock bolts in tunnels with particular reference to poor rock masses. Ph. D thesis. Middle East Technical University. Ankara. Turkey. 250 pp.

Osgoui, R., 2006. On the Assessment of the effect of the anisotropy in in-situ stress on support pressure in tunnels. Proc. Int. Symp. In-Situ Rock Stress, Trondheim, Norway, pp. 307–317.

Osgoui, R., Ünal, E., 2005. Characterization of weak rock masses using GSI and the estimation of support pressure. 40th U.S. Rock Mechanics Symposium, Anchorage, Alaska.

Osgoui, R., Oreste, P.P., 2007. Convergence–control approach for rock tunnels reinforced by grouted bolts, using the homogenization concept. Geotech. Geolog. Eng. 25, 431–440.

Özkan, İ. 1995. Modified rock mass rating (M-RMR) system and roof behavior model. Ph. D thesis. Middle East Technical University. Ankara. Turkey. 370 pp.

Palmström, A., 1996. Characterizing rock masses by the RMi for use in practical rock engineering, Part 1: the development of the rock mass index (RMi). Tunnel. Underground Space Technol. 11, 175–188.

Palmström, A., 2000. Recent developments in rock support estimates by the RMi. J. Rock Mech. Tunn. Technol. 6 (1), 1–19.

Palmström, A., Broch, E., 2006. Use and misuse of rock mass classification systems with particular reference to the Q-system. Tunnel. Underground Space Technol. 21, 575–593.

Panet, M., 1993. Understanding deformations in tunnels. In: Hudson, J.A. (Ed.), In comprehensive rock engineering, vol. 1. Pergamon, Oxford, pp. 663–690.

Pells, P.J., Bertuzzi, R., 2008. Discussion on article titled "Use and misuse of rock mass classification systems with particular reference to the Q-system" by Palmström and Broch (Tunnel. Underground Space Technol; 21: 575–593) Tunnel. Underground Space Technol. 23, 340–350.

Serafim, J.L., Pereira, J.P., 1983. Considerations of the geomechanics classification of Bieniawski. Proc. Int. on Engng Geol. And Underground Constr. LNEC. Lisbon. Portugal.

Signer, S.D., 2000. Load behaviour of grouted bolts in sedimentary rock. Proc. National Institute for Occupational Safety and Health (NIOSH).

Singh, B., Jethwa, J.L., Dube, A.K., Singh, B., 1992. Correlation between observed support pressure and rock mass quality. Tunnel. Underground Space Technol. 7, 59–74.

Singh, B., Goel, R.K., Jethwa, J.L., Dube, A.K., 1997. Support pressure assessment in arched underground openings through poor rock masses. Eng. Geol. 48, 59–81.

Sönmez, H., Ulusay, R., 1999. Modifications to the geological strength index (GSI) and their applicability to stability of slopes. Int. J. Rock Mech. Min. Sci. 36, 743–760.

Sönmez, H., Ulusay, R., 2002. A discussion on the Hoek–Brown failure criterion and suggested modification to the criterion verified by slope stability case studies. Yerbilimleri (Earth Sciences) 26, 77–99.

Stille, H., Holmberg, M., Nord, G., 1989. Support of weak rock with grouted bolts and shotcrete. Int. J. Rock Mech. Min. Sci. Geomech. Abstr. 26 (1), 99–113.

Swoboda, G., Marence, M., 1991. FEM modelling of rockbolts. Proc., Comp. Meth. and Adv.in Geomech., Cairns, Australia, pp. 1515–1520.

Ulusay, R., Özkan, İ., Ünal, E., 1995. Characterization of weak, stratified and clay bearing rock masses for engineering applications. In: Mayer, L.R., Cook, N.W., Goodman, R.E., Tsang, C.F. (Eds.), Fractured and Jointed Rock Masses Conference. A.A. Balkema, Lake Tahoe, California, pp. 233–240.

Ünal, E. 1983. Development of design guidelines and roof control standards for coal mine roofs, Ph.D. Thesis. The Pennsylvania State University. Department of Mineral Engineering. State College. 355 pp.
Ünal, E., 1992. Rock reinforcement design and its application in mining. In: Kaiser, P.K., McCreath, D.R. (Eds.), Proc. Int. Symposium on Rock Support, pp. 541–546. Sudbury, Canada.
Ünal, E., 1996. Modified rock mass classification: M-RMR system. In: Bieniawski, Z.T. (Ed.), Milestone in Rock Engineering: A Jubilee Collection. InBalkema, pp. 203–223.
Ward, W.H., Tedd, P., Berry, N.S.M., 1983. The Kielder experimental tunnel: final results. Géotechnique 33, 275–291.
Wong, H., Subrin, D., Dias, D., 2006. Convergence–confinement analysis of a bolt-supported tunnel using the homogenization method. Can. Geotech. J. 43, 462–483.