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On the Assessment of the Effect of the Anisotropy in *in-situ* Stress on Support Pressure in Tunnels

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ABSTRACT: This paper presents a newly empirical approach to estimate support pressure in rock tunnels. The new approach takes the rock mass quality (Geological Strength Index: *GSI*), the disturbance degree, the post-failure behavior, the size of tunnel, the state of in-situ stresses, and the squeezing-prone condition into consideration. A parametric study was, too, carried out using numerical analysis to investigate the influence of anisotropy in field stresses and the effect of the various shapes and sizes of tunnels on failure-height and support pressure in a variety quality of rock masses. Therefore, a correction factor for stress ratio (horizontal to vertical stress ratio: *k*) is recommended to adjust the support pressure. For this purpose, the rock masses with various ranges of quality were considered to stimulate the very poor, fair, and good quality rock masses whereas arch-shaped and rectangular tunnels were examined in an anisotropic state of field-stress.

1 INTRODUCTION

Reliable prediction of tunnel support pressure (rock load) is a difficult task in the area of rock engineering and has been highly subjective to argument. Starting with Terzaghi's rock load concept (Terzaghi 1946), several empirical approaches using rock mass classification systems (empirical design approaches) have been developed to, either explicitly or implicitly, estimate support pressure in tunnels (Protodyakonov 1963, Deer 1963, 1968, Wickham et al. 1972, Bieniawski 1984, 1989, Barton 1974, 2002, Unal 1983, 1992, 1996, Venkateswarlu 1986, Ghose & Ghosh 1992, Verman 1993, Singh et al. 1992, Singh 1995, Palmström 1995, 1996, 2000, Goel 1996, Bhasin & Grimstad 1996). Most of these approaches classify tunneling conditions into several distinctly different groups and correlate these groups with stable support capacities. However, there have been found in literature some analytical approaches based on elasto-plastic closed-form solutions for support pressure estimation (Talobre 1957, Kastner 1962, Rabcewicz 1964, 1965. Daemen 1975. Hoek & Brown 1980, Brown et al. 1983, Sheory 1985, Carranza-Torres 2004). Only a few efforts based on partially numerical studies (Voegele & Fairhurst 1982) and physical modelings (Whittaker et al. 1992) have, up to date, been made in estimating support pressure.

Although a good many approaches have been developed to estimate support pressure (rock load), three influential parameters on support pressure; namely, the effect of opening size, the effect of the overstressed rock (squeezing ground condition especially in weak rock mass), and the effect of anisotropy in field stress have not been, due to the lack of the numerical studies, comprehended. Nonetheless, those empirical design approaches based on rock mass classification have been realized to be more helpful in the early stage of design procedure.

In this study, an empirical approach (rock mass classification) in tandem with the numerical methods presents a comparable expression in such a way as to take all notably geomechanical parameters into consideration.

This paper mainly presents the findings of research carried out pertaining to the influence of the in-situ stress field on the extension of the broken zone surrounding the tunnels. For this purpose, numerical modeling on the basis of the Finite Element Method (FEM) and Finite Difference Method (FDM) has been used. Different tunnel shapes subjected to particular ratios of horizontal to vertical components of in-situ stress in a variety of rock mass quality have been also examined.

2 ROCK-LOAD HEIGHT AND SUPPORT PRESSURE

2.1 *The concept of rock-load height*

This concept was primarily suggested during a comprehensive study of roof strata in US coal mines by Unal (1983, 1992, and 1996). The theory predicts the load on the support system purely based on the rock mass quality (Bieniawski's Geomechanics Classification "RMR") and tunnel span. Unal's rock-load height concept states that above any underground opening excavated, a roof arch and a ground arch are formed. The existence of these two arches can be identified by examining the stress distribution in the roof strata. The support must withstand the weight of the roof arch and the portion of the ground arch load actively transferred on the roof arch. The major portion of the strata pressure (passive load), on the other hand, is transferred to the sides of the opening due to the existence of the roof arch preserved by the support system. Hence, the total load that should be carried by support system is limited by the rock-load height, which is defined as the height of the potential instability zone, above the roof line and crown for rectangular, arch-roof, and/or horse shoe openings, which will eventually fall if not properly supported.

Given Unal's rock-load height concept, the new proposed empirical function for support pressure estimation can be dependent on the parameters specified in Equation (1):

$$P \approx f(h_t\ ,\gamma, C, S) \qquad (1)$$

The rock-load height, on the other hand, can be expressed as shown in Equation (2):

$$h_t = \frac{100 - \left[\left(1 - \dfrac{D}{2}\right)\sqrt{\dfrac{\sigma_{cr}}{100}}GSI\right]}{100}CSB \qquad (2)$$

where *GSI* = Geological Strength Index, which defines the quality of the rock mass; *D* = disturbance factor that controls the effect of the excavating methods (drill and blast or TBM) on damage around the tunnel; σ_{cr} = uniaxial compressive strength of intact rock for the broken zone around the tunnel; *B* = the span of the tunnel; γ = the unit weight of overburden; *C* = the correction factor for horizontal to vertical field stress ratio (*k*); and *S* = correction factor for squeeze and non-squeeze ground condition.

2.2 *Support pressure estimation*

Few empirical approaches for estimating support pressure have been found to contain more dominant geomechanical parameters (Osgoui, in prep). Most have limitations in their usage .Having realized the inadequacies of existing approaches, an attempt has been made to develop a more comparative approach to estimate the support pressure for tunnels (Osgoui & Unal 2005b).

Substituting Equation 1 into 2, the proposed empirical function is purely defined as:

$$P \approx f(GSI, D, \sigma_{cr}, B, \gamma, C, S) \qquad (3)$$

As indicated by the foregoing support pressure function, nearly all influentially geomechanical parameters are taken into consideration. Similar to its previous counterpart developed by Unal (1983, 1992, 1996), the newly proposed approach has as its main advantage the fact that the quality of rock mass is considered as the *GSI-Index*. Due to its accepted applicability in a broad range of rock mass quality, the *GSI-Index* was chosen to signify the rock mass quality.

It makes it possible to estimate the support pressure for tunnels in various rock mass qualities provided that the *GSI-Index* is determined. Encountered with very poor or poor rock mass where the *GSI* < 27, the *Modified- GSI* has to be used for support pressure estimation (Osgoui & Unal 2005b). It is therefore suggested that the new empirical approach be applied to a wide spectrum of rock mass, the quality ranging from very good to very poor.

The new empirical equation, which was proposed based on geomechanical parameters, is shown in Equation (4).

$$P = \frac{100 - \left[(1 - \dfrac{D}{2})\sqrt{\dfrac{\sigma_{cr}}{100}}GSI\right]}{100}CS\gamma B \qquad (4)$$

where $\sigma_{cr} = s.\sigma_{ci} \quad 0<s<1$

s= post-peak strength reduction factor, characterizing the brittleness of the rock material as discussed later on.

The most common form of the expression can be written when s=*1* as shown in Equation (5):

$$P = \frac{100 - \left[(1 - \dfrac{D}{2})\sqrt{\dfrac{\sigma_{ci}}{100}}GSI\right]}{100}CS\gamma B = \gamma h_t \qquad (5)$$

The rock load per unit length of tunnel can also be expressed as shown in Equation (6).

$$P = \frac{100 - \left[(1 - \dfrac{D}{2})\sqrt{\dfrac{\sigma_{ci}}{100}}GSI\right]}{100}CS\gamma B^2 \qquad (6)$$

2.3 *Parameters used in calculating support pressure*

2.3.1 *The effect of the disturbance factor "D"*

The method of construction has a significant influence on support pressure. Conventional excavation methods (drilling and blasting) cause damage to the rock mass whereas controlled blasting and machine tunneling (TBM) keep the rock mass undisturbed. Singh et al. (1992, 1997) declared that support pressure could be decreased up to 20% for such cases. In the newly proposed empirical approach, this effective parameter was adopted and modified from that pointed out by Hoek et al. (2002).

In tunnels, the effects of heavy blast damage as well as stress relief (relaxation) as a result of the ground being unloaded cause a disturbance in the rock mass being defined by disturbance factor "*D*". 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 such as driving tunnels or large caverns that have been subjected to very heavy blasting. The factor also allows for the disruption of the interlocking of the individual rock pieces within rock masses as a result of the discontinuity pattern (Marinos et al. 2005).

The incorporation of the disturbance factor "*D*" into the empirical equations is based on back-analysis of excavated tunnels. At this stage there is relatively little experience in the use of this factor, and it may be necessary to adjust its corporation in the equations as more field evidence is accumulated. However, the limited experience that is available suggests that this factor does provide a reasonable estimate of the influence of damage due to stress relaxation or blasting of excavated rock faces. In should be noted that the damage decreases with depth into the rock mass and, in numerical modeling, it is generally appropriate to simulate this decrease by dividing the rock mass into a number of zones with decreasing values of "*D*" being applied to successive zones as the distance from the face increases. In one example, which involved the construction of a large underground powerhouse cavern in inter-bedded sandstones and siltstones, it was found that the blast-damaged zone was surrounding each excavation perimeter to a depth of about 2 m (Cheng & Liu 1990).

Results indicate that for the same properties of rock mass and opening, support pressure increases as the disturbance factor decreases. For example, in a rock mass having a *GSI* of 50, the support pressure being imposed on the tunnel can be as much as 40 % if a blasting operation is carried out very poorly ($D=1$).

A guideline for choosing the disturbance factor is given in Table 1. They can be used to provide a realistic starting point for any design and, if the observed or measured performance of the excavation turns out to be better than predicted, the disturbance factor can be reduced.

Table 1. Suggested value for Disturbance factor " *D*"

Description of rock mass	Suggested value of *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	$D=1$

2.3.2 *The effect of intact rock strength*

Since the broken zone extension around an underground opening is dependent upon the strength parameters of the rock, it is suggested that the compressive strength of rock material which is an influential parameter in estimating the radius (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 calculations in accordance with the post-failure behavior of the rock. On the other side, it is substantiated that the extension of the broken zone relies on the residual value of the intact rock strength (Hoek & Brown 1980, Brown et al. 1983, Indraratna & Kaiser 1990 Carranza-Torres 2004, Osgoui, in prep). Hence, the effect of the compressive strength of 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 must, therefore, be considered as:

$$\sigma_{cr} = s.\sigma_{ci} \tag{7}$$

where *s* refers to the strength loss parameter quantifying the jump in strength from the intact condition to residual condition. The parameter *s* characterizes the brittleness of the rock material: ductile, softening, and brittle. By definition, *s* will fall within the range $0<s<1$. Where $s= 1$ implies no loss of strength and the rock material is ductile, or perfectly plastic. By contrast, if $s=0$, the rock is brittle (elasto-brittle plastic) with the minimum possible value for the residual strength (i.e. $\sigma_1=\sigma_3$).

2.3.3 *Correction factor for squeezing ground condition "S"*

There has been a recent interest in tunnels that have undergone large deformation. The cause of great deformation of tunnels is acknowledged to be due to

Table 2. Suggested values for squeezing ground condition factor "*S*"

Strains % (Tunnel closure or convergence/ tunnel diameter)*100	***Rock mass strength / In-situ stress(σcm / Po)***	***Remarks***	***Suggested correction factor "S" for squeezing ground condition***
Less than 1% no squeezing	> 0.5	Few stability problems and very simple tunnel support design methods can be used. Tunnel support recommendations based upon rock mass classifications provide an adequate basis for design.	1
1- 2.5 minor squeezing	0.3-0.5	Convergence confinement methods are used to predict the formation of a plastic zone in the rock mass surrounding a tunnel and of the interaction between the progressive development of this zone and different types of support.	1.5
2.5 -5 severe squeezing	0.2-0.3	Two- dimensional finite element analysis, incorporating support elements and excavation sequence, is normally used for this type of problem. Face stability is generally not a major problem.	0.8
5- 10.0 very severe squeezing	0.15-0.2	The design of the tunnel is dominated by face stability issues and , while two-dimensional finite analysis are generally carried out, some estimates of the effects of forepolling and face reinforcement are required.	1.6
More than 10 extreme squeezing	< 0.15	Severe face instability as well as squeezing of the tunnel make this an extremely difficult three-dimensional problem for which no effective design methods are currently available. Most solutions are based on experience.	1.8

the yielding of intact rock under a redistribution state of stress following excavation which exceeds the rock's strength. If this deformation takes place gradually it is termed as squeezing (Aydan et al. 1993, 1996).

Squeezing of rock is a time-dependent large deformation which occurs around the tunnel and is essentially associated with creep caused by exceeding the shear stress limit. Deformation may cease during construction or continue over a long period of time (ISRM 1981). Squeezing can occur in both rock and soil as long as the particular combination of induced stress and material properties pushes some zones around the tunnel beyond the shear stress limit at which creep starts. The magnitude of tunnel convergence associated with squeezing, the rate of deformation, and the extent of the yielding zone around the tunnel depend on the geological conditions, the in-situ stresses relative to rock mass strength, the ground water flow and pore pressure and the rock mass properties (Barla 1995).

Owing to the fact that almost all tunneling operations in weak rock mass withstand squeezing ground conditions, it is of paramount importance to take this effect into consideration in precisely estimating the support pressure (Osgoui & Unal 2005a, b).

The squeezing degree has been expressed in terms of normalized tunnel convergence or closure (Singh et al. 1992, 1997), normalized convergence ratio (Indraratna & Kaiser 1990), competency factor or strength factor (Bhasin & Grimstad 1996, Hoek & Marinos 2000), and critical strain concept (Hoek & Marinos 2000). Since the tunnel convergence is an important indicator of tunnel stability, the squeezing behavior has been evaluated in terms of tunnel convergence in the current study.

The guideline for squeezing correction factor presented herein was adopted from the results of many case-histories throughout the whole world (Singh et al. 1992, Goel 1994, Barla 1995, Goel et al. 1996, Aydan et al. 1996, Bhasin & Grimstad 1996, Singh et al.1997, Hoek & Marinos 2000) as given in Table 2.

2.3.4 *Correction factor for anisotropy in field stress"k"*

Detailed information about this factor will be given in section 4.

3 NUMERICAL STUDIES FOR ESTIMATION OF ROCK-LOAD HEIGHT AND SUPPORT PRESSURE

When an opening is being excavated, the excavation removes the boundary stress around the circumference of the opening, and the process may be simulated by gradually reducing the internal support pressure. As the support pressure reduced, a plastic zone is formed when the material is overstressed. This region of the rock mass in the plastic state is called the plastic zone (broken zone, disturbed zone, yielding zone, and overstressed zone) which may propagate in the course of tunnel excavation. The configuration of the plastic zone around a tunnel may depend on a number of factors, such as the anisotropy in initial stress state, the tunnel's shape, and the rock mass properties and so on.

For circular openings, an ealasto-plastic closed-form solution makes it possible to determine the radius of the plastic zone or radius of elastic-plastic interface (radius of internal elastic zone) around the tunnel when the internal support pressure is lower than critical pressure. In this case, the assumption of the isotropy in field stress, homogeneity in rock mass, and axi-symmetrical plane strain condition must be taken into account. Several consistent closed-form approaches have been developed over the past 30 years as addressed by Osgoui (in prep).

Numerical methods are, on the other hand, capable of modeling and analyzing the non-circular tunnels in an anisotropic field of stress. Provided that the input properties are sufficiently realistic, an elasto-plastic finite element or finite difference analysis of broken rock may perhaps lead to estimation of a reliable failure height.

Accordingly, in order to determine and to evaluate the extent of the failure zones developing around non-circular openings due to pressure release, a Finite Difference Method "FLAC" (Itasca 2000) and a Finite Element Analysis (FEA) program "PHASE^2" (Rocscience 2005) have been utilized in this study. In addition, the effects of the dominant parameters (i.e. shape and size of tunnel, rock mass quality, and anisotropy in field stress) to the extent of failure height have been examined.

The most significant objective of the numerical analysis was to determine the stress correction factor used for proposed empirical expression. Let the rock-load height of proposed expression be called h_t and let the failure height of numerical analysis be h_f Seeing that the effect of the stress ratio is taken into account in the numerical method, the ratio of h_f /h_t gives a ratio called as the stress correction factor *C*, whose value can be then multiplied in empirical formula to correct the stress effect.

In summary, the primary purposes of the numerical analysis carried out in this study are as follows:

i. To determine the extent of the failure zone (failure height) around arch-shaped and rectangular openings
ii. To investigate the effect of rock mass quality *GSI*, tunnel width *B*, and anisotropy in field stress on failure zone
iii. To compare rock-load height (h_t), calculated by proposed empirical approach with failure height (h_f) determined by the numerical studies.
iv. To find the correction factor for horizontal to vertical stress ratio *k*.

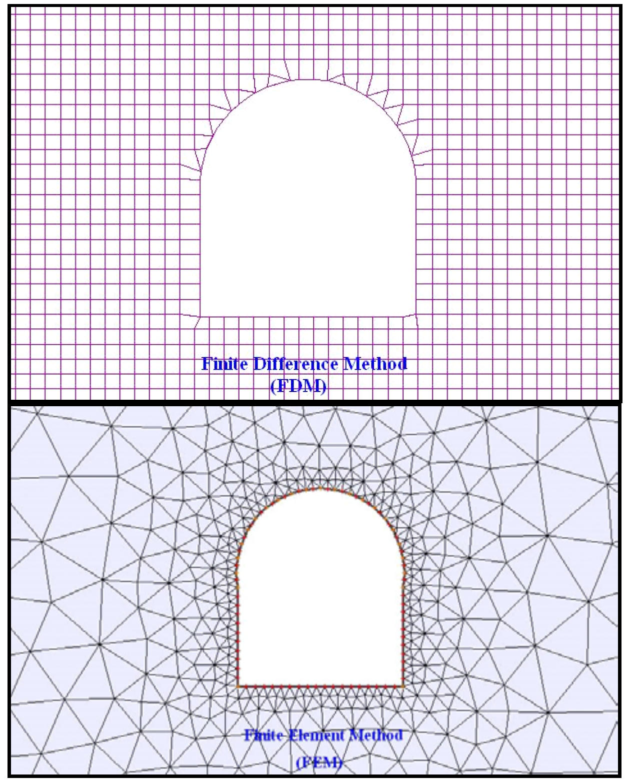

Figure 1. The numerical layout of the arch-shaped tunnel by Finite Difference Method (FDM) and Finite Element Method (FDM) in an infinite Hoek &Brown medium.

3.1 *Numerical models configurations*

The rock mass around the tunnel was considered to be isotropic and homogeneous without any remarkable discontinuity system. The infinite medium condition was required to better simulate the model. Moreover, an elasto-plastic 2-D plane strain condition with a constant far field stress of 10MPa was applied.

To simulate the rock quality three sets of rock mass quality representing the poor, fair and good condition have been adopted using *GSI- index*. (i.e. *GSI= 20, 45, and 85*)

The arch-shaped and rectangular tunnels having widths of 5m, 10m, and 15m have been imposed under an anisotropic field stress with ratio of 0.3, 0.5, 1, 1.5, and 2.5.

A typical layout of an arch-shaped tunnel modeled by Finite Difference Method (FDM) and Finite Element Method (FEM) are shown in the Figure 1 and a summary of the fixed and variable input parameters used in the current study is also presented in Table 3.

In order to determine the effects of the variable parameters on the failure heights, a total of 180 *FLAC* and *PHASE* runs were performed and analyzed.

Table 3. Fixed and variable parameters used in numerical studies.

Fixed parameters			
Rock mass: isotropic, homogeneous, infinite medium condition		2D plane strain condition	
Type of analysis: Elaso-plastic		Failure Criterion: Hoek & Brown 2002	
Field stress: constant 10 MPa		unit weight of rock mass: 0.025 MN/m3	
Vriable Parameters			
Shape of opening: Arch-shaped (horse-shoe) tunnel, Rectangular tunnel			
Geological Sterngth Index (GSI)	**Span (m)**	**Stress ratio k (σh/σv)**	**Rock mass properties**
Poor quality rock mass 20		0,3	σci : 10 MPa , mi=10 , D=0 , mb=0,574 s=0.0001, a=0.544 , σcm = 0,812 MPa, Em=562,34 Mpa, ν = 0,27
	5	0.5	
	10	1	
	15	1,5	
		2,5	
Fair quality rock mass 45		0,3	σci : 50 MPa , mi=12 , D=0 , mb=1,683, s=0.0022, a=0.508 , σcm = 8,536 MPa, Em=5302,55 Mpa, ν= 0,25
	5	0,5	
	10	1	
	15	1,5	
		2,5	
Good quality rock mass 85		0,3	σci : 100 MPa , mi=16 , D=0 , mb=9,346, s=0.1889, a=0.5, σcm = 51,88 MPa, Em=74989,42 Mpa, ν=0,2
	5	0,5	
	10	1	
	15	1,5	
		2,5	

3.2 *Analysis of the results*

The results of the numerical analysis are briefly presented herein to investigate the effects of the variables parameters, *B*, *GSI*, *k*, and shape, on the extent of the failure zone (failure height) above the tunnel and to obtain a correction factor for stress ratio *k*, which is used in empirical approach.

3.2.1 *Stress ratio "k"*

For both arch-shaped and rectangular tunnels, with a further increase in k, apart from the tension failure mode, the profusion of the shear failure mode increases. Numerical analysis of broken zone around the tunnel implied that the extension of failure height above tunnels is predominantly dependent upon the magnitude of the stress ratio *k*. For both 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 as shown in the Figure 2. Generally speaking, for the same tunnels excavated through the similar quality rock masses, the failure height of those tunnels driving under the condition of the k > 1 would result in higher values. An example of this founding for a arched-shaped tunnel driving within a poor quality rock mass (*GSI*=20) is presented (see Fig. 3).

Once the stress ratio *k* reaches to 2.5, both arch-shaped and rectangular profiles exhibit the formation of distinct broken zones largely in the sidewall. Shear failure plays a significant role in the formation of the broken zone with a wedge of failed material attempting to move laterally into the tunnel as also reported by Whittaker et al. (1992). The predominant fracture is that of lateral movement of the sidewalls into the tunnel which particularly generates floor instability.

3.2.2 *Good quality rock mass*

For different values of stress ratio *"k"* and tunnel size, the arch-shaped tunnel excavated in good quality rock mass (*GSI*=85) is self-supported. However,

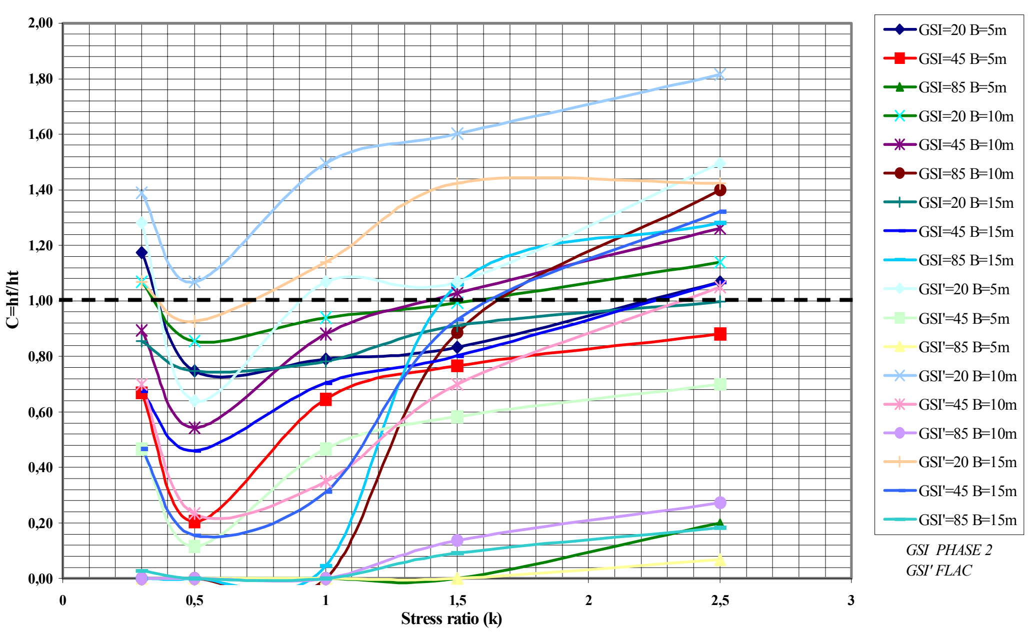

Figure 2. Representation of failure height to rock-load height ratio (hf/ht) or stress correction factor as a function of anisotropy in in-situ stress ratio (k) for various rock mass quality (*GSI*) and spans (5m, 10m, 15m) for arch-shaped tunnels

in the case of highly horizontal stress (k>1.5) the roof is potentially unstable to a greater degree. In contrast, a rectangular tunnel, even if excavated in good quality rock mass, might withstand some problems. The reason behind these observations is that for an arched tunnel, the formed arch decreases, to some extent, the effect of the stress being imposed on the tunnel crown, whereas in the case flat roof, the separation or the sag of the roof strata due to stress gives rise to an increase in the failure height. Simply put, the normal forces will be greater in the case of a rectangular opening with flat roof by virtue of the weight of detached blocks of rock that are free to fall. In contrast, the detached blocks in the case of an arch-shaped tunnel become interlocked on displacement because of the dilatant behavior of rock masses.

3.2.3 *Tunnel size*

Numerical analysis puts forward a significant conclusion indicating that with increasing tunnel size, the failure height above the tunnel especially in poor rock masses (*GSI*=20) increases regardless of the tunnel shape. For two same-sized tunnels whose widths are to be enlarged, a gradual increase in failure height takes place in the good quality rock mass whereas a sudden rise in failure height occurs in the weak rock mass. To put it more simply, the effect of the tunnel size on support pressure in weak rock mass is far more obvious than that in fair to good rock mass.

In strong rock mass (*GSI*=85), it is evident that the failure height and consequent support pressure is independent of tunnel size. In other words, unlike the good rock masses, the support pressure is directly proportional to the size of the tunnel in the case of poor to fair rock masses undergoing squeezing ground condition. Unal (1983) explored this phenomenon in coal mine studies.

These observations are found to be in contradiction with the results advocating that the support pressure is independent of roof span (Barton 1974, Singh et al. 1992, 1997). It is heartening to say that the mentioned finding verifies the empirically proposed equation.

3.2.4 *Arch-shaped tunnel versus rectangular tunnel*

For poor rock mass with *GSI*=20, the results of the empirical approach for both arch-shaped and rectangular tunnels lie in between the PHASE results. Conversely, the FLAC results constitute the upper limit of the failure height envelopes. These phenomena are attributed to two reasons. The first would be due to the fact that no effect of the rock mass disturbance and squeezing ground condition are taken into account in the proposed approach. Considering the influences of the mentioned parameters in empirical proposed approach, the realistic and reliable results would, in turn, be obtained. The second one is that FLAC is far more potent than PHASE in modeling the poor rock masses as its usage ranges even for the soils.

In fair quality rock mass(*GSI*=45) for arch-shaped tunnels, the results of the empirical approach accounts for approximately the upper limit of the failure height envelopes while for rectangular tunnels empirical results remain between the numerical results as shown in the Figure 4. However, for the good quality rock mass where *GSI* =85, the proposed approach envelope stretches out between the envelopes of the numerical results. In this case the furthermost limits are made of from the failure height envelope of high horizontal stress (k=2.5).

In poor and fair rock masses (*GSI* varies between 20 and 45) withstanding squeezing ground condition, the failure height of rectangular tunnels is more than that of the arch-shaped tunnels with the same width.

It is, therefore, evident that there is a good agreement between empirically calculated rock-load height and numerically calculated failure height. Hence, empirical approach can be safely used no matter how quality of the rock mass is.

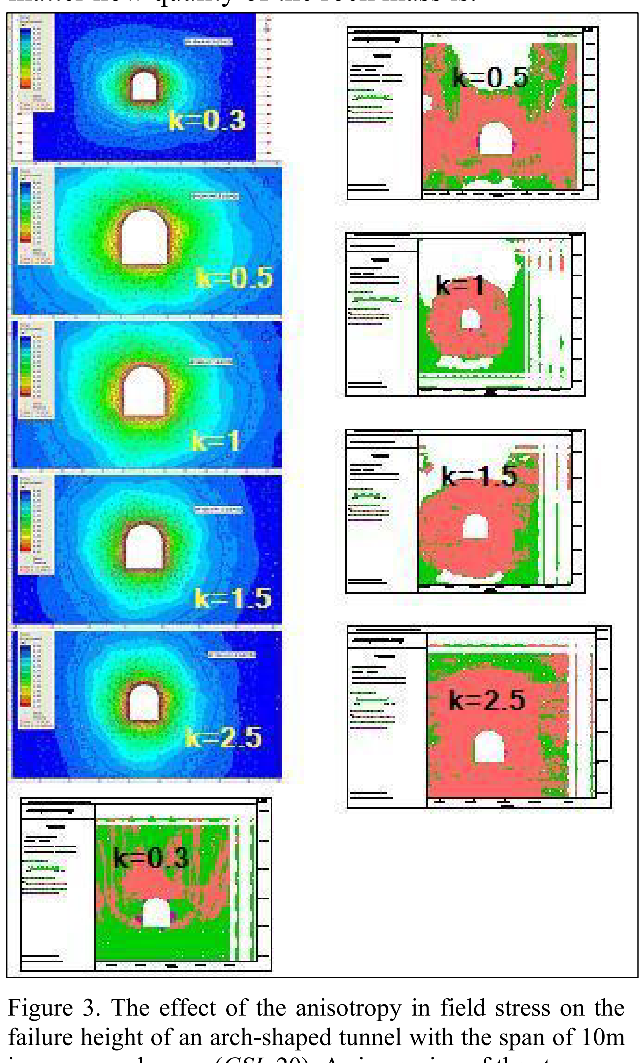

Figure 3. The effect of the anisotropy in field stress on the failure height of an arch-shaped tunnel with the span of 10m in a poor rock mass (*GSI*=20). As increasing of the stress ratio “k” toward 2.5 the failure height is enlarged.

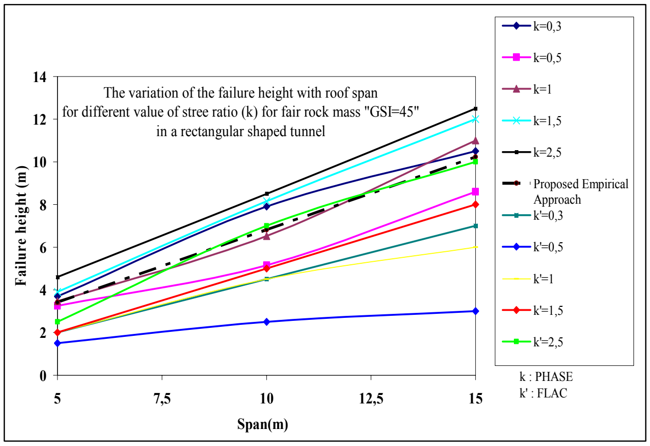

Figure 4. The variation of the failure height with roof span for different value of stress ratio (k) for fair quality rock mass "*GSI=45*" in a rectangular shaped tunnel

4 CORRECTION FACTOR FOR HORIZONTAL TO VERTICAL STRESS RATIO "*C*"

Numerical analysis of broken zone around the tunnel implied that the extension of failure heights above tunnels is basically dependent 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 as previously discussed.

The ratio of the failure height (obtained from numerical methods) to rock-load height (determined by the proposed formula) yields a value called the stress correction factor *C*. This correction value has to be applied while using Equations 2 and 4. However; findings indicated that for a wide variety of k values, the rock-load height form an upper limit to the data points obtained from analytical studies. In other words, the ratio of h_f to h_t in most cases is less than 1. Therefore, a multiplier "*C*" is required to correct the stress ratio. For reliability, the minimum *C* for

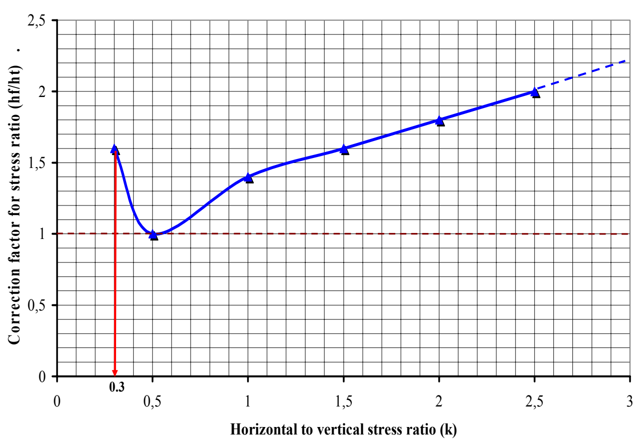

Figure 5. Suggested value for stress correction factor *"C"* used in proposed formula

the proposed formula is always suggested as 1 for *k*=0.5. Figure 5 aims at choosing the stress correction factor.

The applicability of the proposed approach has just been confirmed in estimating support pressure and support capacity and in designing reinforcement system for a rail-road tunnel, excavated within a poor rock mass, in Turkey. More detailed information is addressed to elsewhere (Osgoui & Unal 2005a, b).

5 CONCLUSIONS

A compelling empirical approach to estimate the support pressure has been developed. Not only does the proposed approach take into account the quality and quantity of the rock mass, but it also takes into account the squeezing ground condition and anisotropy in field stress.

Evidence to validate the empirical predictions has been obtained by numerical modeling. Numerical studies have been carried out to study the effects of the anisotropy in field stress in order that a correction factor for stress ratio "*k*" should be included in the proposed empirical expression.

Other influential factors namely tunnel shape and tunnel size affecting the extent of failure height have been numerically investigated to validate the empirically proposed approach.

The numerically calculated failure heights have been found to be in reasonable agreement with those obtained by the empirical approach.

A distinguished conclusion that can be extracted is that the proposed approach can be used safely in the fair to good quality rock mass. More care, however, has to be taken where tunneling in poor rock masses is concerned. In such cases, the influence of the stress conditions found in the form of the squeezing ground condition, completely loss of intact strength, and high horizontal stress should be taken into consideration. The step-by-step procedure for determining the support pressure discussed in the paper might be a useful tool at the early stage of tunnel design.

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