USE OF ROCK MASS RATING (RMR) VALUES FOR SUPPORT DESIGNS OF TUNNELS EXCAVATED IN SOFT ROCKS WITHOUT SQUEEZING PROBLEM

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ABSTRACT

Effect of the rock material strength on the RMR value and tunnel support designs was investigated within this study including site works, analytical and numerical analyses. It was found that rock material strength effect is quite limited in the RMR method to determine an accurate rock mass class to design tunnel support. Since the limitation, rock mass classes are evaluated to be usually misleading and supports designed in accordance with the RMR value are insufficient for tunnels excavated in rock masses with low strength values of rock materials. Totally, five different tunnels in Turkey have been supported using a new strength adjustment factor calculated in consideration of the in-situ stress and the uniaxial compressive strength values of rock materials. As confirmed by the field applications, analytical and numerical analyses, a newly modified RMR value (RMR_{us}) was suggested to be used in tunnel support design works.

Keywords: Rock Support, Empirical Methods in Tunnelling, RMR, Rock Mass Strength, Finite Element Analyses

1 INTRODUCTION

Because of their practicality of use for the support design works, empirical methods are widely used in tunnelling. Among popular empirical methods, RMR and Q methods which have been used worldwide in tunnel constructions since 1970s have their own support design charts. Since the first announcements of RMR and Q methods, various modifications were suggested to improve their performance in tunnelling. There have been important revisions of these empirical methods [1-5]. In addition to RMR and Q methods, New Austrian Tunnelling Method (NATM) has also a support suggestion chart, but the absence of rate marking by numbering in the NATM makes it responsible for significant personal variations. For a reliable support design, only empirical methods are not sufficient. It is preferable in support design to verify empirical methods by results of other methods, such as numerical and analytical methods. As Karl Terzaghi stated, rock masses are made by nature not by man, the products of nature are always complex [6, 7]. The empirical methods do not have enough detailed parameters for being only used in tunnelling. Nevertheless, the use of empirical methods can be accepted to be a helpful part in the support design works.

In this study, a modification of rock material strength effect in the RMR (Rock Mass Rating) value determination has been investigated. The well-known RMR determination details are given in Tables 1-6. As seen in Table 1, the uniaxial compressive strength value of rock materials can only change RMR value by 15. To express the situation from another point of view, it can be noted that two different rock masses with varying rock material strength values of 3 MPa and 150 MPa can have a maximum RMR value difference of 15%. In other words, it is possible to have RMR value of 85 for a rock mass with a quite low rock material strength value of 3 MPa. According to the RMR support suggestion chart, there is no need to support that tunnel excavated in the rock mass with the rock material strength of 3 MPa. Considering a medium-depth like 100 meters, a thick failure zone is estimated to occur around the tunnel, due to the induced stresses. Therefore, it is explicit that rock material strength role on the RMR value should be more dominant for rock masses with low material strength values. In this study, we aim to suggest a rock material strength parameter scoring modification in the RMR value determination to improve its support design performance in weak rock conditions.

Underground instability problems can be classified into two main groups of structural and stress controlled instabilities. In structural instabilities, discontinuities in the rock masses and their frictional load bearing capacity are determinative for the rock mass strength. In situ stresses are generally not high and the failure around underground openings mostly starts due to the gravitational forces of the loosening rock mass volume. The structural instabilities occur as a result of reaching load bearing capacities of discontinuities, without failure in rock materials. On the other hand, failing in rock materials are widely seen in stress controlled instabilities resulting from high level of induced stresses around underground openings [8-12]. In some cases, hybrid failure mechanisms including both structural and stress controlled instabilities can be induced in rock masses.

Rock masses with no or few and minor joints have stress controlled failures, since the rock material strength is a more dominant parameter than strength of joints. This study aims to focus on the stress controlled instability problems in soft rocks with few and minor discontinuities. It is a common way to practically accept the use of continuum mechanics in modelling rock masses with no or few minor joints [13-15]. Use of the continuum rock mass models supplies a significant advantage for an effective support design ability, by estimating occurrence and borders of the plastic zone around tunnels [16-18].

Within this study, a modification for the support design based on the RMR value is suggested in accordance with the results obtained from various field works in five different tunnels in Turkey. We aimed to make the use of the RMR value more effective to design support for weak rock masses with practically no or few minor joints. To investigate whether the new modifications for the RMR are usable, the results obtained from this empirical study are compared with those obtained from numerical models and an analytical plastic zone estimation approach for continuum rock mass conditions.

For the support design chart, the rock bolt usability in very low strength rock masses was also investigated in this study. As seen in the RMR support chart given in Table 7, rock bolts are suggested to use in very poor rock masses with RMR values lower than 20. In case of having quite low rock material strength values, it is known that rock bolts cannot supply a sufficient anchorage and support pressure. Furthermore, the drilling processes in such poor rock masses make extra worsening and damage, which is able to trigger instabilities [19-21].

2 FIELD STUDY

Rock supports designed in accordance with the RMR support chart including the new updates for the rock material strength property have been applied in five different tunnels in Turkey. Information about the tunnels reported in this study can be found in Table 8. Cores taken from the working tunnels and mine galleries were used to evaluate RMR values of different rock formations. The RQD (Rock Quality Designation) value which is an important parameter for RMR determination was calculated with length measurements on the cores bored. In addition to the geotechnical drills, RMR values have been regularly determined at tunnel faces after each of the face advance process. Discontinuity properties (roughness, spacing, weathering, infilling, aperture, water condition, etc.) were carefully investigated to be rated. To determine the rock material strength values in the tunnels, rock blocks were brought to the laboratory, cored and cut by the sawing machine to have the length to diameter ratio of 2 to prepare specimens for the uniaxial compressive strength (UCS) test. Besides, specimens from the core boxes of the geotechnical drills were used in the UCS tests. Loading rate was chosen to be 0.5 MPa/sec in the uniaxial compressive strength (UCS) tests. In addition to the UCS test in the laboratory, the geological hammer was used to instantly check the strength intervals of the rock materials in tunnels. The use of geological hammer was an auxiliary method for the laboratory tests. Rock material strength intervals were determined using the geological hammer according to the explanations given in Table 9 and Table 10 [22, 23]. In Table 11, rock mass classes according to the RMR values are given as multiplied by blasting and weakness adjustment factors. To designate the adjustment factors, "controlled blasting" condition was selected. For some poor rock conditions, blasting adjustment factor was not used since the mechanical excavation was done. The RMR determination procedure is seen in Tables 1-6. The RMR value is the sum of ratings of parameters for rock material strength (R1), RQD (R2), Joint spacing (R3), Condition of joints (R4), Ground water condition (R5), orientation (R6). If there is no orientation in the rock mass, R6 value is taken as 0. Table 2 and Table 3 are given to guide the determination of R6. Another guide table, Table 4, is given for the condition of joints rating (R3). As seen in Eq. 1, last step for evaluation of the RMR_{89} value is multiplying the sum of ratings by adjustment factors (Fa, Fb) whose details are given in Table 5.

$$RMR_{89}=(R1+R2+R3+R4+R5+R6) \times F_a \times F_b \quad (1)$$

As the tunnels in this study were excavated in formations with low rock material strength values and mostly fair or good discontinuity properties, the data from them were used to make an update of rock material strength adjustment factor. As previously stated, in case of having a high rate for discontinuity properties, the rock material strength adjustment factor (SAF) is needed to be used to make the effect of low rock material strength values on the tunnel stability more dominant. As long as having highly jointed rock masses, discontinuity properties have the major role. Therefore, the rock material strength adjustment factor (SAF) was not suggested for highly jointed rock masses. To use the SAF, the total rate from discontinuity (RQD, spacing of discontinuities, condition of discontinuities) properties should be over 52 per 70. In case of supplying that condition, tunnel depth and in situ stresses are determinative for the SAF value. After using SAF, the RMR values and rock mass classes for the tunnels investigated in this study were changed as seen in Table 11. All the tunnels in this study were supported according to the RMR value updated multiplying by SAF (RMR_{us}) (Eqs. 2-4).

$$RMR_{us}=RMR_{89}.SAF \quad (2)$$

$$SAF= (\sigma_{ci} /\gamma z)/7 \qquad for\ \sigma_{ci} /\gamma z < 7 \quad (3)$$

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$$SAF = 1 \qquad \text{for } \sigma_{ci}/\gamma z \geq 7 \qquad (4)$$

where, σ_{ci} is the uniaxial compressive strength of rock materials (MPa), γ is the unit volume weight of rock masses (kN/m^3), z is the depth of tunnel (m). Because of ignoring the exact effect of low strength values of rock materials, insufficient support for tunnels can be designed in case of using the non-updated (standard) RMR_{89} value. For instance, rock bolting with a span of 2.5 meters and a thin shotcrete liner with 5-7 cm thickness in the galleries of Murgul Kabaca derivation tunnel which has the rock material uniaxial compressive strength (UCS) of 11.1 MPa were assessed to be sufficient according to non-updated RMR_{89} value of 67. When considering the update of rock material strength effect and the use of SAF, RMR value decreases to 55. According to the updated RMR_{us} value of 55, rock bolts with 1.5-meter span and the shotcrete liner with a thickness of 12 cm were applied in the relevant part of the tunnel.

Table 1. Classification parameters and values

PARAMETERS		VALUES						
R1	Point Load Index	>10MPa	10-4 MPa	4-2 MPa	2-1 MPa	Non-applicable	Non-applicable	Non-applicable
	Uniaxial Compressive Strength (σ_{ci})	>250 MPa	250-100	100-50	50-25	25-5	5-1	<1
	RATING	**15**	**12**	**7**	**4**	**2**	**1**	**0**
R2	Rock Quality Designation RQD	% 100-90	90-75	75-50	50-25	<25	<25	<25
	RATING	**20**	**17**	**13**	**8**	**3**	**3**	**3**
R3	Joint spacing (cm)	>200	200-60	60-20	20-6	<6	<6	<6
	RATING	**20**	**15**	**10**	**8**	**5**	**5**	**5**
R4	Condition of joints	Very rough and unweathered, Wall rock tight and discontinuous	Rough and slightly weathered, wall rock surface separation <l mm	Slightly rough and moderately to highly weathered, wall rock surface separation <l mm	Slick sided wall rock surface, 1-5 mm	Soft gouge, > 5mm	Continuous discontinuity	
	RATING	**30**	**25**	**20**	**10**	**0**	**0**	**0**
R5	Ground water condition	Completely dry	Damp	Wet	Dripping	Flowing		
	RATING	**15**	**10**	**7**	**4**	**0**		

Table 2. Influence of orientation on assessment

Joint orientation assessment for tunnels	Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Value	0	-2	-5	-10	-12

Table 3. Joint dip and strike effect in tunnelling, guide for Table 2

Strike perpendicular to tunnel axis				Strike parallel to tunnel axis		Irrespective of strike
Drive with dip		Drive against dip				
Dip 45°-90°	Dip 20°-45°	Dip 45°- 90°	Dip 20°-45°	Dip 45°-90°	Dip 20°-45°	Dip 0°-20°
Very favourable	Favourable	Fair	Unfavourable	Very unfavourable	Fair	Fair

Table 4. Guide for evaluation of condition of joints (R4)

Parameter	Value				
Joint length (continuity)	<1m (6)	1-3m (4)	3-10m (2)	10-20 m (1)	>20m
Separation	No (6)	<0.1mm (5)	0.1-1.0 mm (4)	1-5 mm (1)	>5mm (0)
Roughness	Very rough (6)	Rough (5)	Slightly rough (3)	Straight (1)	Slippery (0)

Fill		Hard Fill		Soft Fill	
	No fill (6)	<5mm (4)	>5mm (2)	<5 mm (2)	> 5mm (0)
Weathering	No weathering (6)	Low weathering (5)	Mid-level weathering (3)	High Weathering (1)	Very high weathering (0)

Table 5. Adjustment factor values (Fa and Fb)

Method/ Situation	**Applicable Term**	**Adjustment Factor (F_a)**
1. Mechanical Excavation	Without damage	1.0
2. Controlled blasting	Low damage	0.94-0.97
3. Good blasting	Medium damage	0.90-0.94
4. Poor blasting	High damage	0.90-0.80
5. No prior information about blasting	Medium damage	0.90
Adjustment for weakness planes		
Situation		Adjustment Factor (F_b)
No plane of weakness		1.0
Hard dykes		0.90
Soft ore zones		0.85
Rock and ore contact zones or inhomogeneous roof rock		0.80
Folds, Synclinals/ Anticlinals		0.75
Fault zones		0.70

Table 6. Rock mass class and RMR values

RMR	Rock Mass Class
81-100	Class 1 (very good rock)
61-80	Class 2 (good rock)
41-60	Class 3 (fair rock)
21-40	Class 4 (poor rock)
0-20	Class 5 (very poor rock)

Table 7. Support Suggestions according to RMR values

RMR value	**Excavation**	**Support**		
		Rock bolts, fully grouted, 20 mm diameter	**Shotcrete**	**Steel sets**
81-100	Full face, 3 m advance	Generally no support required except spot bolting		
61-80	Full face, 1.5-3m advance, complete rock bolting 20 m from face	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh or FRS (fibre reinforced shotcrete)	5-7 cm FRS (fibre reinforced shotcrete) where required	None
41-60	Top heading and bench, 1.5-3 m advance in top heading, commence support after each blast, complete rock bolting 10 m from face	Systematic bolts 4 m long, spaced 1.5-2 m in crown and walls with wire mesh or FRS	5-10 cm in crown and 5 cm in sides as FRS	None
21-40	Top heading and bench, 1.0-1.5 m advance in top heading, install support concurrently with excavation, complete rock bolting 10 m from face	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh or FRS	10-15 cm in crown and 10 cm in sides as FRS	Light to medium ribs spaced 1.5 m where required
<20	Multiple drifts, 0.5-1.0 m advance in top heading, install support concurrently with excavation, shotcrete as soon as possible after blasting	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh or FRS, invert bolts	15-20 cm in crown, 15 cm in sides, and 5 cm on face	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required, close invert

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Table 8. Selected tunnels in this study

	Length (m)	Cross-section shape	Cross-section area (m^2)	Max. Depth (m)	Location (City, Country)
Cerattepe Mine Main Haulage Gallery-North	1087	Horseshoe	40	175	Artvin, Turkey
Cerattepe Mine Main Haulage Gallery-South	985	Horseshoe	40	190	Artvin, Turkey
Murgul Kabaca Derivation Tunnel	748	Horseshoe	45	70	Artvin, Turkey
Akarsen South Mineralization Approach Tunnel	450	Horseshoe	35	135	Artvin, Turkey
Kızık Roadway Tunnel	632	Horseshoe	45	60	Ankara, Turkey

Table 9. Estimation of uniaxial compressive strength values of rock materials using standard geological hammer according to BSI (1981)

Uniaxial compressive strength of rock materials	Explanation (Standard geological hammer use)
< 1. 25 MPa	Crumbles in hand
1.25 MPa – 5 MPa	Thin slabs break easily in hand
5 MPa – 12.5 MPa	Thin slabs break by heavy hand pressure
12.5 MPa - 50 MPa	Lumps broken by light hammer blows
50 MPa – 100 MPa	Lumps broken by heavy hammer blows
100 MPa – 200 MPa	Lumps only chip by heavy hammer blows
> 200 MPa	Rocks ring on hammer blows. Sparks fly.

Table 10. Estimation of uniaxial compressive strength values of rock materials using standard geological hammer according to ISRM (1978)

Uniaxial compressive strength of rock materials	Explanation (Standard geological hammer use)
< 1 MPa	Intended by thumbnail
1 MPa – 5 MPa	Crumbles under firm blows with point of geological hammer; can be peeled by a pocket knife
5 MPa – 25 MPa	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer.
12.5 MPa - 50 MPa	Cannot be scraped or peeled with a pocket knife; specimen can be fractured with single firm blow of geological hammer
50 MPa – 100 MPa	Specimen requires more than one blow of geological hammer to fracture it
100 MPa – 200 MPa	Specimen requires many blows of geological hammer to fracture it.
> 200 MPa	Specimen can only be chipped with geological hammer.

Table 11. Rock mass classes for tunnel locations with different RMR89 and RMR_{us} values

Tunnel/Gallery name	Length of parts (SAF<1)	For using RMR_{89}	For using RMR_{us}
Cerattepe Mine Main Haulage Gallery-North	127 m	Class 2	Class 3
Cerattepe Mine Main Haulage Gallery-South	105 m	Class 2	Class 3
Murgul Kabaca Derivation Tunnel	66 m	Class 2	Class 3
Akarsen S. Mineral. Approach Tunnel	113 m	Class 2	Class 4
Kızık Roadway Tunnel	52 m	Class 2	Class 3

As seen in Table 11, bolt span decreases from 2.5 m (2.5 m x 2.5 m) to 1 m (1.5 m x 1.5 m) because of the change in the rock mass classes from Class 2 to Class 3, due to the use of RMR_{us} value. In the case of having 1-meter plastic zone thickness, nearly 18 tons' load is estimated to apply on a rock bolt inserted with the span of

2.5 meters. In other words, load of 6.25 m3 rock volume are borne per one bolt in case of having the bolting span of 2.5 meters (2.5 m x 2.5 m). In many cases, load bearing capacities of typical grouted bolts are about 20 tons. In this regard, it is not safe to design the rock bolts to be loaded by 90% of its bearing capacity. Instead of using non-updated RMR_{89}, the load on a bolt can be decreased by 64% by designing the bolts to have the span of 1.5 meters, according to the RMR_{us} value.

On the other hand, it should be reminded that rock bolts cannot have a sufficient anchorage in the weak rock masses with quite low strength values. In the bargain, rock bolting can be disadvantageous as a result of damage in very poor rock masses with low slake durability. For instance, there was a collapse after drilling processes in Akarsen Mine, a case study area of this study. Therefore, rock bolts were not used in the Akarsen mine because of the damage of poor rock mass with the watery drilling operations. In the following parts of this paper, failure around tunnels in this study and needed support pressures to supply stability are analysed with analytical and numerical studies.

3 ANALYTICAL STUDY

In Eqs. 5-13, an approach for estimating plastic zone occurrence and thickness is given [24]. Eq. 5 and Eq. 6 are usable for higher k ratios than 1/3 and $2A_{pc}/(6A_{pc}+1)$, respectively. Additionally, Eq. 10 is suggested to be used for W_p calculations when the k ratio is smaller than $2A_{pc}/(6A_{pc}+1)$. To choose a suitable equation, A_{pc} can be practically considered as given in Eq. 12. This approach for plastic zone thickness estimation is suggested for isotropic, homogeneous and elastic rock masses. W_p is the distance between plastic zone boundaries in the direction of the horizontal diameter of tunnels and H_p is the distance between plastic zone boundaries in the direction of vertical diameter of tunnels. H_p and W_p parameters are seen in Figure 1. k is the ratio of horizontal in situ stress to vertical in situ stress, and Θ is angle with the horizontal.

$$H_p = D\sqrt{\left(\left[\Gamma\left[\sigma_v(1+k)/\left\{2\left(\frac{\sigma_c + k_p\left[\sigma_v(1+k)+\frac{\sigma_v(1-k)}{2}\left(\frac{4D^2}{H_p^{\,2}}\right)\right]}{k_p+1}\right)-\sigma_v\right\}\right]\right]+k_f\right)k_m} \tag{5}$$

$$W_p = D\sqrt{\left(\left[\alpha\left[\sigma_v(1+k)/\left\{2\left(\frac{\sigma_c + k_p\left[\sigma_v(1+k)-\frac{\sigma_v(k-1)}{2}\left(\frac{4D^2}{W_p^{\,2}}\right)\right]}{k_p+1}\right)-\sigma_v\right\}\right]\right]+k_f\right)k_m} \tag{6}$$

$$k_f = \sqrt{1-k^{-1}} \quad \text{for } W_p,\ k>1$$

$$k_f = \sqrt{1-k} \quad \text{for } H_p,\ k<1 \tag{7}$$

$$k_f = 0 \quad \text{for } W_p,\ k<1 \text{ and for } H_p,\ k>1$$

$$k_m = k^{-\left([2\sigma_v/\sigma_c]^{\alpha^{-2}}\right)}, \text{ for } W_p \tag{8}$$

$$k_m = \sqrt{k}^{\left([2\sigma_v/\sigma_c]^{\alpha^{-1}}\right)}, \text{ for } H_p \tag{9}$$

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$$W_p = D\sqrt{\left(\left[\sigma_v k \Big/ \left\{2\left(\frac{\sigma_c + k_p\left[\sigma_v(1+k) - \frac{\sigma_v(1-k)}{2}\left(\frac{4D^2}{W_p{}^2}\right)\right]}{k_p+1} - \sigma_v k\right)\right\}\right]\Gamma\right)(3k^{-1}-5)k_w} \tag{10}$$

$$k_w = [\sigma_v/2(\sigma_c - \sigma_v)]k^{-1}(\sigma_v/\sigma_c + 1) \approx A_{pc}k^{-1}(\sigma_v/\sigma_c + 1) \tag{11}$$

$$A_{pc} = \sigma_v/2(\sigma_c - \sigma_v) \tag{12}$$

$$k_p = (1+\sin\phi)/(1-\sin\phi) \tag{13}$$

where, σ_c is uniaxial compressive strength of the rock mass, σ_v *and* σ_h are in situ vertical and in situ horizontal stresses, respectively. *D* is the tunnel diameter in the equations. When *k* ratio is greater than 1, Γ is equal to 1. If *k* ratio is smaller than 1, Γ parameter is equal to k (when $k>1$, Γ=1, when $k<1$, Γ=k). α is 1 when *k* is smaller than 1, and is equal to $1/k$ when the *k* ratio is greater than 1 (when $k<1$, α=1, when $k>1$ α=$1/k$). To use in calculations, relation between uniaxial compressive strength values of rock masses and rock materials were considered as given in Eq.14, and internal friction angle of rock masses (ϕ) were determined as given in Eq. 15 [25].

$$\sigma_c/\sigma_{ci} = (RMR_{89})/(RMR_{89}+6(100-RMR_{89})) \tag{14}$$

$$\phi = 25(1+0.01RMR_{89}) \quad \text{for RMR>20} \tag{15}$$

The mean rock material strength (σ_{ci}) values for the tunnel parts needed to use SAF are given in Table 11. Additionally, Table 12 and Table 13 include various parameters to use in the plastic zone thickness estimation equations, such as tunnel diameter, depth, vertical stresses roughly calculated multiplying depth of the tunnel parts and the unit volume weight of rock masses. Because there was no in situ stress measurements in the field, different *k* ratios were used in the plastic zone thickness estimation equations to investigate different in situ stress distribution conditions. There were no observations of a determinative deformation in tunnels for an excessive *k* ratio. The plastic zone thickness and plastic zone load estimations for investigating usability of the RMR_{89} are given in Table 14.

Table 12. Representative values for tunnels to use in analytical and numerical study (σ_{ci}: uniaxial compressive strength of rock materials, σ_c: uniaxial compressive strength of rock masses)

	σ_{ci} (MPa)	σ_c (MPa)	γz (MPa)	SAF	Representative Model
Cerattepe Mine Main Haulage Gallery-North	29.7	10.2	4.9	0.87	Model 1
Cerattepe Mine Main Haulage Gallery-South	32.0	10.5	5.4	0.85	Model 1
Murgul Kabaca Derivation Tunnel	11.1	3.1	1.9	0.83	Model 2
Akarsen S. Mineral. Approach Tunnel	19.3	5.2	3.5	0.78	Model 3
Kızık Roadway Tunnel	10.6	2.9	1.7	0.89	Model 2

Table 13. Thickness of the plastic zone in vertical (H_t) and in horizontal (W_t) according to the approach suggested by Komurlu et al. (2015) [$H_t=(H_p-D)/2$, $W_t=(W_p-D)/2$]

Model	γz (MPa)	σ_c (MPa)	k	H_t (m)	W_t (m)
Model 1	5.4	10.5	0.5	0	0.43
	5.4	10.5	1	0.06	0.06
	5.4	10.5	1.5	0.97	0
	5.4	10.5	2	2.01	0
Model 2	1.9	3.1	0.5	0	1.32
	1.9	3.1	1	0.19	0.19

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	1.9	3.1	1.5	1.53	0
	1.9	3.1	2	3.85	0.17
Model 3	3.5	5.2	0.5	0	1.46
	3.5	5.2	1	0.28	0.28
	3.5	5.2	1.5	1.67	0.05
	3.5	5.2	2	4.36	0.39

Table 14. Stresses due to the plastic zone load ($P_d = \gamma H_t$) for the condition of k= 2 (Unit volume weight of rock mass is considered as 27 kN/m^3)

Model	**H_t**	**P_d (kPa)**
Model 1	2.0 m	54
Model 2	3.9 m	105
Model 3	4.4 m	119

As seen in Table 11, the plastic zone calculations were performed for rock masses with the RMR$_{89}$ values between 60 and 80, hence "Class 2" rock masses according to RMR$_{89}$ were analysed within this study. According to the RMR support chart given in Table 7, 3 m long rock bolts with 2.5 m x 2.5 m spacing is suggested to be used for Class 2 rock masses. To check whether its spacing is sufficient, load bearing capacities of rock bolts can be calculated using Eqs. 16 [26]. As another important point, the plastic zone thicknesses were found to be higher than the bolt length of 3 m which is suggested for the Class 2 rock masses, when k ratio is 2.

$$P_{b\max} = \frac{T_{bf}}{s_c s_l} \tag{16}$$

T_{bf} is the maximum load bearing capacity of a bolt, which is typically about 200 kN for grouted rock bolts with the diameter of 20 mm. T_{bf} depends on rock bolt material, diameter and also grout mix and its workmanship quality. s_c and s_l are spacing of the bolts perpendicular and parallel to the tunnel advance direction, respectively. P_{bmax} is the maximum support pressure supplied by rock bolting.

The plastic zone thicknesses in the tunnel parts whose classes decreased from "Class 2" to "Class 3" by using SAF values are not safe enough for applying 2.5 m x 2.5 m rock bolt spacing and a thin shotcrete liner as suggested in accordance with the use of RMR$_{89}$. In case of using supports suggested for Class 3 rock masses instead of the Class 2, a proper support can be supplied as required for the tunnel stability. The necessity to use strength adjustment factor (SAF) for rock materials is approved with the plastic zone thickness calculations. Even though plastic zones are aimed to carry themselves by bolting and prevention of their loosening, the tunnel supports are designed to be able to bear all the weight of the plastic zone. Loads of the loosened plastic zones, load bearing capacities of supports suggested for different rock classes in accordance with the RMR value are respectively given in Table 14 and Table 15. Loads in the table were calculated by multiplying plastic zone thickness and unit volume weight of the rock masses. The unit volume weight of rock masses can vary widely. Especially, metallic ore masses in the mine galleries have great unit volume weight values. The unit volume weight of 27 kN/m^3 was used in calculations given in Table 14. It should be noted herein that the load applied from plastic zones can be much higher in the metallic mines.

Table 15. Maximum support pressures supplied by rock bolts (T_{bf} is 200 kN)

Rock bolt span: 2.5 m x 2.5 m, Shotcrete thickness: 5 cm Class 2 (RMR=61-80)	**Rock bolt span: 1.5 m x 1.5 m Shotcrete thickness: 8 cm Class 3 (RMR=41-60)**	**Rock bolt span: 1.0 m x 1.0 m Shotcrete thickness: 12 cm Class 4 (RMR=21-40)**
P_{bmax} (kPa)	P_{bmax} (kPa)	P_{bmax} (kPa)
32	89	200

Plastic zone shapes change according to the k value. As seen in Figure 1, H_p is bigger than W_p when k is higher than 1. On the other hand, W_p is bigger than H_p when k is smaller than 1. For the hydrostatic stress distribution condition (k=1), W_p is equal to H_p around the tunnels with circular cross-section. To support the plastic zone around tunnels, rock bolt length should be selected considering whichever is bigger one within the W_p and H_p. For a proper plastic zone reinforcement application, rock bolts should be longer than the thickness of the plastic zone and be inserted in the elastic zone. As seen from the results, plastic zones in some cases are determined to occur with higher thicknesses than the bolt lengths suggested by the RMR support chart given in Table 7. Furthermore, the bolt span in support suggestions for the Class 2 (RMR 61-80) type rock masses was found quite

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large for supplying an appropriate load bearing capacity. Therefore, shotcrete liners become more dominant in terms of supplying the stability of the rock masses reinforced with non-effective bolts. It should be noted herein that the use of the bolt spans and lengths selected in accordance with the RMR_{us} value are determined to be more effective in comparison with the use of the unmodified RMR_{89} value.

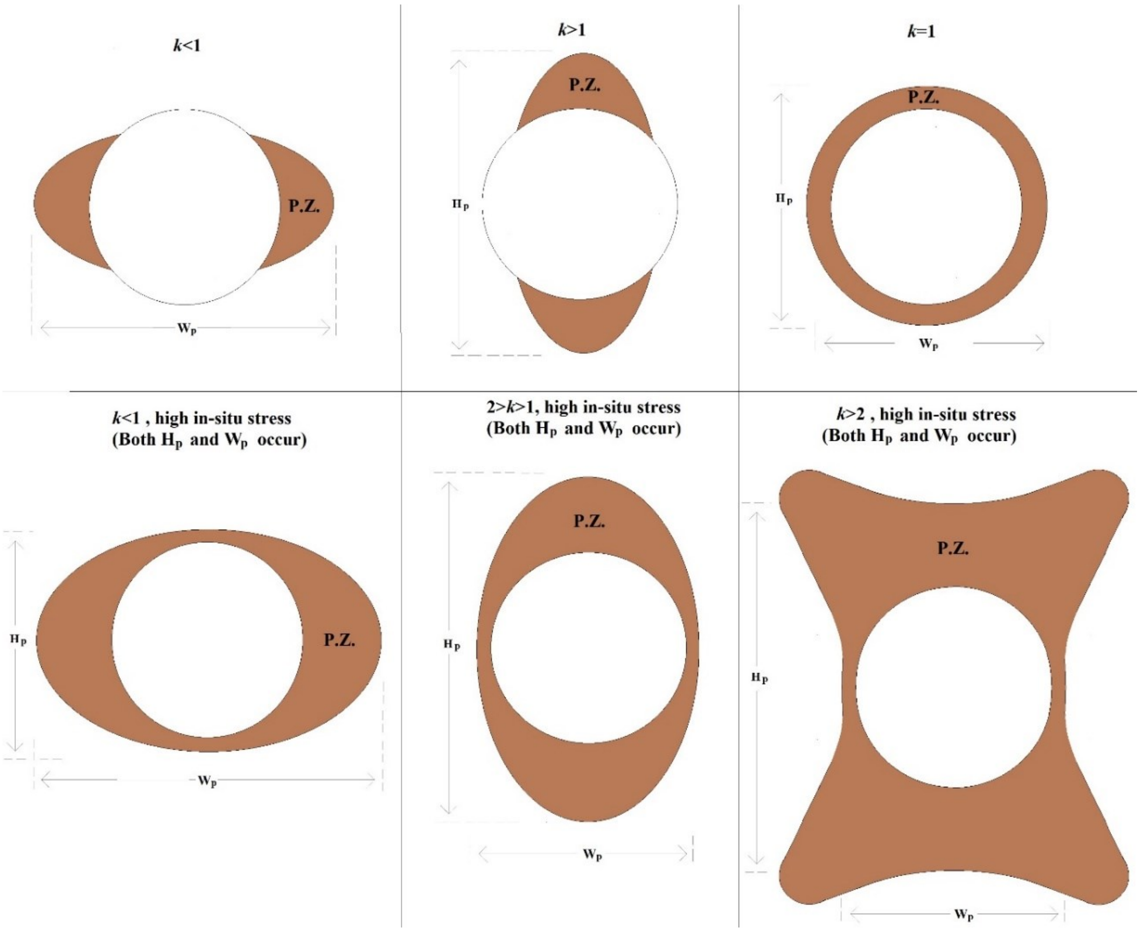

Figure 1. W_p and H_p parameters, some typical plastic zone (P.Z.) shapes for different k ratios

4 NUMERICAL STUDY

We performed a series of Finite Element Analyses (FEM) to investigate the support design in tunnels excavated in different rock mass classes using ANSYS software with special elements and material models for brittle materials like rocks and concretes. Rock mass strengths were calculated in line with rock material strength and RMR values (Eq. 14). Numerical modelling was performed for different modulus of elasticity and Poisson's ratio values. In the analyses, the ratio of modulus of elasticity to uniaxial compressive strength (MR value) of rock masses was changed from 400 to 800. Poisson's ratio (v) of the rock masses was taken as 0.2 and 0.4 for different rock mass models. Without analysing the conditions of extremely low or high horizontal in situ stresses, k values of 0.5, 1, 1.5 and 2 were used. The investigated models (Model 1-3) were also analysed by numerical modelling. The reason for various in situ stress distributions, rock mass strength, modulus of elasticity and Poisson's ratio values is to investigate supports selected in line with RMR_{us} value for different rock mass conditions.

To assess whether the loads of the plastic zones can be borne by the shotcrete liners, further finite element analyses were carried out for shotcrete liners in addition to the tunnel models in this study. The plastic zone load applied as distributed on the tunnel roof with the diameter of 7 m was modelled to analyse stress distributions and load bearing capacities of the shotcrete liners with different thicknesses. In the numerical modelling study, uniaxial compressive strength of the shotcrete material was taken 25 MPa. Modulus of elasticity and Poisson's ratio of the shotcrete material model were 16 GPa and 0.3, respectively. Eight-node solid brick elements (Solid65) were used for the three-dimensional modelling, which have the capability of cracking in tension, crushing in compression, plastic deformation, and three degrees of freedom at each node, including transition in the nodal x, y and z directions. Materials were modelled by considering the linear and non-linear properties defining the behaviours of the elements. The models were defined as linear elastic until the crack initiation occurs. After the crack initiation, change of the normal and shear stresses has been re-calculated by the program. The re-calculated shear stresses

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were transferred by the plasticity due to the generated open and closed cracks. The shear transfer coefficient was accepted as 0.3 and 0.1 for closed and open cracks, respectively. In addition, the stiffness reduction factor considered as 0.6 to define plasticity had an important role in the behaviour of cracked elements. These models predicted the failure of brittle materials according to the Willam–Warnke failure criteria used for concrete, rocks and other cohesive-frictional materials such as ceramics [27]. A static analysis was performed for each of the models, and the full Newton–Raphson method was used for non-linear analysis. For displacement-controlled loading, loads were divided into multiple sub-steps until the total load was achieved. Stress distributions and cracking mechanisms for all the models were plotted.

The mesh length in the rock mass models was chosen to be 0.2 m around tunnel where is the most critical part for the start of failure and increase from 0.2 m depending on the distance from the tunnel. Various finite element models with different meshes were analysed in an effort to ensure that the selected mesh is dense enough to provide sufficient solution convergence. In the shotcrete models, the mesh size was selected to be 5 mm. Some figures for meshes in the tunnel and shotcrete models are given in Figure 2 and Figure 3, respectively.

Numerical results in Table 16 and Table 17 are for plastic zone occurrence around tunnel and loads of the plastic zones in cases of different MR and Poisson's ratio values. The maximum support pressures of the shotcrete liner models with different thicknesses are in Table 18. Some stress distributions can be seen in Figures 4-6. We found that the plastic zone thickness can vary with the change in Modulus of elasticity and Poisson's ratio values. Especially, Poisson's ratio significantly changed the results. The plastic zone thickness was determined to increase with a decrease in modulus of elasticity and a decrease in the Poisson's ratio.

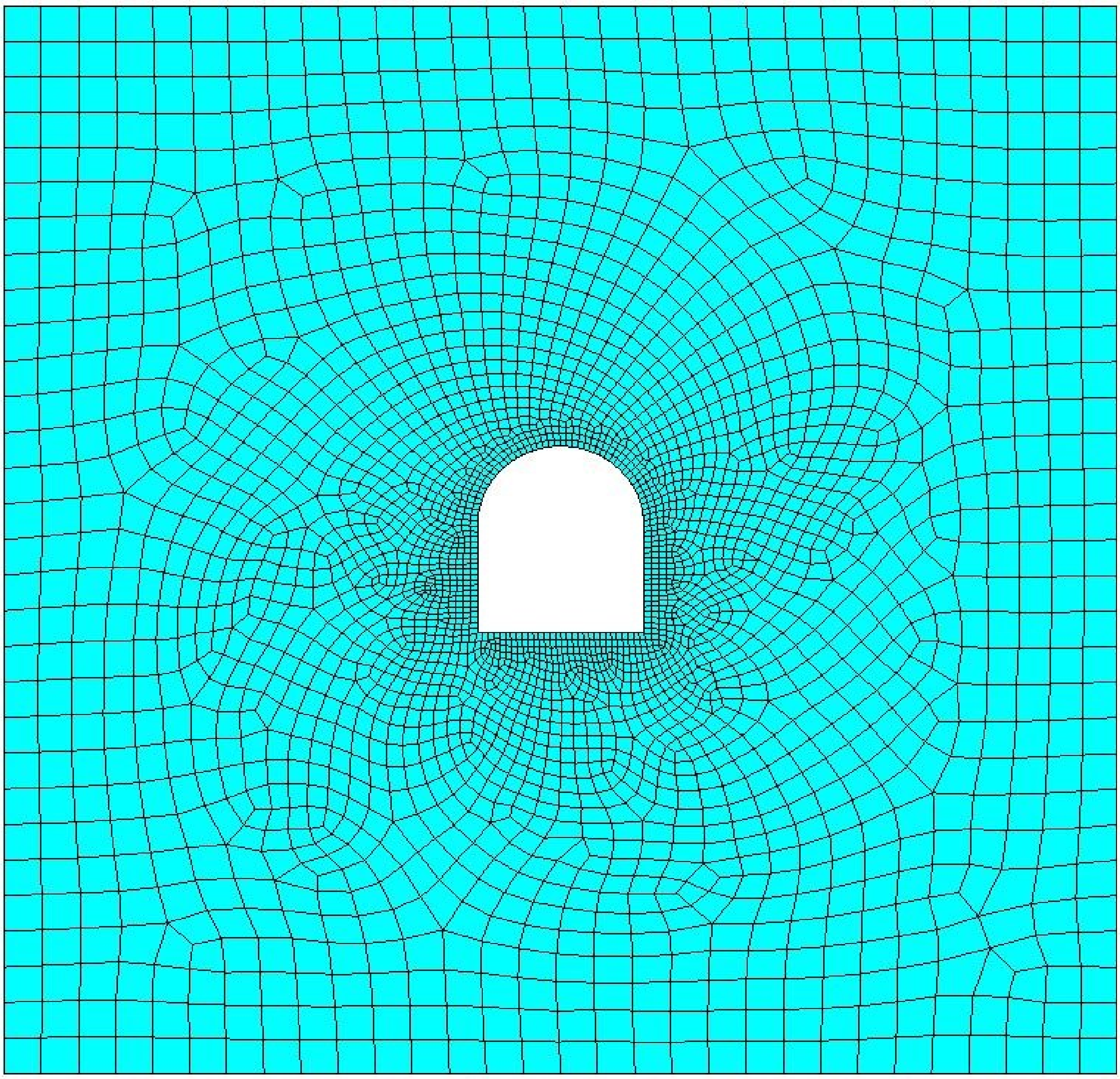

Figure 2. Meshes in tunnel models

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Figure 3. Meshes in shotcrete models

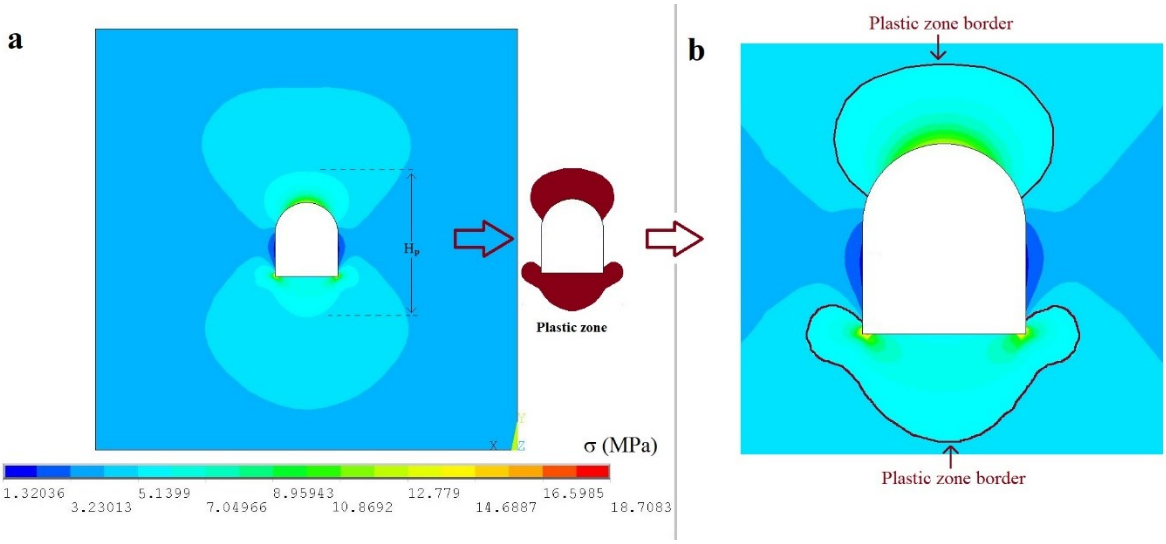

Figure 4. a) Stress distribution around tunnel, b) plastic zone borders (k= 1.5, MR= 800, v= 0.2, in-situ vertical stress= γz= 3.5 MPa)

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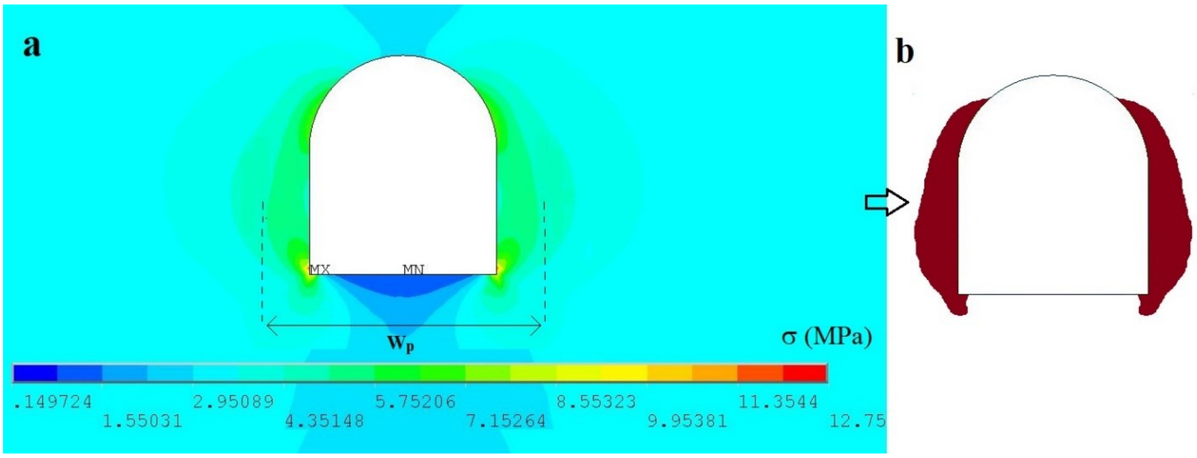

Figure 5. Stress distribution around tunnel, b) plastic zone (k= 0.5, MR= 400, v= 0.2, in-situ vertical stress= γz= 1.9 MPa)

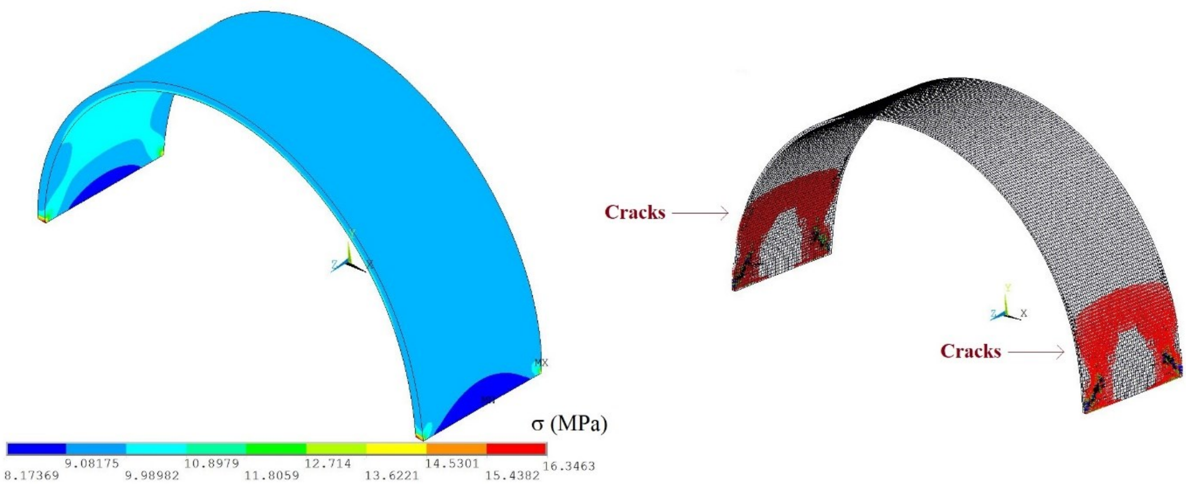

Figure 6. Stress (von Mises) distribution in shotcrete and cracks in the failed shotcrete model

Table 16. Numerical study results (t_p: thickness of the plastic zone, Dir.: direction, H: through the height of the tunnel, W: through the width of the tunnel)

Model	γz (MPa)	σc (MPa)	k	MR400, v0.2 t_p (m)	MR400, v0.2 Dir.	MR400, v0.4 t_p (m)	MR400, v0.4 Dir.	MR800, v0.2 t_p (m)	MR800, v0.2 Dir.	MR800, v0.4 t_p (m)	MR800, v0.4 Dir.
1	5.4	10.5	0.5	0.0	-	0.0	-	0.0	-	0.0	-
	5.4	10.5	1	0.6	H	0.3	-	0.5	H	0.0	-
	5.4	10.5	1.5	1.5	H	0.8	H	1.3	H	0.5	H
	5.4	10.5	2	3.6	H	2.7	H	3.4	H	2.6	H
2	1.9	3.1	0.5	1.8	W	1.1	W	1.5	W	0.9	W
	1.9	3.1	1	1.2	H	0.7	H	1.0	H	0.5	H
	1.9	3.1	1.5	2.4	H	1.5	H	2.1	H	1.2	H
	1.9	3.1	2	9.3	H	6.4	H	8.9	H	6.0	H
3	3.5	5.2	0.5	2.1	W	1.3	W	1.9	W	1.1	W
	3.5	5.2	1	1.4	H	0.9	H	1.2	H	0.7	H
	3.5	5.2	1.5	2.7	H	1.8	H	2.5	H	1.6	H
	3.5	5.2	2	10.5	H	7.0	H	9.8	H	6.7	H

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Table 17.Minimum (MR: 800 and v: 0.4) and Maximum (MR: 400, v: 0.2) loads of the plastic zone according to the results obtained from numerical study

	H_{tmin}	P_{dmin} (kPa)	H_{tmax}	P_{dmax} (kPa)
Model 1	2.6 m	70	3.6	97
Model 2	6.0 m	162	9.3	251
Model 3	6.7 m	181	10.5	284

Table 18. Maximum support pressures supplied by shotcrete liners (P_{smax}) with different thicknesses

t_c	P_{smax}
5 cm	227 kPa
8 cm	353 kPa
12 cm	419 kPa

Considering the change in the in-situ stress distribution, the plastic zone thickness is like to notably increase with an increase in the k ratio value. Since there is no parameter of k ratio, modulus of elasticity and Poisson's ratio of the rock masses in the empirical approaches, the support design should be convenient for use in different conditions. According to the results of the numerical analyses, shotcrete liners with the thickness of 5 cm which is suggested for the Class 2 type rock masses cannot be safe to support the plastic zones of the models when k ratio is 2. Based on the numerical analyses, the rock mass classes like Class 2, Class 3 or Class 4 were found for suggesting to be determined using RMR_{us} value instead of the RMR_{89} value.

5 DISCUSSION

In case of a very low rock mass strength, rock bolts anchorage performances are notably decreased [28-30]. Additionally, rock masses are disturbed while drilling processes because of low slake durability indexes of the poor rock materials. As also seen from the experience in the Akarsen mine, rock bolts are not able to be used in such kind rock masses. When rock bolts are not effective in quite poor rock masses and steel sets or pre-reinforcements are not used, the aim of preventing plastic zone to be loosened is carried out by a proper shotcrete application. Under the condition of low rock mass strength values, mechanical excavation with short advance distances should be applied with a proper shotcrete liner including reinforcements like wire meshes or fibres, which is sprayed right after the excavation process. If a rock mass has quite limited stand-by time, pre-reinforcements like forepoling and injection applications are applicable to prevent the loosening of the plastic zone.

For the case of using no pre-reinforcement and having non-effective rock bolts, load bearing performances of shotcrete liners were investigated within this study. According to the results of this study, the RMR_{89} value was assessed to be not usable to set a proper shotcrete liner to support the plastic zones of rock masses with low rock material strength values.

Rock bolts were not used in the Akarsen mine because of the damage of poor rock mass with the watery drilling operations. Also, there was no pre-reinforcement application in the mine. The strategy was advancing by the mechanical excavation and spraying fibre reinforced shotcrete with the thickness of 12 cm, right after each excavation steps. The shotcrete thickness was selected in accordance with the rock mass class determined using the RMR_{us} value. It should be noted herein that a high ground pressure for making rock squeezing problem was not seen in all the mines and tunnels worked in this study. Therefore, the findings and suggestions of this study are not for the squeezing rock masses. In case of having a squeezing problem, special yielding supports should be used to combat against problems resulting from excessive deformations [31-36].

The stability of the five tunnels in this study, which were supported in accordance with the RMR_{us} values confirms the use of strength adjustment factor (SAF) to determine accurate rock mass class and support details. The contemporary rock supports are set to reinforce a rock mass to carry itself and prevent the loosening of the plastic zone, whereas all the load of the plastic zone is aimed to be borne in the conventional support strategy. In soft rock masses, tunnel supports are designed to be able to carry the load of the plastic zone if rock bolts cannot work properly. Therefore, the plastic zone borders were determined first by both analytical calculations and numerical analyses.

The results obtained from the plastic zone thickness approach by Komurlu et al. [24] and numerical models with the MR value of 800 and the Poisson's ratio of 0.4 were found to be the most similar within different models with varying MR and Poisson's ratio values. The numerical models gave significantly higher plastic zone thicknesses than those of the analytical models, with a decrease in Modulus of Elasticity and Poisson's ratio values. This outcome is parallel with other previous studies [37-40]. Because the empirical methods are not suggested for a specific rock mass property, they should be able to be used in different rock masses with a wide variety in different properties like modulus of elasticity, Poisson's ratio and k ratio values. The MR value of 400 and

Poisson's ratio of 0.2 can be assessed to be in typical conditions and should be verified for bearing capacity of the shotcrete liners under the plastic zone load [41-44]. According to this study, rock supports suggested in the RMR chart for Class 2 type rock masses cannot be able to supply stability of investigated tunnels under the load of the plastic zone induced when the k ratio is 2. Therefore, the use of the SAF is found to be necessary.

The k ratio, the ratio of horizontal to vertical in-situ stresses changes in accordance with many parameters such as Poisson's ratio, depth, joints and cracks, water, temperature, topographic features, surface load, tectonic stresses (active tectonic stress, remnant tectonic stress), residual stresses (like magma cooling, metamorphism, metasomatism, etc.), terrestrial stresses like seasonal variations, moon pull, diurnal stresses and other geological features [45-51]. Because in-situ stress distribution and k ratio can immediately change underground, safe yielding zone estimation is preferable for the rock support design in engineering applications [52-55]. The k ratio of 2 is not extraordinary high to see in the rock engineering applications. Especially for shallow tunnels in rock, it is possible to see higher k values than 2 [56-58]. Therefore, an empirical rock support suggestion method should be convenient to use for the case of k=2. However, the RMR_{89} determined without using strength adjustment factor (SAF) was not found usable to supply a sufficient support for the case of high in situ stress and k ratio values. Depending on the RMR_{89} and SAF values, rock mass classes of the tunnels in this study were changed from Class 2 to Class 3 or Class 4 as the RMR_{us} value was used instead of the RMR_{89}. The use of the RMR_{us} value was determined to be advantageous in terms of supplying needed support pressures and the safety of tunnels.

6 CONCLUSION

The well-known RMR_{89} approach was assessed to have the lack of a convenient rock material strength effect parameter, the importance of which to determine RMR value should be more dominant especially for materials with quite low strengths. To fix the problem of ignoring an effectual parameter for the rock material strength value, a strength adjustment factor (SAF) depending on the in-situ stress and uniaxial compressive strengths of rock materials was suggested within this study. According to the results obtained from the field study, analytical and numerical analyses, a modified RMR value of RMR_{us} that includes the input of SAF parameter was assessed to be applicable for empirical support design works in tunnelling. The details of calculating the SAF parameter can be seen in Eqs. 3 and 4.

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