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Full Length Research Paper

Analysis of support requirements for a tunnel portal in weak rock: A case study from Turkey

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In this paper, engineering geological properties of the entrance portal section of Konakönü Tunnel, located on Black Sea Coastal Highway, Turkey, is studied. Moderately weathered basaltic and andesitic tuffs orderly to be driven by the entrance portal section. Tunnel portal design in weak rock presents some special challenges to the designers, since misjudgements in the design of tunnel can lead to very costly failures. The entrance portal was analyzed by empirical and numerical methods. Both field and laboratory studies were carried out during this study. Field studies involved geological mapping, detailed discontinuity surveying and sampling. Laboratory tests were carried out to determine the physico-mechanical and elastical properties of rock units. For this purpose, tunnel stability analyses were performed at the entrance portal section. The RMR (rock mass rating), Q (rock mass quality), RMi (rock mass index), NATM (new Austrian tunnelling method) and GSI (geological strength index) systems were employed for empirical rock mass quality determination and preliminary tunnel portal support design. The parameters calculated by the empirical methods were used as input parameters for the FEM (finite element method) analysis. The results from the two methods were compared and preliminary support designs were carried out. This comparison suggests that for more reliable support design, empirical and finite element methods should be combined.

Key words: Finite element method, rock mass classification systems, tunnel portal, weak rock.

INTRODUCTION

Determination of the most suitable and economical support system contribute to applicability of engineering projects for both design and construction stages.

Sometimes, even a small misinterpretation in the design stages can lead to costly and time-consuming failures at the construction phases (Sari and Pasamehmetoglu, 2004). Empirical and numerical methods are commonly used methods when underground engineering structures are designed. Rock mass classification systems are very useful tools for the preliminary design stage of a project, when very little detailed information on rock mass is available.

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The RMR, Q, RMi, NATM and GSI rock mass classification systems have been used by many researchers and have gained a universal acceptance (Barton, 2002; Ramamurthy, 2004; Basarir et al., 2005; Hoek and Diederichs, 2006; Gurocak et al., 2007; Gurocak, 2011). The rock mass classification systems have been originally obtained from many tunnelling case studies and they have been successfully applied to many tunnel construction designs in the past decades (Sari and Pasamehmetoglu, 2004). Although rock mass classification systems are very useful during the preliminary design stage, they do not adequately calculate stress distributions, support performance and deformations around the tunnel (Genis et al., 2007). Underground projects will be safer and more economical, if empirical approaches supported by numerical approaches.

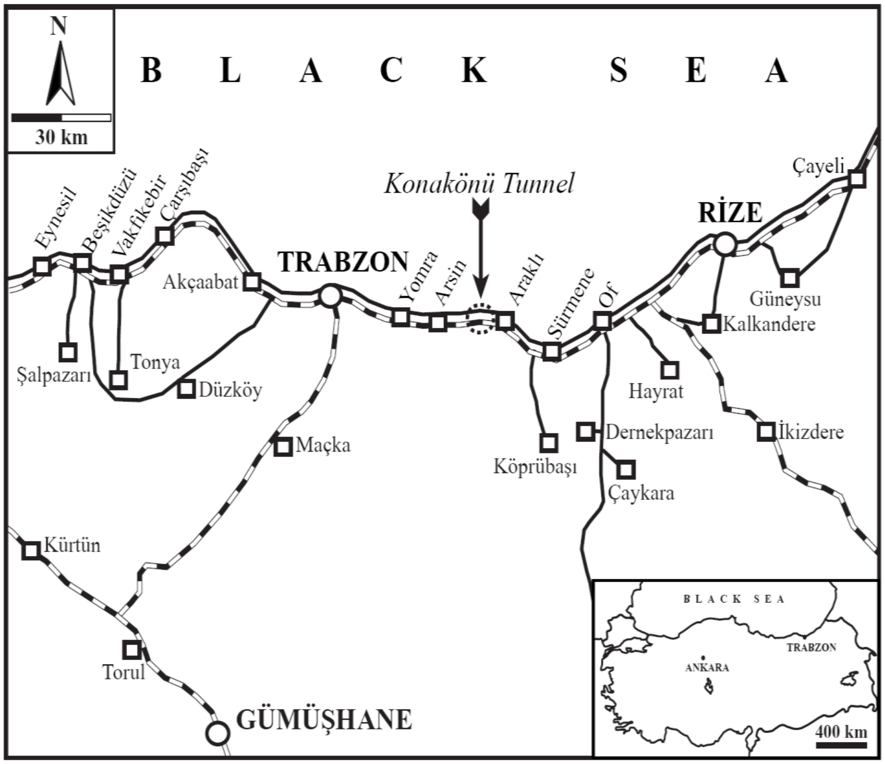

Figure 1. Location map of the study area.

The study area is located in the northern part of Turkey (Figure 1) where Konakönü Tunnel is located on the Black Sea Coastal Highway. Because of the heavy traffic load on the Black Sea Region, construction of the Konakönü Tunnel was decided by General Directorate of Highways authorities.

The double tubed tunnel has a horseshoe geometry, with a span of 12 m, height of 9 m and a length of 1890 m. The tunnel is excavated through the volcanic rocks of Eocene-aged Kabaköy Formation. Along the tunnel alignment, the lithology of the rock changes in short distances; because entrance portal involving moderately weathered basaltic and andesitic tuffs, it is the most problematic section of the tunnel. Hence, if these kinds of weak rock units analyse very carefully at the design stages, it will contribute to the applicability of the project in terms of its coast, safety and timing. Therefore, a detailed geological and geotechnical study was carried out in the study area to determine the engineering geological characteristics of the rock masses at entrance portal section. The rock masses were classified according to rock mass classification systems and the suppot design of the tunnel portal were selected by using both empirical and numerical methods.

GEOLOGY OF THE STUDY AREA

The study area is located in northern part of Eastern Pontide Tectonic Unit, Black Sea Region, Turkey. Eocene-aged Kabaköy Formation showing wide expansion along Konakönü (Arakli-Trabzon) tunnel alignment was first described by Güven (1993). This formation lithologically consists of andesite-basalt and their pyroclastics (Figure 2). The following volcanic rocks comprising moderately to highly weathered basaltic-andesitic tuffs, agglomerate and basalt are located in tunnel alignment. Moderately weathered basaltic and andesitic tuffs orderly to be driven by the entrance portal section. Moderately weathered basaltic tuffs have pale green and andesitic tuffs that have light gray colours. Tuffs contain sporadic pyrite occurrences and haematitisation is commonly seen in some places throughout the outcrops. Agglomerates consist of rounded-basalt pyroclastics having diameters of between 2 to 30 cm. Chloritisation is seen as patches in cement material of rock. Basalts are in dark to dark gray colour and they present a massive texture without any observable weathering products. Basaltic dykes, generally located along faults are associated with the Kabaköy Formation (Kaya, 2008). The Pliocene-aged Beşirli Formation is observed throughout the eastern part of the study area. The formation occurs as local outcrops a few tens of meters thick, rests disconformably on the underlying units and consists of weakly cemented conglomerates and breccias. Also, coarse sandstone and thickly bedded sandy limestone and basaltic

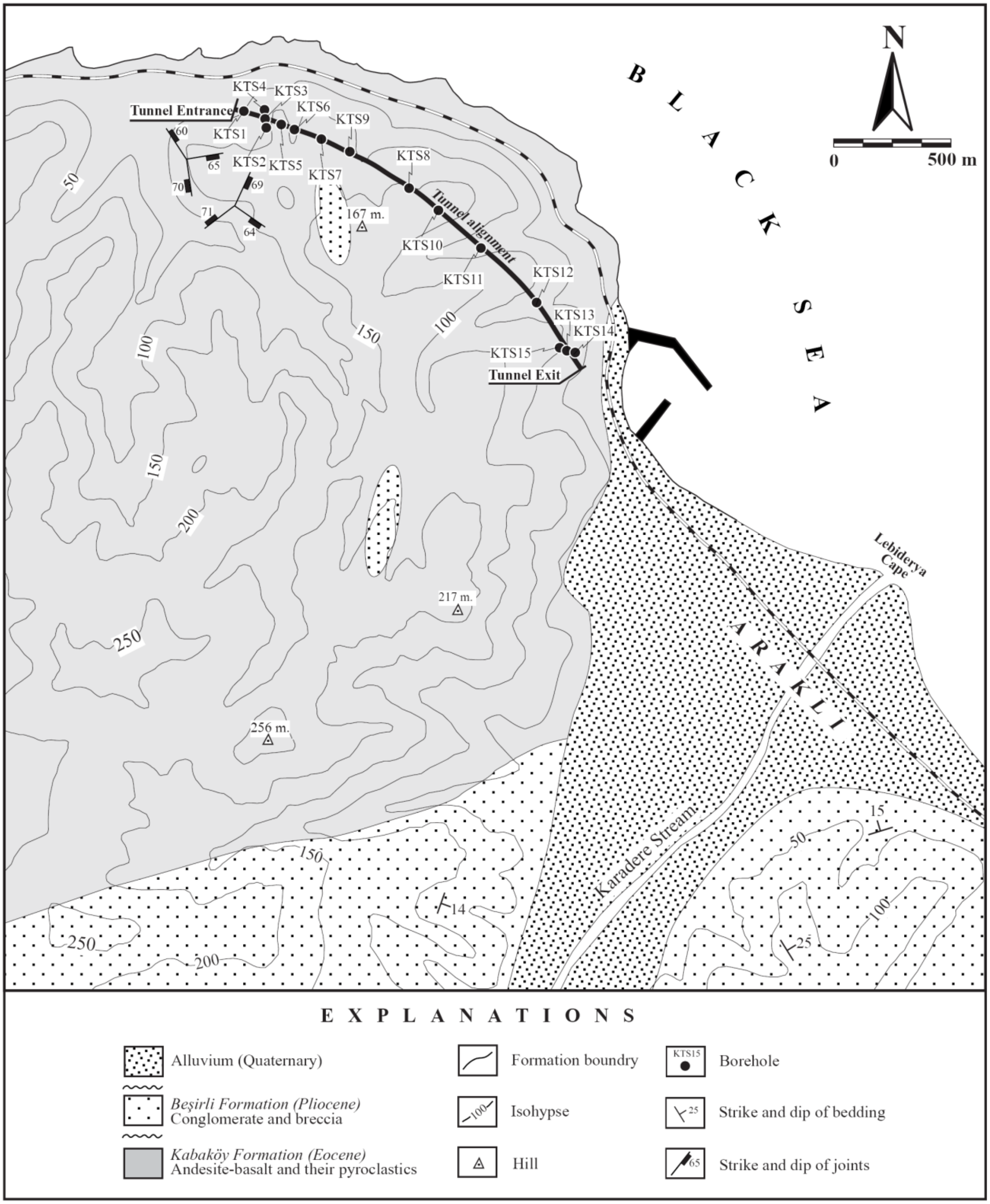

Figure 2. Geological map of the study area.

agglomerates can be observed within this formation (Güven, 1993). Quaternary recent sedimentary units are represented by alluvium of the streams and river valleys deposits. The widest outcrop can be observed in Karadere stream valley. It is composed of block, gravel, sand, silt and clay sized materials. The thickness of the alluvium varies in the range of 16 to 28 m (Güven, 1993).

FIELD AND LABORATORY STUDIES AT THE SITE

Field and laboratory studies consist of field observations,

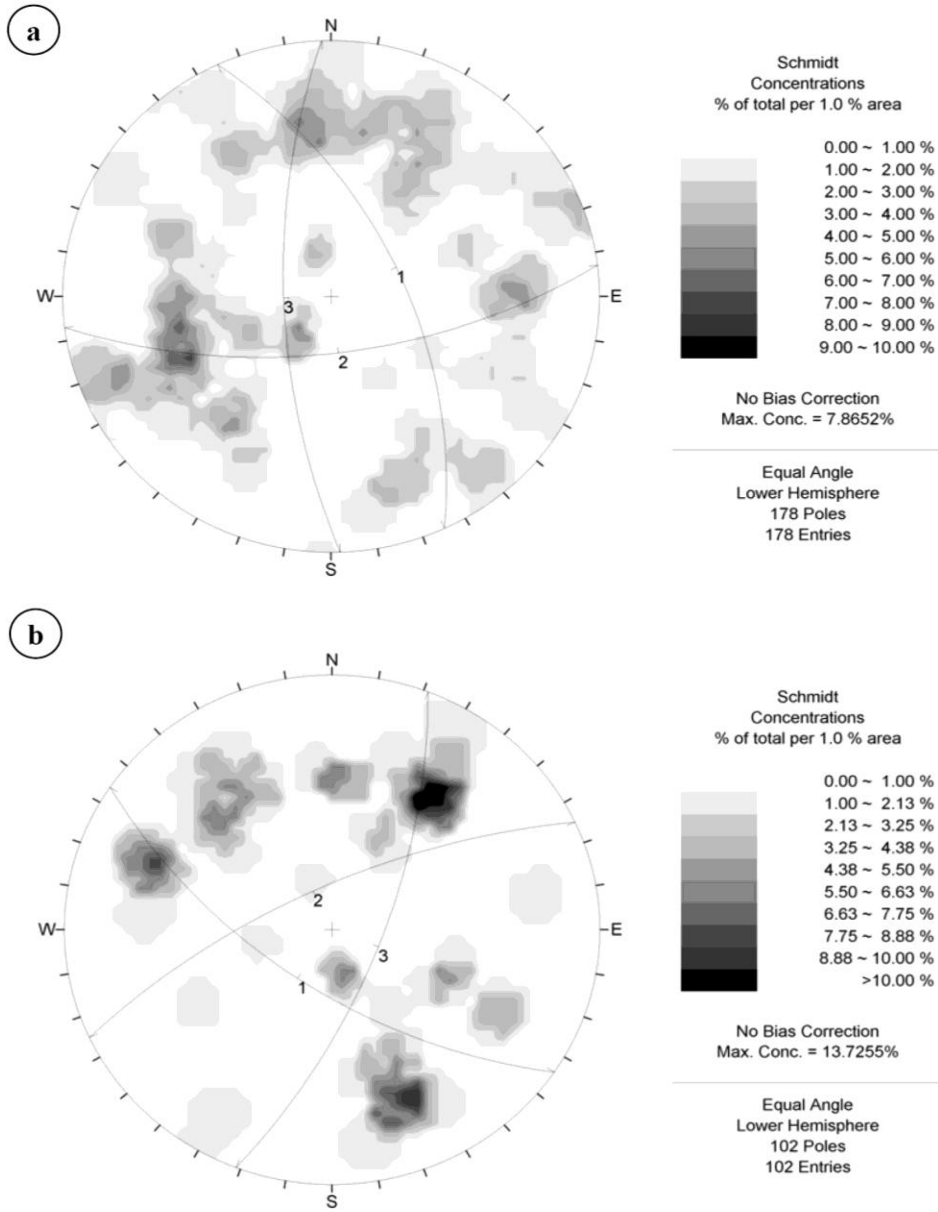

Figure 3. Stereographic projection of joint sets in moderately weathered basaltic tuff (a), and moderately weathered andesitic tuff (b).

scan-line surveys, boreholes and laboratory tests on the samples collected from field. Quantitative description of rock discontinuities such as orientation, persistence, roughness, spacing, degree of weathering, infilling and aperture were determined in the field in accordance with the ISRM suggested methods (ISRM, 1981). In the study area, a total of 280 joint measurements were taken from moderately weathered basaltic and andesitic tuffs. Discontinuity orientations were processed by utilizing a computer software, called Dips v5.1 (Rocscience, 2002), based on equal-area stereographic projection and dominant discontinuity sets have been distinguished on the entrance portal section (Figure 3). The determined dominant joint sets for moderately weathered basaltic tuff are written as follows:

Joint set 1: 60/65
Joint set 2: 65/173
Joint set 3: 70/268

The major orientations of the joint sets for moderately weathered andesitic tuff are listed as follows:

Joint set 1: 64/214
Joint set 2: 71/335
Joint set 3: 69/111

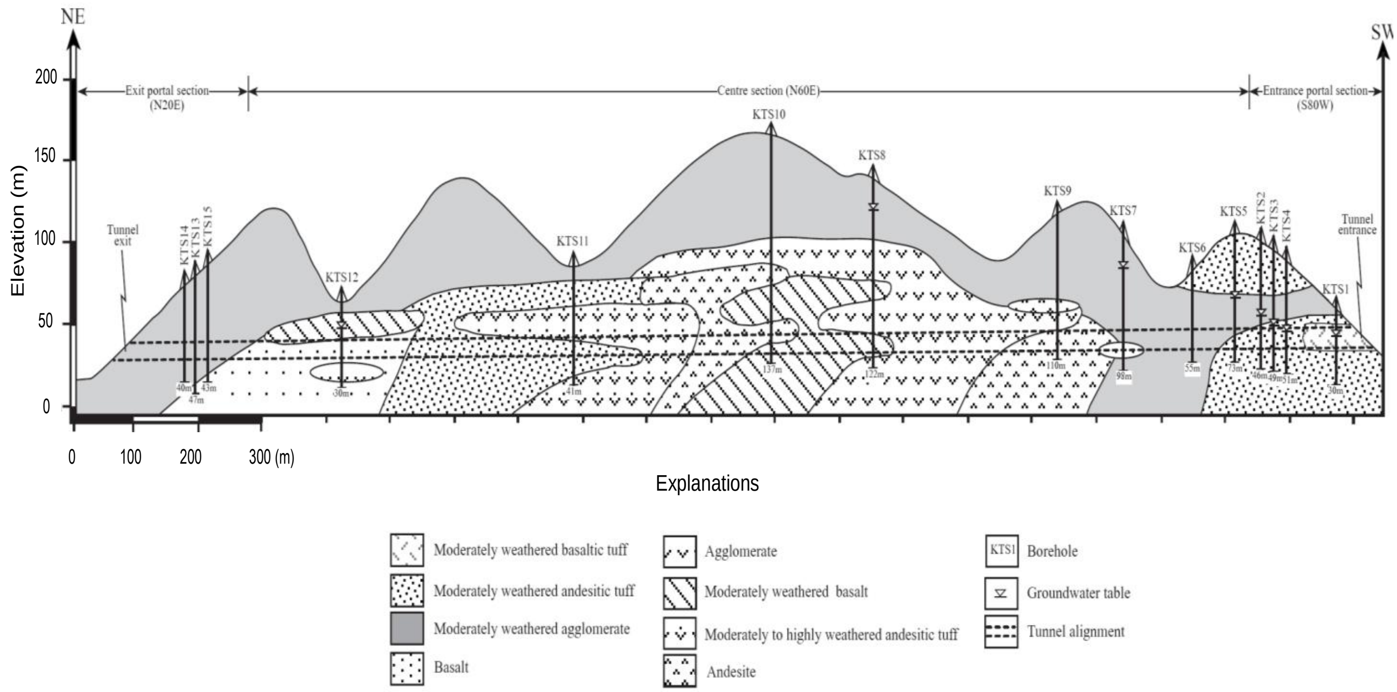

Figure 4. Borehole locations and geological cross-section along the tunnel route.

Table 1. Laboratory test results of the intact rock.

Parameters, symbol, unit	Moderetaly weathered basaltic tuff	Moderetaly weathered andesitic tuff
Uniaxial compressive strength, σ_{ci}, MPa	17.31-19.51 (18.58)[a]	21.04-39.21 (29.42)[a]
Point load strength index, $I_{S(50)}$, MPa	0.39-2 (0.80)[a]	0.77-4 (1.80)[a]
Unit weight, γ, kN/m^3	18.32-22.16 (20.11)[a]	18.04-20.81 (19.22)[a]
Young's modulus, E_i, Gpa	10.98	10.59
Poisson's ratio, υ	0.33	0.32

[a] Average value.

15 boreholes with a total length of 972 m and 4 of them with a total length of 176 m at the entrance portal section have been drilled on the Konakönü Tunnel alignment in order to observe the rock mass characteristics at the tunnel depth. The borehole locations and a geological cross-section along the tunnel route are shown in Figure 4. Laboratory experiments were conducted on core specimens, taken from core drillings and field. Laboratory experiments were carried out in accordance with the methods suggested by ISRM (1981) to determine the physico-mechanical and elastical properties of rock units, including uniaxial compressive strength, point load strength index, unit weight, Young's modulus and Poisson's ratio. Table 1 presents the results of the laboratory tests that were performed by the Applied Geology Laboratory of Geological Engineering Department of Karadeniz Technical University.

Engineering properties of joints were described using the scan-line survey method following ISRM (1981) description criteria. According to ISRM (1981), the joint sets in the moderately weathered basaltic tuff are closely spaced, with moderate persistence, open, smooth-undulating and moderately to highly weathered. The joint sets in the moderately weathered andesitic tuff are moderately spaced, with moderate persistence, open, smooth-undulating and moderately to highly weathered.

RQD (rock quality designation) values range from 55 to 91% for moderately weathered basaltic tuff and from 57 to 93% for moderately weathered andesitic tuff.

ROCK MASS CLASSIFICATION SYSTEMS

In this paper, most widely used rock mass classification systems such as RMR, Q, RMi, NATM and GSI were employed to characterize the rock masses and to estimate the rock mass strength parameters and deformation modulus.

RMR classification system

The RMR system was developed by Bieniawski (1974) based on experience in underground projects in South Africa. Afterwards, this classification system has gone through significant changes. These changes are mostly due to the ratings added for ground water, joint condition and joint spacing. In order to use this system, the uniaxial compressive strength of the intact rock, RQD, joint spacing, joint condition, joint orientation and groundwater condition have to be known. In this paper, the 1989 version of RMR (Bieniawski, 1989) system was used.

Q classification system

Barton et al. (1974) have developed Q system and it is also known as NGI rock mass classification (Norwegian Geotechnical Institute). This system is defined by the function of rock RQD, J_n (joint sets), J_r (joint roughness), J_a (joint alteration), J_w (water pressure) and SRF (stress reduction factor).

$$Q = \left(\frac{RQD}{J_n}\right)\left(\frac{J_r}{J_a}\right)\left(\frac{J_w}{SRF}\right) \quad (1)$$

A stress free form of Q was defined later by Goel et al. (1995) as Q_N which is given in Equation 2:

$$Q_N = \left(\frac{RQD}{J_n}\right)\left(\frac{J_r}{J_a}\right)J_w \quad (2)$$

Hoek et al. (1995) proposed the Q′ (modified tunnelling quality index), calculated in the same way as the standard Q system, except that the SRF and J_w are taken as 1.00:

$$Q' = \left(\frac{RQD}{J_n}\right)\left(\frac{J_r}{J_a}\right) \quad (3)$$

Recently, Barton (2002) has compiled the system again and has made some changes on the support recommendations. He has also included the strength of the rock material in the system.

$$Q_c = Q\left(\frac{\sigma_{ci}}{100}\right) \quad (4)$$

Where σ_{ci} is the strength of intact rock in MPa.

RMi classification system

The RMi system was developed by Palmström (1995) for the general characterisation of rock mass strength and rock mass deformation, design, calculation of the constants in the Hoek–Brown failure criterion for rock masses and TBM (tunnel boring machine) progress (Basarir et al., 2005). In 2000, some significant changes and adjustments for estimating preliminary rock support by using block volume and tunnel diameter were made. The RMi is a volumetric parameter indicating the approximate uniaxial compressive strength of a rock mass. It is expressed for jointed rock as (Palmström, 2000):

$$RMi = \sigma_{ci} JP = \sigma_{ci} 0.2\sqrt{jC} Vb^D \;\; (D = 0.37 jC^{-0.2}) \quad (5)$$

Where, σ_{ci}: the uniaxial compressive strength of intact rock in MPa, D: block diameter measured in meter, jC: joint factor, which is a combined measure for the jL (joint size), jR (joint roughness) and jA (joint alteration), given as:

$$jC = jL\frac{jR}{jA} \quad (6)$$

Vb: block volume, measured in m^3, JP: jointing parameter, which incorporates the main joint features in the rock mass. It can be found from the following equation:

$$JP = 0.2\sqrt{jC}Vb^D \quad (7)$$

Vb (block volume) was calculated using the following equation (Palmström, 2000):

$$Vb = \beta Jv^{-3} \quad (8)$$

Where, β is block shape factor and Jv is volumetric joint count.

Jv (volumetric joint number) was determined by using the Equation 9 proposed by Palmström (2005).

$$Jv = 44 - \frac{RQD}{2.5} \quad (9)$$

Where, RQD is rock quality designation.

NATM

NATM was developed between 1957 and 1965 in Austria. It was given its name in Salzburg in 1962 to distinguish it from old Austrian tunnelling approach. The main contributors to the development of NATM were Ladislaus von Rabcewicz, Leopold Müller and Franz Pacher. The main idea is to use the geological stress of the surrounding rock mass to stabilize the tunnel itself (Bieniawski, 1989). The main rock mass types based on NATM was divided into 10 classes by the Austrian standart Ö-NORM B2203 (1994). In this study, the excavation sequence and support systems were selected in accordance with the Ö-NORM B2203 (1994) standart and assessment with the NATM was made by using correlations with the RMR and Q systems according to the procedure given by the Turkish General Directorate of Highways (1997).

GSI

Geological strength index was developed by Hoek et al. (1995). The GSI is based on the appearance of a rock mass and its structure. Marinos and Hoek (2000) used additional geological properties in the Hoek–Brown failure criterion and introduced a new GSI chart for heterogeneous weak rock masses. The value of GSI was obtained from the last form of the quantitative GSI chart which was proposed by Hoek and Marinos (2000).

The input parameters of rock mass classification systems are given in Table 2 and calculated values of RMR, Q, Q_N, Q_C, Q′, RMi, GSI, NATM for moderately weathered basaltic and andesitic tuffs are given in Table 3. The results show that rock mass classification with the Q system leads to a more conservative rock mass classification than the RMR system. For the moderately weathered basaltic and andesitic tuffs, the Q system gives extremely poor rock mass classes while the RMR system gives poor rock mass classes (Table 3).

ESTIMATING ROCK MASS PROPERTIES

The rock mass strength parameters such as Hoek–Brown constants, deformation modulus and uniaxial compressive strength are essential input values for the numerical analysis. For this reason, the significance of the rock mass strength estimation parameters has increased during the past decades and there have been numerous studies on this subject. Rock mass properties were calculated by using empirical equations based on RMR, Q_c, Q_N, Q, Q′, RMi and GSI systems.

Deformation modulus of rock masses

Because, in-situ determination of the E_{mass} (deformation modulus of rock mass) is costly and often very difficult, several empirical equations have been suggested by researchers for estimating the modulus of deformation of the rock mass. Therefore, more relevant equations to tunnel design have been selected and the proposed equations by different researchers are presented in Table 4.

Strength of rock masses

Different researchers have recommended several empirical equations to calculate the σ_{cmass} (strength of rock mass) based on rock mass classification systems. The most widely used equations are given in Table 5. The rock mass strength values are calculated using these equations.

Hoek-Brown constants of rock masses

The Hoek–Brown failure criterion for rock masses uses m_b, s and a constants. Several researchers proposed correlations for Hoek–Brown constants. Some of these are applicable to tunnel design (Sari and Pasamehmetoglu, 2004; Özsan et al., 2009). Some suggested equations to calculate m_b and s constants are given in Table 6. Hoek et al. (2002) suggested the following equation to determine the ‘a constant’:

$$a=\frac{1}{2}+\frac{1}{6}\left(e^{-GSI/15}-e^{-20/3}\right) \tag{10}$$

In Hoek et al. (2002), equation D is the disturbance factor that depends on the amount of disturbance in the rock mass associated with the method of excavation. In this study, it was assumed that mechanical excavation is applied. Thus, the value of D was considered to be 0. To determine the m_i constants of the rock materials, Roclab v1.0 (Rocscience, 2007) software was used.

Residual strength parameters of rock masses

The post-peak behavior of rocks is important in the design of underground excavations because it has a significant influence upon the stability of the excavations.

At lower confinement levels, such as near excavation walls, most rocks exhibit some post-peak strength loss, and when strained sufficiently reach the residual strength (Cai et al., 2007). Due to the lack of large scale tests, it is very difficult to correctly represent the strain softening behavior of rock masses. Cai et al. (2007) consider this may overestimate the GSI_r (residual geological strength index) values for very good quality rock masses and proposed a new method based on the observation of actual rock mass failure processes from laboratory and in situ tests (Özsan et al., 2009). The GSI_r value can be empirically expressed as a function of the peak GSI value as:

$$GSI_r=GSIe^{-0.0134GSI} \tag{11}$$

Once the reduced GSI_r is obtained, the residual Hoek–

Table 2. RMR, Q and RMi rating for moderately weathered basaltic and andesitic tuffs.

Classification system	Input parameters	Moderately weathered basaltic tuff		Moderately weathered andesitic tuff	
		Value of parameters	Rating	Value of parameters	Rating
RMR	Uniaxial compressive strength (MPa)	18.58	2.8	29.42	4
	RQD (%)	85	17	89	17.8
	Discontinuity spacing (cm)	15.4	7	29.3	8.4
	Discontinuity condition				
	Persistence (m)	3-10	2	3-10	2
	Aperture (mm)	1-5	1	1-5	1
	Roughness	Smooth	1	Smooth	1
	Filling	<5 mm soft clay	2	<5 mm soft clay	2
	Weathering	Highly weathered	1	Highly weathered	1
	Goundwater condition	Dripping	4	Dripping	4
	Discontinuity orientation adjustment	Very unforable	-12	Very unforable	-12
Q	Joint set number (J_n)	Three joint sets + random	12*2	Three joint sets + random	12*2
	Joint roughness number (J_r)	Smooth, undulating	2	Smooth, undulating	2
	Joint alteration number (J_a)	Swelling clay fillings	10	Swelling clay fillings	10
	Joint water reduction factor (J_w)	Minor inflow	1	Minor inflow	1
	Stress reduction factor (SRF)	Multiple occurencess of weakness zones containing chamically integrated rock, very loose surrounding rock (any depth)	10	Multiple occurencess of weakness zones containing chamically integrated rock, very loose surrounding rock (any depth)	10
RMi	Joint condition factor (jC)				
	Joint roughness factor (jR)	Smooth, undulating	2	Smooth, undulating	2
	Joint alteration factor (jA)	Highly weathered	8	Highly weathered	8
	Joint size factor (jL)	3-10 m	2	3-10 m	2
	Block volume (Vb)				
	Block shape factor (β)	36	-	36	-
	Volumetric joint count (Jv)	6.47/m^3	-	3.41/m^3	-

Brown strength parameters can be calculated using the following equations:

$$m_{br} = m_i \exp\left[\frac{GSI_r - 100}{28}\right] \quad (12)$$

$$s_r = \exp\left[\frac{GSI_r - 100}{9}\right] \quad (13)$$

Table 3. The estimated rock mass classification systems.

Classification systems	Moderately weathered basaltic tuff		Moderately weathered andesitic tuff	
	Average value	Classification	Average value	Classification
Basic RMR	38	Poor	41.5	Fair
Adjusted RMR	26	Poor	29.5	Poor
Q	0.070	Extremely poor	0.074	Extremely poor
Q_N	0.70	Very poor	0.74	Very poor
Q′	0.70	Very poor	0.74	Very poor
Q_c	0.013	Extremely poor	0.022	Extremely poor
RMi	1.11	Weak	4.12	Medium
NATM	-	B3-Rolling C1- Rock bursting	-	B3-Rolling C1- Rock bursting
GSI	29	Very blocky	35	Very blocky

Table 4. The proposed emprical equations for calculation of E_{mass}.

Researchers	Equations	Notes	Equation number
Serafim and Pereira (1983)	$E_{mass} = 10^{(RMR-100)/40}$ (GPa)	For RMR<50	10
Nicholson and Bieniawski (1990)	$E_{mass} = E_i[0.0028RMR^2 + 0.9e^{(RMR/22.82)}]/100$ (GPa)		11
Mitri et al. (1994)	$E_{mass} = E_i[0.5(1 - (Cos(\pi RMR/100)))]$ (GPa)		12
Palmström (1995)	$E_{mass} = 5.6RMi^{0.375}$ (GPa)	For RMi > 0.1	13
Hoek and Brown (1997)	$E_{mass} = \sqrt{\frac{\sigma_{ci}}{100}10^{\frac{GSI-10}{40}}}$ (GPa)		14
Read et al. (1999)	$E_{mass} = 0.1(RMR/10)^3$ (GPa)		15
Ramamurthy (2001)	$E_{mass} = E_i e^{[(RMR-100)/17.4]}$ (GPa)		6
Ramamurthy (2001)	$E_{mass} = E_i e^{(0.8625 \log Q - 2.875)}$ (GPa)		17
Palmström and Singh (2001)	$E_{mass} = 7RMi^{0.4}$ (GPa)	For 1 < RMi < 30	18
Hoek et al. (2002)	$E_{mass} = \left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}10^{\frac{GSI-10}{40}}}$ (GPa)	For σ_{ci}<100 MPa	19
Barton (2002)	$E_{mass} = 10.Q_c^{1/3}$ (GPa)		20
Ramamurthy (2004)	$E_{mass} = E_i e^{-0.0035[5(100\text{-}RMR)]}$ (GPa)		21
Ramamurthy (2004)	$E_{mass} = E_i e^{-0.0035[250(1-0.3\log Q)]}$ (GPa)		22
Hoek and Diederichs (2006)	$E_{mass} = 100000\left(\frac{1-(D/2)}{1+e^{(75+25D-GSI)/11}}\right)$ (MPa)		23
Sonmez et al. (2006)	$E_{mass} = E_i 10^{[((RMR-100)(100\text{-}RMR)/4000\exp(\text{-}RMR/100))]}$ (GPa)		24
Chun et al. (2006)	$E_{mass} = 0.3228e^{(0.0485RMR)}$ (GPa)		25

RMR = rock mass rating; RMi = rock mass index; Q = rock mass quality; Q_c = normalized Q; GSI = geological strength index; σ_{ci} = uniaxial compressive strength of intact rock (MPa); E_i = Young's modulus (MPa) and D = disturbance factor.

Table 5. The proposed emprical equations for calculation of σ_{cmass}.

Researchers	Equations	Notes	Equation number
Hoek and Brown (1980)	$\sigma_{cmass} = \sigma_{ci}\sqrt{\exp(RMR-100)/9}$ (MPa)		26
Yudhbir et al. (1983)	$\sigma_{cmass} = \sigma_{ci}e^{7.65(RMR-100)/100}$ (MPa)		27
Ramamurthy (1986)	$\sigma_{cmass} = \sigma_{ci}\left[\frac{E_{mass}}{E_i}\right]^{0.7}$ (MPa)		28
Ramamurthy (1986)	$\sigma_{cmass} = \sigma_{ci}e^{(RMR-100)/18.75}$ (MPa)		29
Goel (1994)	$\sigma_{cmass} = \frac{5.5\gamma(Q_N)^{1/3}}{\sigma_{ci}B^{0.1}}$ (MPa)		30
Kalamaras and Bieniawski (1995)	$\sigma_{cmass} = \sigma_{ci}e^{(RMR-100)/24}$ (MPa)		31
Palmström (1995)	$\sigma_{cmass} = \sigma_{ci}JP$ (MPa)		32
Bhasin and Grimstad (1996)	$\sigma_{cmass} = \frac{\sigma_{ci}}{100}7\gamma Q^{1/3}$ (MPa)		33
Sheorey (1997)	$\sigma_{cmass} = \sigma_{ci}e^{(RMR-100)/20}$ (MPa)		34)
Singh et al. (1997)	$\sigma_{cmass} = \sigma_{ci}s^{n}$ (MPa)	For GSI≥25 n = 0.5; for GSI<25 n = 0.6	35
Trueman (1998)	$\sigma_{cmass} = 0.5e^{0.06RMR}$ (MPa)		36
Aydan and Dalgic (1998)	$\sigma_{cmass} = \frac{RMR}{RMR+\beta(100-RMR)}\sigma_{ci}$ (MPa)	β = 6	37
Barton (2000)	$\sigma_{cmass} = 5\gamma Q_c^{1/3}$ (MPa)		38
Hoek et al. (2002)	$\sigma_{cmass} = \sigma_{ci}s^{a}$ (MPa)		39
Ramamurthy (2004)	$\sigma_{cmass} = \sigma_{ci}e^{-0.008\left[250(1-0.3\log Q)\right]}$ (MPa)		40
Ramamurthy (2004)	$\sigma_{cmass} = \sigma_{ci}e^{-0.008\left[5(100\text{-}RMR)\right]}$ (MPa)		41

RMR = rock mass rating; Q = rock mass quality; Q_c = normalized Q; Q_N = stress-free Q; s, a = rock mass constants; JP = jointing parameter; σ_{ci} = uniaxial compressive strength of intact rock (MPa); E_i = Young's modulus (GPa); E_{mass} = deformation modulus of rock mass (GPa); B = tunnel width (m) and γ = density of rock mass (t/m^3).

$$a_r = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI_r/15} - e^{-20/3}\right) \quad (14)$$

Where m_i is intact rock constant.

Because the rock masses are in a damaged residual state, D = 0 is used for the residual strength parameter calculation (Cai et al., 2007). The calculated deformation modulus, strength, Hoek–Brown constants and residual strength constants of moderately weathered basaltic and andesitic tuffs are listed in Table 7.

SUPPORT DESIGN OF THE ENTRACE PORTAL OF TUNNEL

Although rock mass classification systems are generally applied to carry out the support design of tunnels, these systems cannot give a quantitative description to a specific rock mass and they fail to predict interaction between the surrounding rock mass and supporting system, thus fail to give descriptions on the developments of the support and behaviour of supported structures such as tunnel deformation and stress redistribution (Basarir, 2006). Thus, in this study for the safe tunnel support design, empirical and numerical methods were employed.

Emprical support design

Based on RMR, Q, RMi and NATM classification systems, the necessary support systems, proposed excavation

Table 6. The proposed emprical equations for calculation of m_b and s constants.

Researchers	Equations	Equation number
Hoek et al. (1995)	$m_b = m_i 0.135(Q')^{1/3}$	42
Palmström (1995)	$m_b = m_i JP^{0.64}$	43
Hoek and Brown (1998)	$m_b = m_i e^{\left(\frac{RMR-100}{28}\right)}$	44
Hoek et al. (2002)	$m_b = m_i e^{\left(\frac{GSI-100}{28-14D}\right)}$	45
Hoek et al. (1995)	$s = 0.002Q'$	46
Palmström (1995)	$s = JP^2$	47
Hoek and Brown (1998)	$s = e^{\left(\frac{RMR-100}{9}\right)}$	48
Hoek et al. (2002)	$s = e^{\left(\frac{GSI-100}{9-3D}\right)}$	49

RMR = rock mass rating; Q = rock mass quality; Q'= modified Q; GSI = geological strength index; JP = jointing parameter; D = disturbance factor and m_i = intact rock parameter and γ = density of rock mass (t/m^3).

Table 7. Calculated rock mass strength parameters.

Parameters, symbol, unit	Moderetaly weathered basaltic tuff	Moderetaly weathered andesitic tuff
E_{mass} (GPa)	0.03-7.30 (2.47)[a]	0.03-12.33 (3.36)[a]
σ_{cmass} (MPa)	0.16-6.55 (1.69)[a]	0.26-13.16 (3.06)[a]
m_b constant	1.03-2.15 (1.54)[a]	1.03-3.69 (2.04)[a]
s constant	0.0010-0.9163 (0.0016)[a]	0.0015-0.9208 (0.0058)[a]

[a] Average value.

methods and support installation time for the rock units along the entarance portal of tunnel are presented in Table 8. The proposed excavation method and support types are also illustrated in Figure 5. RMR (rock mass rating) classification system suggests support elements only for horseshoe shaped span of 10 m, vertical stress lower than 25 MPa, drilling and blasting construction (Bieniawski, 1989). Therefore, the preliminary support systems proposed by Q and RMi systems were applied and verified with Phase^2 software for the entarance portal of tunnel.

Numerical support design

To determine the induced stresses, deformations, developed plastic zone around the tunnel and to verify the results of the empirical analysis, the computer software Phase^2 v6.0, 2D plastic finite element program, developed by Rocscience (2006) was used in the numerical analysis. An automatic mesh around the tunnel is generated and based on the elasto-plastic analysis, deformations and stresses are computed in this program. In order to analyse tunnel stability and deformations in rock masses and to explore the concept of rock support interaction, a very simple model is utilized. Three-noded-triangular finite elements were used in the mesh (Figure 6). Finer zoning was used around the excavation. Also, seisminc load was added to the model taking into account the earthquake risk of the region. According to the earthquake zoning map, the study area falls within the zone classified as fourth degree earthquake risk where the expected acceleration values are between 0.1 and 0.2 g (Turkish Ministry of Public Works and Settlement, 1996). Tunnel width and height are 12 and 9 m, respectively. The entrance portal of tunnel will have an average overburden of about 25 m for moderately weathered basaltic tuff and about 45 m for moderately weathered andesitic tuff. The outer model boundary was set by considering the overburden of tunnel. In order to simulate excavation of entrance portal of tunnel in moderately weathered basaltic and andesitic tuffs, two finite element models were generated using same mesh and tunnel geometry and different material properties. Hoek–Brown failure criterion was used to estimate yielded elements and plastic zone of rock masses in the vicinity of tunnel.

The rock mass properties assumed in this analysis

Table 8. Emprical tunnel support categories according to RMR, Q, RMi and NATM systems.

	Rock unit	Moderately weathered basaltic tuff	Moderately weathered andesitic tuff
	Construction phase	Top heading and bench	Top heading and bench
	Excavation method	Mechanical excavation	Mechanical excavation
	Round length	1-1.5 m advance in top heading and bench	1-1.5 m advance in top heading and bench
	Stand-up time	Immediate collapse	Immediate collapse
	Support time	Install support concurrently with excavation 10 m from face	Install support concurrently with excavation 10 m from face
	Max. unsuported roof span (m)	0.7	0.7
	Max. unsuported wall heigth (m)	0.7	0.7
RMR	Value (adjusted)	26	29.5
	Support	Roof: Systematic rock bolts (Ø20 mm, fully grouted) L = 4-5 m long, spaced 1-1.5 m. 100-150 mm thick shotcrete with wire mesh. Wall: Systematic rock bolts (Ø20 mm, fully grouted) L = 4-5 m long, spaced 1-1.5 m. 100 mm thick shotcrete with wire mesh. Additional support: Ligth to medium ribs spaced 1.5 m where required.	Roof: Systematic rock bolts (Ø20 mm, fully grouted) L = 4-5 m long, spaced 1-1.5 m. 100-150 mm thick shotcrete with wire mesh. Wall: Systematic rock bolts (Ø20 mm, fully grouted) L = 4-5 m long, spaced 1-1.5 m. 100 mm thick shotcrete with wire mesh. Additional support: Ligth to medium ribs spaced 1.5 m where required.
Q	Value	0.070	0.074
	Span/ESR	12	12
	Support	Roof: Systematic rock bolts L = 3.8 m long, spaced 1.2-1.3 m (B). 150-250 mm thick steel fibre reinforced shotcrete (Sfr). Wall: Systematic rock bolts L = 3.4 m long, spaced 1.2-1.3 m (B). 150-250 mm thick steel fibre reinforced shotcrete (Sfr). Additional support: Steel rib-reinforced-shotcrete arches (RRS).	Roof: Systematic rock bolts L = 3.8 m long, spaced 1.2-1.3 m (B). 150-250 mm thick steel fibre reinforced shotcrete (Sfr). Wall: Systematic rock bolts L = 3.4 m long, spaced 1.2-1.3 m (B). 150-250 mm thick steel fibre reinforced shotcrete (Sfr). Additional support: Steel rib-reinforced-shotcrete arches (RRS).
RMi	Value	1.11	4.12
	Support	Roof: Systematic rock bolts L = 3.7 m long, spaced 1.2-1.5 m. 200-250 mm thick steel fibre reinforced shotcrete. Wall: Systematic rock bolts L = 2.8 m long, spaced 1.5-1.7 m. 100-150 mm thick steel fibre reinforced shotcrete.	Roof: Systematic rock bolts L = 3.5 m long, spaced 1.5-1.7 m. 100-150 mm thick steel fibre reinforced shotcrete. Wall: Systematic rock bolts L = 2.7 m long, spaced 1.7-2.0 m. 60-80 mm thick steel fibre reinforced shotcrete.
NATM	Class	B3/C1	B3/C1
	Support	Utilization of systematic support in roof and wall.	Utilization of systematic support in roof and wall.

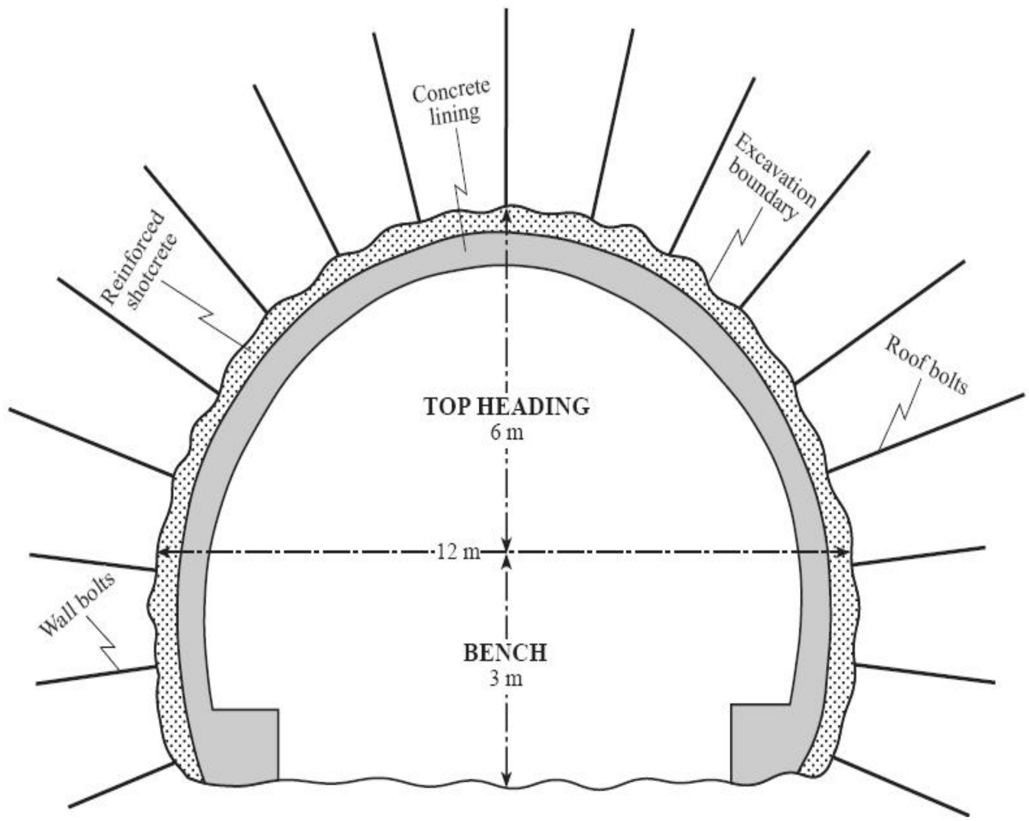

Figure 5. Tunnel cross-section.

were obtained from the estimated values and are given in Table 9 (Turkish General Directorate of Highways, 2006).

The loading conditions for vertical stress are taken as an increasing trend with depth due to its overburden weight and are estimated by:

$$\sigma_v = \gamma H \tag{15}$$

Where, γ is unit weight of the rock mass in MN/m^3 and H is the depth of overburden in meter.

It is more difficult to estimate undisturbed horizontal stress. It is known that they are variable at shallow depth, tending to a hydrostatic state in deep environment (Hoek and Brown, 1978). In this case, the horizontal stress estimation is obtained from the equation suggested by Sheorey et al. (2001) as in Equation (16).

$$\sigma_h = \frac{\nu}{1-\nu}\sigma_v + \frac{\beta E_{mass} G}{1-\nu}(H+100) \tag{16}$$

Where β = 8 × 10^{-6}/°C (coefficient of linear thermal expansion), G = 0.024°C/m (geothermal gradient), υ is the Poisson's ratio and E_{mass} is deformation modulus of rock mass in MPa.

The double tubed tunnel has been modeled in two stage excavation as top heading and bench. This construction phase has been simulated by $Phase^2$ programme. Both unsupported and supported cases were analysed for each rock unit. In the first step, in situ stress distributions (gravity loading due to the thickness of the overburden at the tunnel design elevation) were examined. In the following two step, the principal stress distributions, yielded points and the induced displacements developed around the tunnels were analyzed using top heading followed by excavating the entire tunnel. In the final step, the performance of the preliminary support systems obtained from the Q and RMi classification systems was investigated. The properties of support elements, such as length, pattern of bolts and thickness of shotcrete are same as those proposed in Table 8. The characteristics of the support elements are presented in Table 10. The numerical analyses were undertaken for unsupported cases (Figures 7 and 8) and the radius of the plastic zone for each rock unit were determined (Table 11). The maximum stress concentration develops at the top and the corner of the excavations. It can be seen from Table 11 that the maximum total displacement values for each section are very small. However, the extent of the plastic zone and elements undergoing yielding suggest that

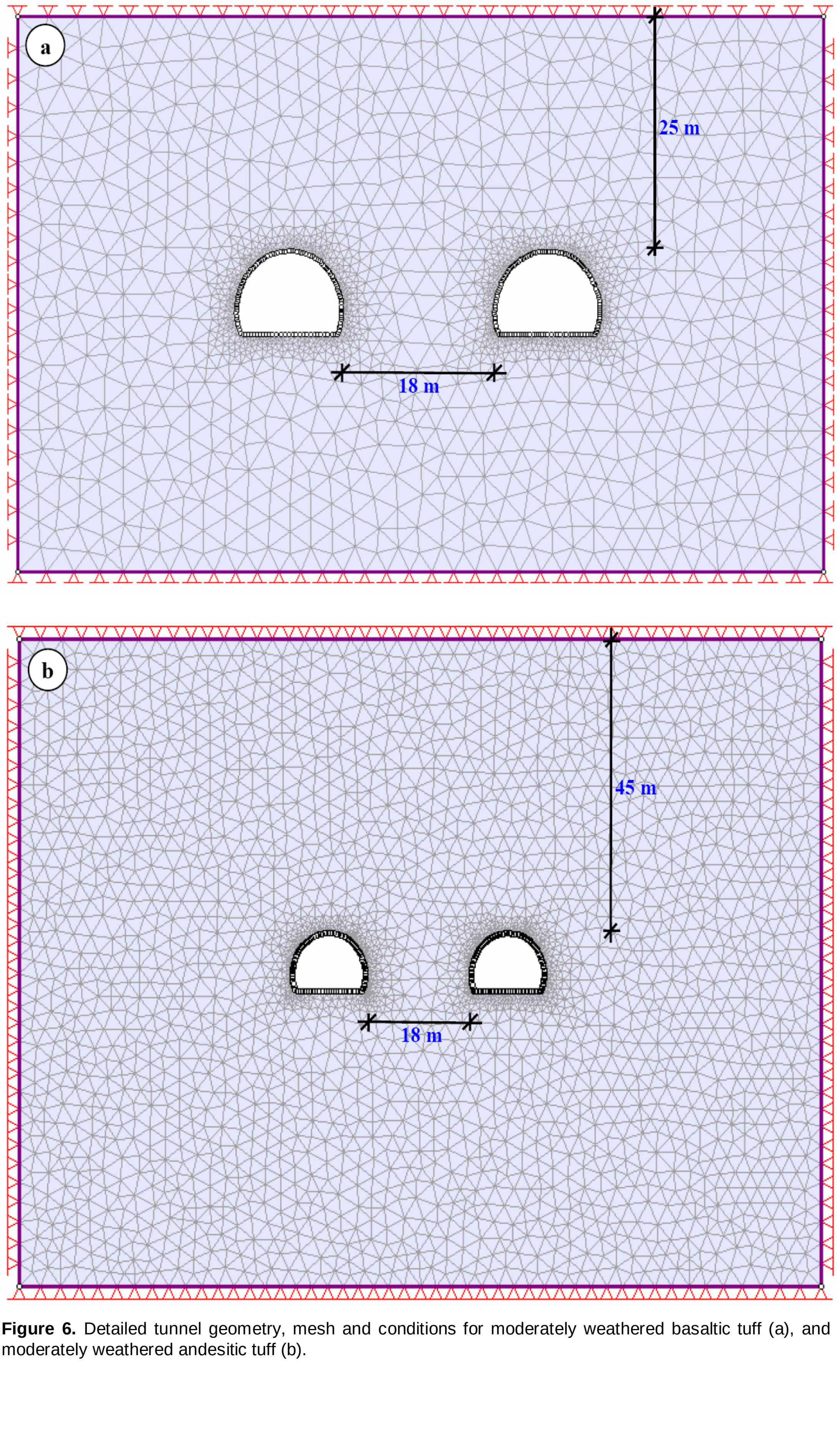

Figure 6. Detailed tunnel geometry, mesh and conditions for moderately weathered basaltic tuff (a), and moderately weathered andesitic tuff (b).

Table 9. Material properties of rocks for the numerical model.

Property	Moderately weathered basaltic tuff	Moderately weathered andesitic tuff
Elastic type	Isotropic	Isotropic
E_{mass} (GPa)	2.47	3.36
Poisson's ratio (υ)	0.33	0.32
σ_{cmass} (MPa)	1.69	3.06
Material type	Plastic	Plastic
m_i constant	13	13
m_b constant	1.54	2.04
s constant	0.0016	0.0058
a constant	0.515	0.511
m_{br} constant	0.738	0.799
s_r constant	0.00013	0.00017
a_r constant	0.545	0.539
Dilation parameter	0°	0°
Vertical stress, σ_v (MPa)	0.503	0.865
Horizontal stress, σ_h (MPa)	0.248	0.408

Table 10. The characteristics of the support elements employed in the analyses.

Property	Shotcrete	Rock bolt
Young's modulus, E (GPa)	20	200
Poisson's ratio, υ	0.2	-
Peak compressive strength, σ_{cp} (MPa)	20	-
Residual compressive strength, σ_{cr} (MPa)	3.5	-
Peak tensile strength, σ_{tp} (MPa)	3.1	-
Residual tensile strength, σ_{tr} (MPa)	0	-
Peak load (MN)	-	0.25
Residual load (MN)	-	0.025
Type	-	Ø28 mm, fully bonded.

there would be stability problems for the entrance portal of tunnel.

RESULTS

In this study, detailed engineering geological investigations were carried out to estimate the rock mass quality and support elements for moderately weathered basaltic and andesitic tuffs on the entrance portal section of the Konakönü Tunnel. Based on the information collected in the field and laboratory, the rock mass classes of tunnel ground and empirical support types/categories were determined according to the RMR, Q, RMi and NATM classification systems. According to RMR system, moderately weathered basaltic and andesitic tuffs were classifed as poor rock masses. Rock mass classification with the Q system, both moderately weathered basaltic and andesitic tuffs were classifed as extremely poor rock masses. The Hoek-Brown failure criterion was used for the rock masses to make a reliable estimate of the geomechanical properties of the rock masses. Because of limitations of RMR system, support elements suggested by Q and RMi classification systems were investigated in numerical analyses. Usage of empirical indexes in numerical modeling gave more realistic results. It was determined that, the support elements suggested by Q and RMi classification systems were successful to overcome the stability problem and to decrease in both the number of yielded elements and the size of plastic zone around the portal section of tunnel.

DISCUSSION

According to the results, rock mass classification with the Q system leads to a more conservative rock mass classification than the RMR system. While the RMR system gives poor rock mass classes, the Q system gives extremely poor rock mass classes for the rock units

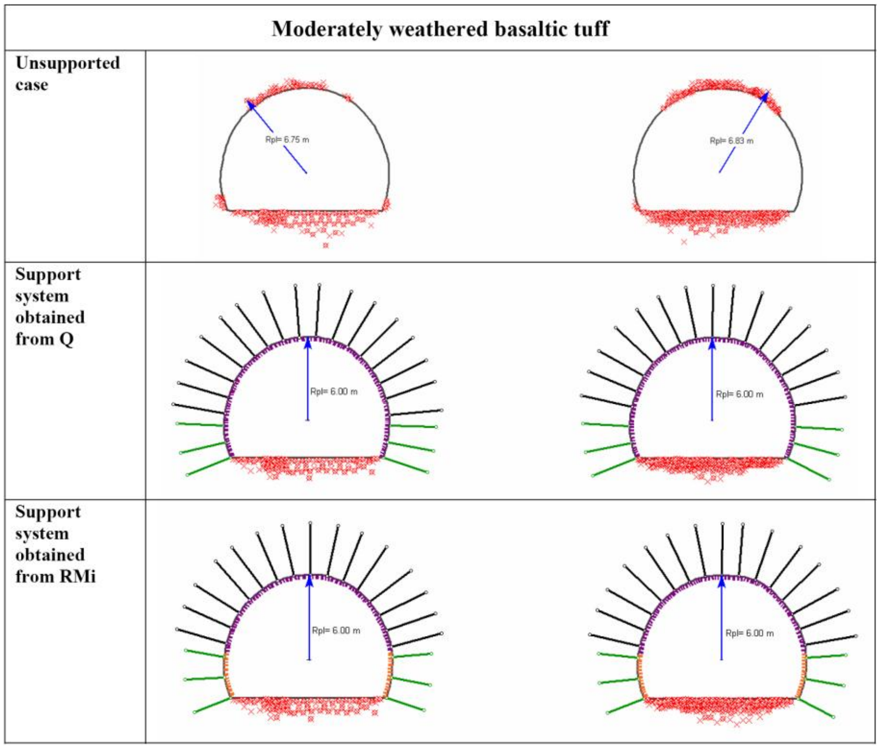

Figure 7. Radius of plastic zone and maximum total displacements for unsupported and supported cases in moderately weathered basaltic tuff.

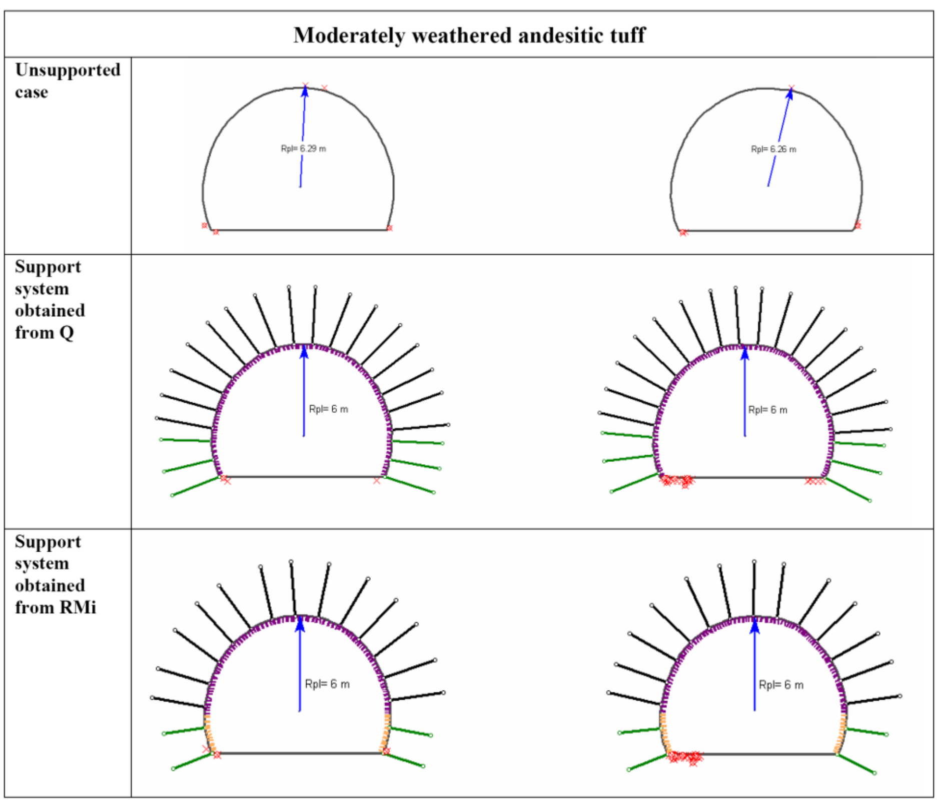

Figure 8. Radius of plastic zone and maximum total displacements for unsupported and supported cases in moderately weathered andesitic tuff.

Table 11. Stresses, displacements and radius of plastic zones for the unsupported and supported case.

		Moderately weathered basaltic tuff						Moderately weathered andesitic tuff					
		Unsupported case		Supported case for Q		Supported case for RMi		Unsupported case		Supported case for Q		Supported case for RMi	
		Left tube	Right tube	Left tube	Right tube	Left tube	Right tube	Left tube	Right tube	Left tube	Right tube	Left tube	Right tube
Roof	σ_1 (MPa)	0.50	0.55	0.84	0.84	0.88	0.88	1.80	1.65	1.50	1.50	1.65	1.65
	σ_3 (MPa)	0.02	0.02	0.12	0.15	0.16	0.12	0.04	0.07	0.22	0.22	0.14	0.18
	U_t (m)	6.50e-004	6.50e-004	3.30e-004	3.30e-004	3.85e-004	3.85e-004	7.65e-004	7.65e-004	5.25e-004	5.25e-004	6.40e-04	6.40e-04
Floor	σ_1 (MPa)	0.25	020	0.18	0.24	0.16	0.32	0.90	0.90	0.90	0.90	0.90	0.90
	σ_3 (MPa)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	U_t (m)	1.04e-003	1.04e-003	8.25e-004	9.90e-004	8.80e-004	9.35e-004	1.19e-003	1.19e-003	1.05e-003	1.05e-003	1.12e-003	1.12e-003
Left wall	σ_1 (MPa)	0.30	0.30	0.24	0.24	0.32	0.32	0.45	0.45	0.45	0.45	0.45	0.45
	σ_3 (MPa)	0.02	0.02	0.09	0.09	0.08	0.08	0.00	0.04	0.15	0.15	0.05	0.05
	U_t (m)	1.43e-003	1.04e-003	1.21e-003	9.35e-004	1.27e-003	9.35e-004	1.87e-003	1.28e-003	1.65e-003	1.13e-003	1.76e-003	1.28e-003
Right wall	σ_1 (MPa)	0.30	0.25	0.24	0.24	0.32	0.32	0.45	0.45	0.45	0.45	0.45	0.45
	σ_3 (MPa)	0.00	0.00	0.09	0.09	0.04	0.08	0.00	0.00	0.07	0.15	0.09	0.09
	U_t (m)	1.04e-003	1.50e-003	8.80e-004	1.27e-003	8.80e-004	1.21e-003	1.28e-003	1.96e-003	1.13e-003	1.73e-003	1.20e-003	1.76e-003
R_{pl} (m)	-	6.74	6.57	6.00	6.00	6.00	6.00	6.55	6.00	6.00	6.00	6.00	6.00

U_t = Maximum total displacement (m) and R_{pl} = radius of plastic zone (m).

in entrance portal section of tunnel. Support elements obtained from the Q and RMi classification systems were evaluated for each rock unit. The radius of the plastic zone and maximum total displacement decreased after rock bolt and shotcrete installation. Altough, the radius of the plastic zone decreased after support installation, the maximum total displacement values obtained using RMi are smaller than obtained using Q. These results show that the support system suggested by RMi is more capable than Q to ensure the stability of rock units in entrance portal of tunnel.

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